Geotechnologies, Inc.
Consulting Geotechnical Engineers

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

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Value Schools
680 Wilshire Place, Suite 315
Los Angeles, California 90005

Attention: Enrique Diaz

Subject: Geotechnical Engineering Investigation
Proposed Charter School
233-241 North Westmoreland Avenue, Los Angeles, California

Ladies and Gentlemen:

This letter transmits the Geotechnical Engineering Investigation for the subject property prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
EDWARD F. HILL
No. 2126
Exp. 6/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

EDWARD F. HILL
G.E. 2126

EFH:km

Distribution: (5) Addressee

Email to: [ediaz@redhookcap.com]

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED CHARTER SCHOOL
233-241 NORTH WESTMORELAND AVENUE
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included seven exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the design team. The site is proposed to be developed with a charter school. The structure is proposed to be two stories built at- or near existing site grades. Column loads are estimated to be between 300 and 500 kips. Wall loads are estimated to be between 2 and 6 kips per lineal foot. These loads reflect the dead plus live load, of which the dead load is approximately 75 percent. Smaller structures are also proposed including shade structures and privacy walls. Grading is expected to consist of removal and recompaction of existing unsuitable soils in the area of the building.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The property is located at 233-241 North Westmoreland Avenue in the City of Los Angeles, California. The site is relatively level with very little elevation change. Drainage across the site is by sheetflow to the adjacent improved streets.

The site is currently developed with commercial structure and paved parking. The vegetation on the site consists of a few small trees and shrubs due to the commercial nature of the site.

The neighboring development consists of commercial structures to the north, south and east. A school exists to the west.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on December 28 and 29, 2017 by excavating seven exploratory excavations. The exploratory excavations varied in depth from 20 to 60 feet. The exploration was prosecuted with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers and hand labor. Where hand labor is utilized, the upper reaches of the excavations were on the order of 30 inches square. The deeper portions of the excavations were advanced with a 5-inch hand auger. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-7.

The location of exploratory excavations was determined by information furnished by the client. Elevations of the exploratory excavations were determined by hand level or interpolation from data provided. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Additional Geotechnical Exploration

The site has also been explored by Twining. Twining prepared a report entitled "Additional Geotechnical Investigation", Project Number 190207.1, dated April 12, 2019. That report is based on four geotechnical excavations which varied from 16-1/2 feet to 26-1/2 feet. The excavations encountered between 5 and 7-1/2 feet of fill materials. The borings logs are included herein and boring locations are shown on the plot plan. This data is intended to supplement the subsurface exploration and testing prosecuted by this office.

Existing Structures

Test Pits 1 and 2 were excavated within an existing structure. Test Pit 1 exposed a 9-inch concrete slabs over 4-1/2 inches on base materiel. Test Pit 1 also exposed what appears to be a grade beam. The grade beam was found to be 48 inches in depth. Test Pit 2 encountered an 8-inch concrete slab-on-grade. The Test Pit also exposed what appears to be a pile cap which was on the order of 52 inches in depth. Below the pile cap the shaft of a pile was observed. The pile was observed to be 24 inches in diameter.

Geologic Materials

Fill materials were encountered in each of the exploratory excavations. The fill was found to vary between 2-1/2 and 15 feet in depth. The fill was found to consist of silty sands and sandy clays which are dark grey to yellowish brown, moist, medium dense and fine grained. Cobbles, rock fragments and debris were observed locally. The native soils underlying the site were found to consist of silty sands, sands and silty clays which are dark brown to yellowish brown, moist to wet, medium to very dense, and fine to medium grained.

The geologic materials consist of detrital sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was encountered at depths between 16-1/2 to 25 feet below ambient site grade in the geotechnical excavations. The historic high groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report 026 Plate 1.2 entitled "Historically Highest Ground Water Contours". Review of this plate indicates that the historically highest groundwater level is not well defined in the area of the site. The closest historic high groundwater contour is over 1-1/2 miles to the northwest of the site. That contour indicates a depth of 20 feet below grade.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during exploration due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

The site is underlain by unconsolidated alluvial sediments deposited by river and stream action that are deeper than 200 feet.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines “active” and “potentially active” faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the

CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazards Maps of the State of California (CDMG, 1999), and Reference materials provided by the City of Los Angeles do not classify the site as part of the potentially “Liquefiable” area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

A site-specific liquefaction analysis was performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). The enclosed liquefaction analysis was performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (Blake, 1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on Boring 2. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. Based on the collected SPT data, the enclosed liquefaction analysis indicates that the soils underlying the site would not be capable of liquefaction during the design-based earthquake.

Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

Some seismically-induced settlement of the proposed structures should be expected as a result of strong ground-shaking, however, due to the uniform nature of the underlying geologic materials, excessive differential settlements are not expected to occur.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the City of Los Angeles Inundation and Tsunami Hazard Areas map indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the City of Los Angeles Inundation and Tsunami Hazard Areas map indicates the site appears to lie within mapped inundation boundaries due to a seiche or a breached upgradient reservoir. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference slope geometry across or adjacent to the site.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed charter school is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

The existing fill materials are not suitable for support of the proposed foundations, floor slabs or additional fill. Existing fill materials were found to be a maximum of 15 feet in depth. No geotechnical reports for the site approving the existing fill were encountered during research by ownership or this office.

Exploration in the footprint of the proposed structure by this office and Twining indicates that the building area is underlain by 2-1/2 to 7-1/2 feet in depth. It is recommended that the existing fill should be removed and recompacted for support of the proposed structure.

A plan which appears to address one of the existing site structures was provided to this office. The plan provided is a very poor copy however it appears to address the southerly structure referring to it as Building “C”. The plan was prepared by Harvey Goodman and is dated 1977. Sheet S-1, Detail E appears to indicate that the building is supported on 20 end-bearing piles. The depth of each of the piles appears to be handwritten on the copy which was provided. The depths range from 10-1/2 feet to 25 feet. It is the recommendation of this firm that the existing foundations should be abandoned in place. The upper five feet of the piles should be cut off and removed. The resulting void should be filled with controlled fill.

Smaller structures which are not tied-in to the proposed structure may be supported on conventional foundations bearing in native soils where the existing fill is shallow. Where the fill is deeper, these smaller structures should be supported on friction piles. The piles should penetrate the existing fill to bear in the underlying native soils.

SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a “Stiff Soil” Profile, according to Table 20.3-1 of ASCE 7-10, and ASCE 7-16. This information and the site coordinates were input into the OSHPD seismic utility program at https://seismicmaps.org in order to calculate ground motion parameters for the site.

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS		
California Building Code	2016	2019
ASCE Design Standard	7-10	7-16
Risk Category	I, II & III	II
Site Class	D	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.530g	2.032g
Site Coefficient (F_a)	1.0	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.530g	2.032g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.686g	1.355g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.901g	0.725g
Site Coefficient (F_v)	1.5	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.352g	1.086g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.901g	0.724g*

* *According to ASCE 7-16, a Long Period Site Coefficient (F_v) of 1.7 may be utilized provided that the value of the Seismic Response Coefficient (C_s) is determined by Equation 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or equation 12.8-4 for $T > T_L$. Alternatively, a site-specific ground motion hazard analysis may be performed in accordance with ASCE 7-16 Section 21.1 and/or a ground motion hazard analysis in accordance with ASCE 7-16 Section 21.2 to determine ground motions for any structure.*

FILL SOILS

The maximum depth of fill encountered on the site was 15 feet. The exploration in the footprint of the proposed structure by this office and Twining indicates that the building area is underlain by 2-1/2 to 7-1/2 feet in depth. This material and any fill generated during demolition should be penetrated by proposed foundations.

EXPANSIVE SOILS

The onsite geologic materials are in the low to moderate expansion range. The Expansion Index was found to be between 48 and 82 for bulk samples remolded to 90 percent of the laboratory maximum density. Reinforcing beyond the minimum required by the City of Los Angeles Department of Building and Safety is not required.

WATER-SOLUBLE SULFATES

The Portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually the two most common sources of exposure are from soil and marine environments.

The sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be greater than 0.2% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be severe for geologic materials with greater than 0.2% and Type V cement should be utilized for concrete foundations in contact with the site soils. Additionally, a water-cement ratio of 0.45 should be maintained in the poured concrete and concrete strength should be a minimum of 4,500 psi.

The design of the concrete mix is not within the area of expertise of the geotechnical engineer. It is recommended that a competent engineer familiar with concrete mix design should develop the recommendations for this project based on the tested severe sulfate exposure indicated above.

METHANE ZONES

This office has reviewed the City of Los Angeles Methane Zone and Methane Buffer Zones map. Based on this review it appears that the subject property is located within a Methane Buffer Zone as designated by the City. A qualified methane consultant should be retained to consider the requirements and implications of the City's Methane Buffer Zone designation. A copy of the portion of the map covering the Project Site is included herein.

GRADING GUIDELINES

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. The soils tested by this firm would require the 95 percent compaction requirement.

Comparative compaction is defined, for purposes of these guidelines, as the ratio of the in-place density to the maximum density as determined by applicable ASTM testing.

All fill should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditions to within 3 percent of the optimum moisture content of the particular material placed. All fill shall be compacted to at least 90 or 95 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. in general accordance with the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 or 95 percent compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed.

Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 20. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 or 95 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in general accordance with the most recent revision of ASTM D 1557.

Wet Soils

At the time of exploration the soils which will be exposed during grading were locally above optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane may require significant drying and aeration prior to recompaction.

Pumping (yielding or vertical deflection) of the high-moisture content soils at the bottom of the excavation may occur during operation of heavy equipment. Where pumping is encountered, angular minimum ¾-inch gravel and/or crushed concrete should be placed and worked into the subgrade. The exact thickness of the gravel would be a trial and error procedure, and would be determined in the field. It would likely be on the order of 1 to 2 feet thick.

The gravel will help to densify the subgrade as well as function as a stabilization material upon which heavy equipment may operate. It is not recommended that rubber tire construction equipment attempt to operate directly on the pumping subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which in turn will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Abandoned Seepage Pits

No abandoned seepage pits were encountered during exploration and none are known to exist on the site. However, should such a structure be encountered during grading, options to permanently abandon seepage pits include complete removal and backfill of the excavation with compacted fill, or drilling out the loose materials and backfilling to within a few feet of grade with slurry, followed by a compacted fill cap.

If the subsurface structures are to be removed by grading, the entire structure should be demolished. The resulting void may be refilled with compacted soil. Concrete and brick generated during the seepage pit removal may be reused in the fill as long as all fragments are less than 6 inches in longest dimension and the debris comprises less than 15 percent of the fill by volume. All grading should comply with the recommendations of this report.

Where the seepage pit structure is to be left in place, the seepage pits should cleaned of all soil and debris. This may be accomplished by drilling. The pits should be filled with minimum 1-1/2 sack concrete slurry to within 5 feet of the bottom of the proposed foundations. In order to provide a more uniform foundation condition, the remainder of the void should be filled with controlled fill.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

LEED Considerations

The Leadership in Energy and Environmental Design (LEED) Green Building Rating System encourages adoption of sustainable green building and development practices. Credit for LEED Certification can be assigned for reuse of construction waste and diversion of materials from landfills in new construction.

In an effort to provide the design team with a viable option in this regard, demolition debris could be crushed onsite in order to use it in the ongoing grading operations. The environmental ramifications of this option, if any, should be considered by the team.

The demolition debris should be limited to concrete, asphalt and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials and wood.

For structural fill applications, the materials should be crushed to 2 inches in maximum dimension or smaller. The crushed materials should be thoroughly blended and mixed with onsite soils prior to placement as compacted fill. The amount of crushed material should not exceed 20 percent. The blended and mixed materials should be tested by this office prior to placement to insure it is suitable for compaction purposes. The blended and mixed materials should be tested by Geotechnologies, Inc. during placement to insure that it has been compacted in a suitable manner.

FOUNDATION DESIGN

Conventional Foundations

Conventional foundations may bear in newly placed controlled fill or native soils. All conventional foundations for a structure should bear in the same material.

Continuous foundations may be designed for a bearing capacity of 2,500 pounds per square foot and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 3,000 pounds per square foot and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 75 pounds per square foot. The bearing capacity increase for each additional foot of depth is 340 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

The bearing capacities indicated above are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Deepened Footings

Where the recommended overexcavation cannot be prosecuted such as adjacent to existing buildings or property lines, foundations will require deepening to bear in competent native soils. The deepened portion of the footings may be filled with concrete of the same mix as that specified for the footing. The initial pour would not require reinforcing as it is simply passing the load through to the recommended bearing material. Once the initial pour has hardened, the footing may be reinforced and poured on top of the first pour. Some method of creating a positive bond between the two pours should be employed. Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

Foundation Reinforcement

Based on City of Los Angeles minimum requirements all continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.28 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot.

The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be three quarters of an inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed one quarter of an inch.

Modulus of Subgrade Reaction

A unit modulus of subgrade reaction of 300 pounds per cubic inch (518 kcf) may be utilized for design of foundations. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with the larger footings:

$K = K_1*[(B+1)/(2*B)]^2$

Where:
K = Reduced Subgrade Modulus
K_1 = Unit Subgrade Modulus
B = Foundation Width (feet)

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory geologic materials, if necessary.

Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

FRICTION PILE FOUNDATIONS

Vertical Capacities

A deepened foundation system consisting of friction piles should be utilized for support of the smaller structures which are not tied-in to the proposed school structure. The capacities of drilled cast-in-place piles are shown on the enclosed “Drilled Cast in Place Pile Capacities” chart.

Capacities based on dead plus live load are indicated. A one-third increase may be used for transient loading such as wind or seismic forces. The capacities presented are based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Piles in groups should be spaced at least 2-1/2 diameters on center. If the piles are so spaced, no reduction in the downward or upward capacities need be considered due to group action.

Lateral Design

Lateral loads may be resisted by the piles, and by the passive resistance of the soils against the pile caps. The passive resistance of the existing soils against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in this value may be used for wind or seismic loads. The resistance of the piles, and the passive resistance of the soils against pile caps and grade beams may be combined without reduction in determining the total lateral resistance.

Maximum recommended allowable lateral capacities for 1/2 inch deflection of fixed and free-head friction piles are presented on the enclosed table, "Lateral Load Capacities of Drilled Cast-In-Place Piles," in the Appendix of this report. No factors of safety have been applied to the lateral load values calculated to induce 1/2-inch lateral deflection. Lateral capacities provided are for drilled cast-in-place piles, penetrating the materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 pounds per square inch (psi), and minimum pile lengths equal to the depths to maximum moment indicated.

Maximum recommended allowable lateral capacities for 0.5-inch deflection for single, isolated, fixed-head and free-head piles are presented in the Appendix. No factors of safety have been applied to the lateral load values calculated to induce the calculated lateral deflection. Lateral capacities provided are for concrete piles embedded into the underlying native soils encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 pounds per square inch (psi).

Single isolated piles may be classified as piles spaced at or greater than 8 widths on center. For pile groups where piles will be spaced closer than 8 diameters on center in the direction of loading, the following reduction factor may be utilized to determine the allowable lateral pile capacities for the trailing piles to maintain the 0.5-inch pile deflection.

Pile Spacing	Percentage of Lateral Passive Resistance
7B	85%
5B	55%
2-1/2B	25%

Where B is the diameter of the proposed piles.

A one-third increase may be used for transient loading such as wind or seismic forces. The capacities presented are based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Pile Installation

Due to the nature of the existing geologic materials encountered during exploration, significant caving is not anticipated during drilling of the proposed piles above the water table. Where the bottom of the proposed piles will be below the water level, casing or the use of drilling mud will be required in order to achieve the required depth and maintain an open hole to allow the placement of the steel and concrete. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least overnight before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the shafts should not be left open overnight.

Settlement

The maximum settlement of pile-supported foundations is not expected to exceed 1/2 inch. Differential settlement is expected to be negligible.

RETAINING WALL DESIGN

Cantilever Retaining Walls

Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 30 pounds per cubic foot for walls retaining up to 6 feet of earth.

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Retaining Wall Drainage

Subdrains may consist of 4-inch diameter perforated pipes, places with perforated facing down. The pipe shall be encased in at least one foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one-inch crushed rocks. As an alternative, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to once inch crushed rocks, wrapped in filter fabric.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

Where retaining walls are to be constructed adjacent to property lines there is usually not enough space for emplacement of a standard pipe and gravel drainage system. Under these circumstances, the use of a flat drainage produce is acceptable.

Some municipalities do not allow the use of flat-drainage products. The use of such a product should be researched with the building official. As an alternative, omission of one-half of a block at the back of the wall on eight foot centers is an acceptable method of draining the walls. The resulting void should be filled with gravel. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard pipe and gravel system constructed under the slab. This method should be approved by the retaining wall designer prior to implementation.

Dynamic (Seismic) Earth Pressure

The maximum dynamic active pressure is equal to the sum of the initial static pressure and the dynamic (seismic) pressure increment. Under the most recent building code, as interpreted by most building departments, seismic earth pressure is required in the design of restraining walls which support over 6 feet of earth. The proposed walls are less than 6 feet in height therefore the dynamic earth pressure may be omitted.

Surcharge from Adjacent Structures

As indicated herein, additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures for retaining walls and shoring design.

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2008-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

Resultant lateral force: $R = (0.3*P*h^2)/(x^2+h^2)$

Location of lateral resultant: $d = x*[(x^2/h^2+1)*\tan^{-1}(h/x)-(x/h)]$

where:

R	=	resultant lateral force measured in pounds per foot of wall width.
P	=	resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall.
x	=	distance of resultant load from back face of wall measured in feet.
h	=	depth below point of application of surcharge loading to top of wall footing measured in feet.
d	=	depth of lateral resultant below point of application of surcharge loading measure in feet.
$\tan^{-1}(h/x)$	=	the angle in radians whose tangent is equal to h/x.

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 or 95 percent of the maximum density in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

Excavations on the order of 3 to 5 feet in vertical height may be required. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SLABS ON GRADE

Concrete floor slabs should derive all support from the pile foundations.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 or 95 percent of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, where necessary, it is recommended that a qualified consultant should be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor on various components of the structure.

Where any dampness would be objectionable or where the slab will be cast below the historic high groundwater level, it is recommended that floor slabs should be waterproofed. A qualified waterproofing consultant should be engaged in order to recommend a product and/or method which would provide protection from unwanted moisture.

Based on ACI 302.2R-30, Chapter 7, for projects which do not have vapor sensitive coverings or humidity controlled areas, a vapor retarder is not necessary. Where a vapor retarder is considered necessary, the design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements. The necessity of a vapor retarder is not a geotechnical issue and should be confirmed by qualified members of the design team.

Based on ACI 302.2R-30, Chapter 7, for projects with vapor sensitive coverings, a vapor barrier should be provided. Figure 7.1 shows that the slab should be poured on the vapor barrier. Where humidity controlled areas are proposed and the base materials and slabs will not be within a water-tight system, Figure 7.1 shows that the barrier should be covered with a 4 inch layer of dry granular material. ACI notes that the decision whether to locate the material in direct contact with the slab or beneath a layer of granular fill should be made on a case by case basis. The necessity of a vapor retarder as well as the use of dry granular material, as discussed above, is not a geotechnical issue and should be confirmed by qualified members of the design team.

ACI 302.2R-30, Chapter 7 discusses benefits derived from concrete poured on a granular layer as well as directly on the vapor retarder. Changes to the concrete used, such as slump, mix or admixtures are also discussed. This is also not a geotechnical issue and should be confirmed by qualified members of the design team. It is the recommendation of this firm that the design team become familiar with ACI 302.2R-30, Chapter 7.

Groundwater was encountered on the subject site at a depth of 16-1/2 feet. Proposed concrete slabs-on-grade do not need to be supported on a layer of compacted aggregate to provide a capillary break.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way.

Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 or 95 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	4
Moderate Truck	4	6

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should consist of Crushed Aggregate Base which conform with Section 200-2.2 of the most recent edition of "Standard Specifications for Public Works Construction", (Green Book).

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base. In addition where landscaping is planned adjacent to pavement, it is recommended that a cutoff wall should be provided along the edge of the pavement. The cutoff wall should extend at least 12 inches below the depth of the base course.

The management of pavement wear primarily is focused on the distress caused by vertical loads. The reduction of vertical loading from large vehicles is assisted by increasing the number of axles. Multi-axle groups reduce the peak vertical loading and, when closely spaced, reduce the magnitude of the strain cycles to which the pavement is subjected. However, where tight low-speed turns are executed, non-steering axle groups lead to transverse shear forces (scuffing) at the pavement-tire interface.

With asphaltic concrete pavements, tensile shear stresses from tires can cause surface cracking and raveling, thus, the increased use of non-steering axle groups results in increased pavement wear in the vicinity of intersections and turnarounds where tight low speed turns are executed.

When designing intersections and turnarounds the turn radius should be as large as possible. This will lead to reduced “scuffing” forces. Where tight radius turns are unavoidable, the pavement surface design should take into account the high level of “scuffing” forces that will occur and thickened pavement and subgrade and base course keyways should be considered to assist in the reduction of lateral deflection.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Due to the deep fill and shallow groundwater encountered during exploration, stormwater infiltration would not be recommended for this site.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction.

This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.

EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might effect the proposed development.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. Samples from bucket-auger drilling are obtained utilizing a California Modified Sampler with successive 12-inch drops of a kelly bar, whose weight is noted on the excavation logs. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in general accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve.

General accordance with the most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples in general accordance with the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations.

The dry unit weight is determined in pounds per cubic foot and shown on the “Excavation Logs”, A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed in general accordance with the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests in general accordance with the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined in general accordance with the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.

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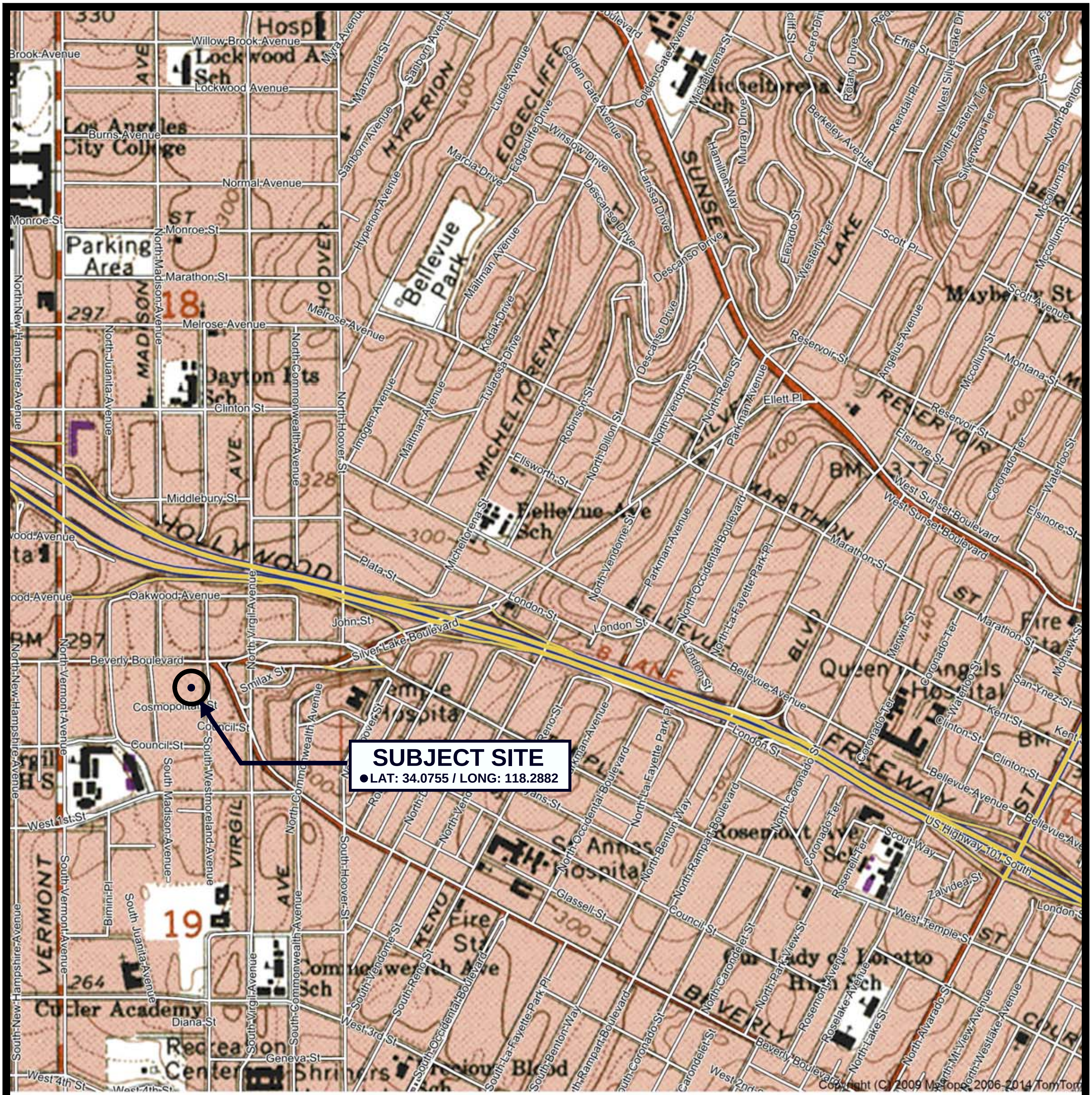


SCALE 1:12000

0.0 | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | 0.6

MILES

0 | 1000 | 2000 | 3000

FEET

N

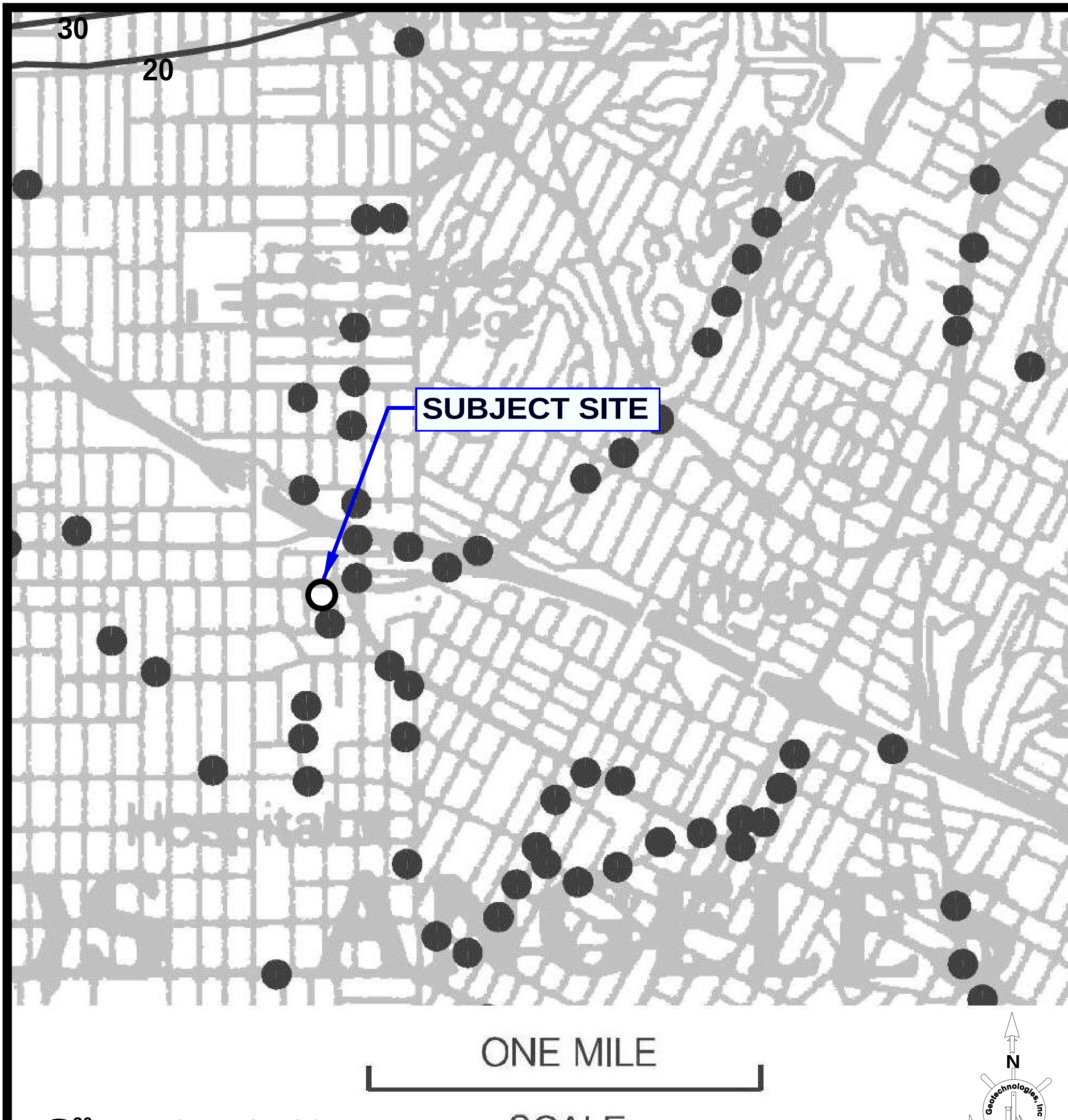
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS

FILE NO. 21536



20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 026
HOLLYWOOD 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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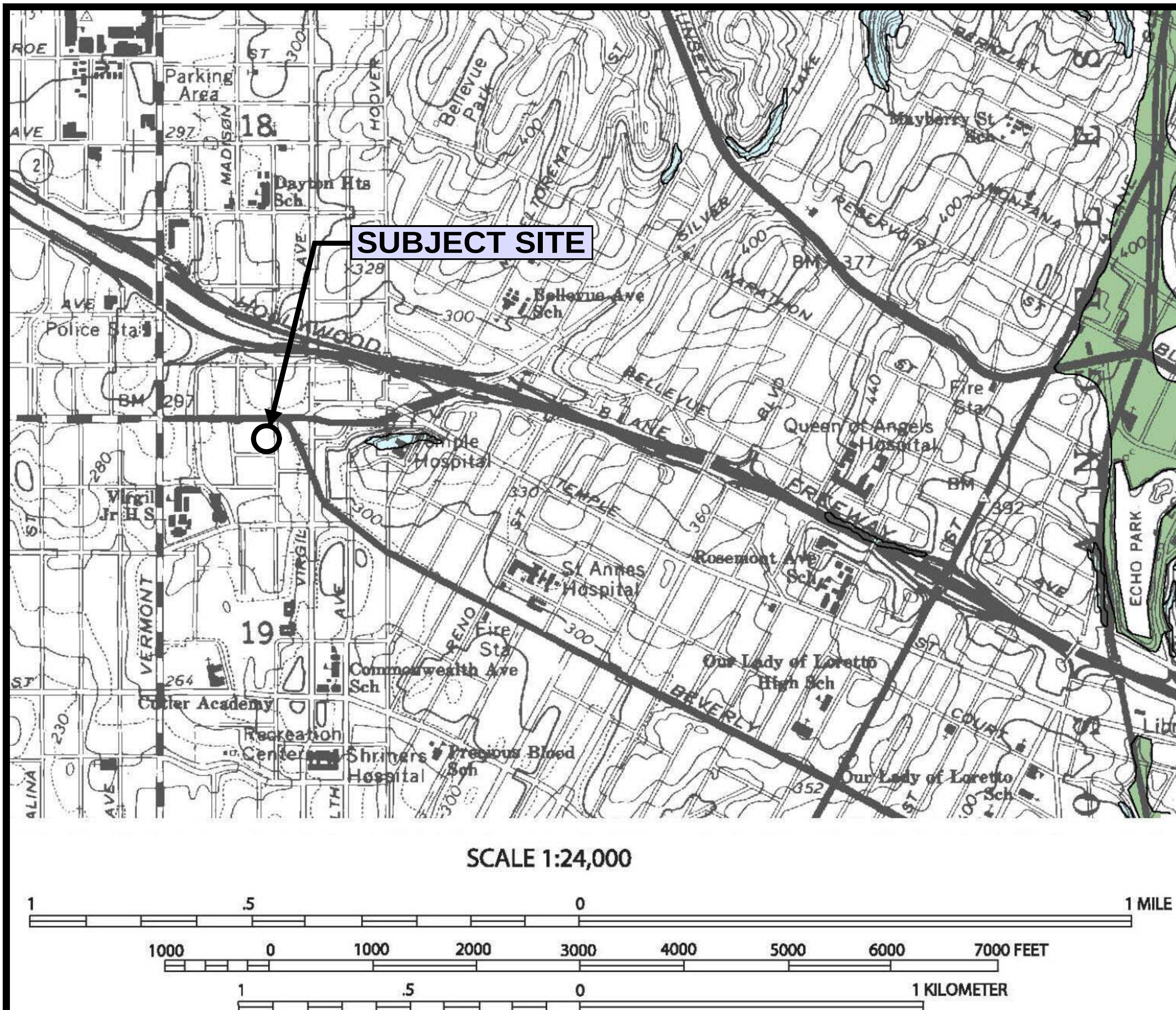
VALUE SCHOOLS

FILE NO. 21536



LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, HOLLYWOOD QUADRANGLE OFFICIAL MAP (CDMG, 1999)

SEISMIC HAZARD ZONE MAP

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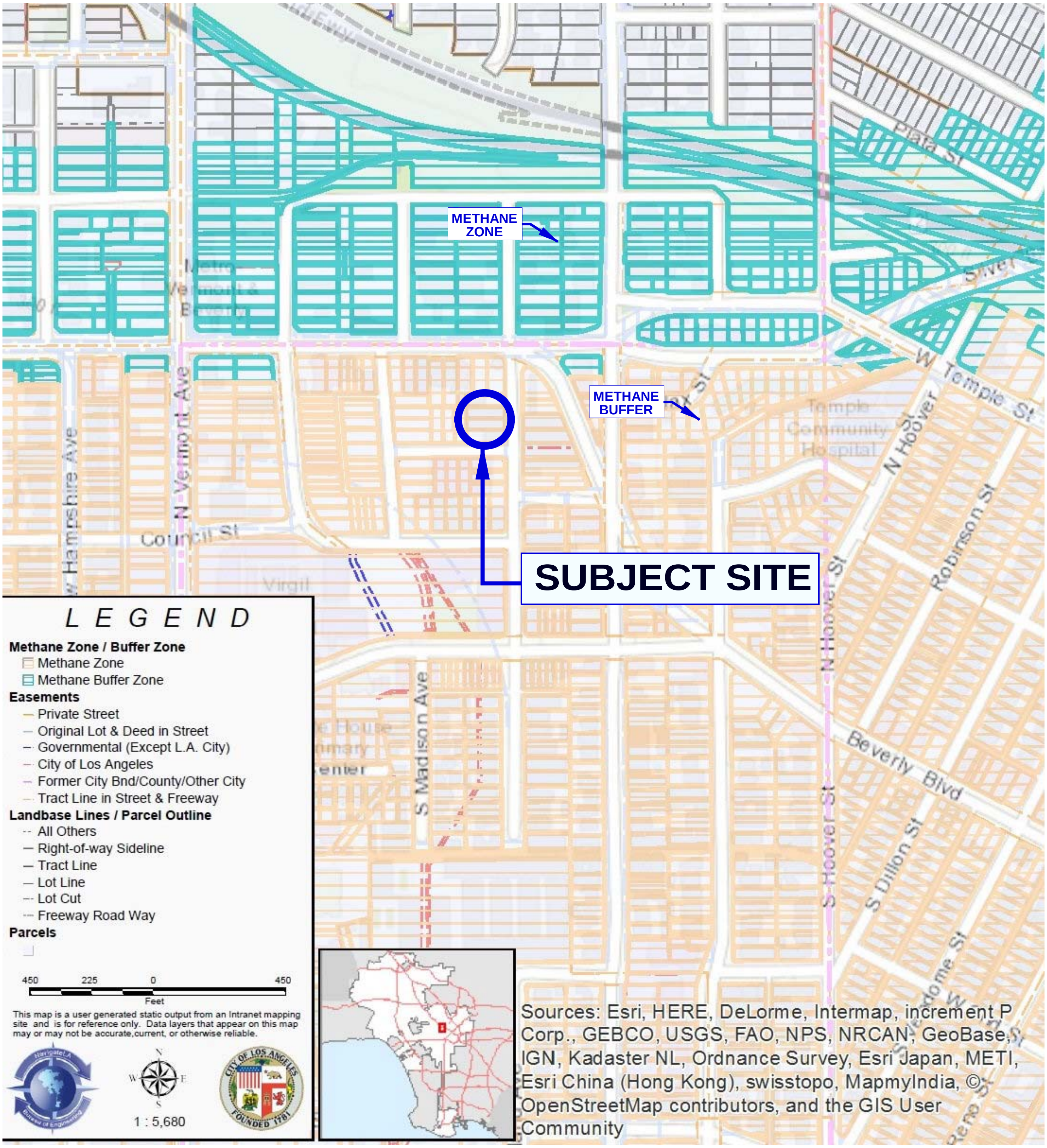
Consulting Geotechnical Engineers

VALUE SCHOOLS

FILE NO. 21536



REFERENCE: **http://navigatela.lacity.org/NavigateLA/**

METHANE ZONE RISK MAP

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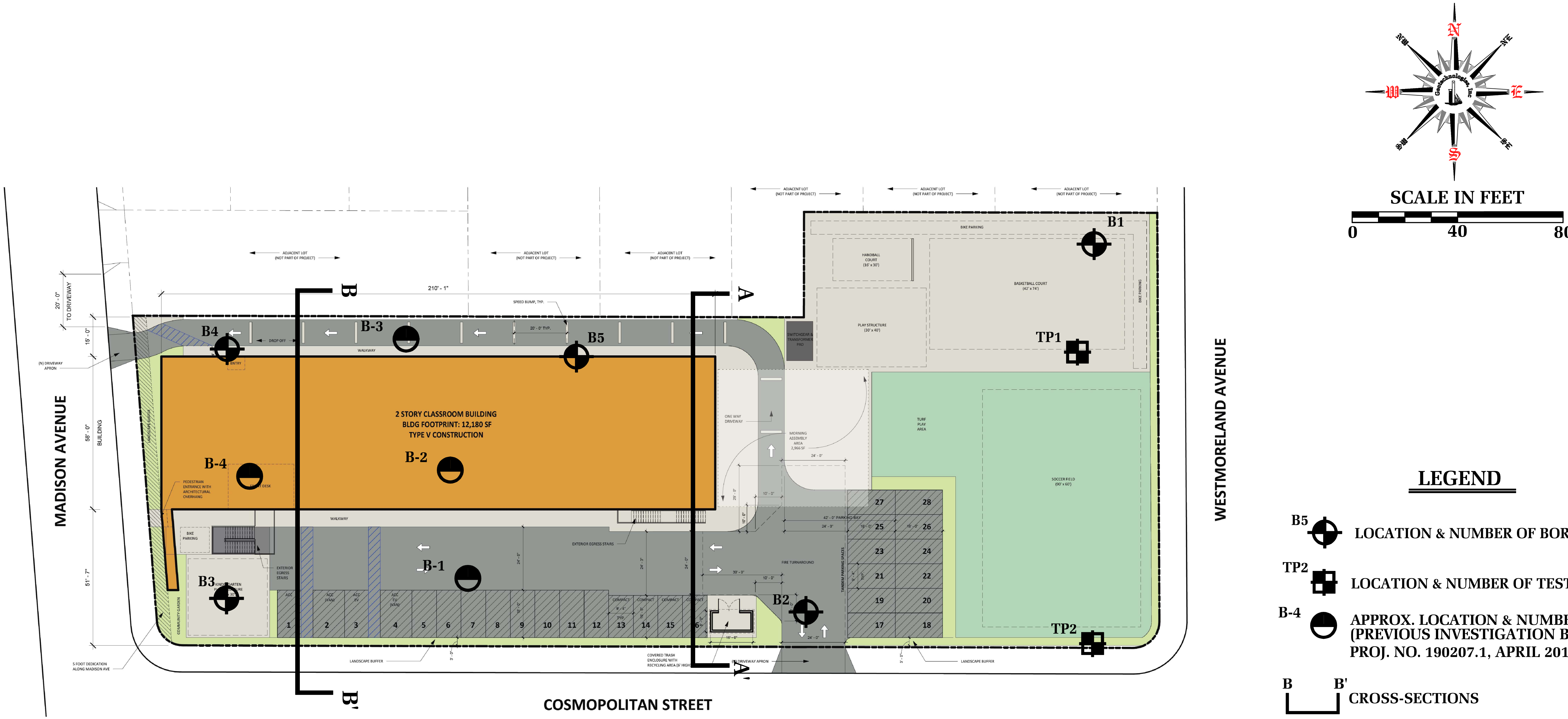
VALUE SCHOOLS

FILE NO. 21536



LEGEND

B5 — LOCATION & NUMBER OF BORING

TP2 — LOCATION & NUMBER OF TEST PIT

B-4 — APPROX. LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION BY TWINING, PROJ. NO. 190207.1, APRIL 2019)

B B' — CROSS-SECTIONS

REFERENCE: PROPOSED NEW CONSTRUCTION SITE PLAN PROVIDED BY CSDA DESIGN GROUP DATED NOVEMBER 12, 2019

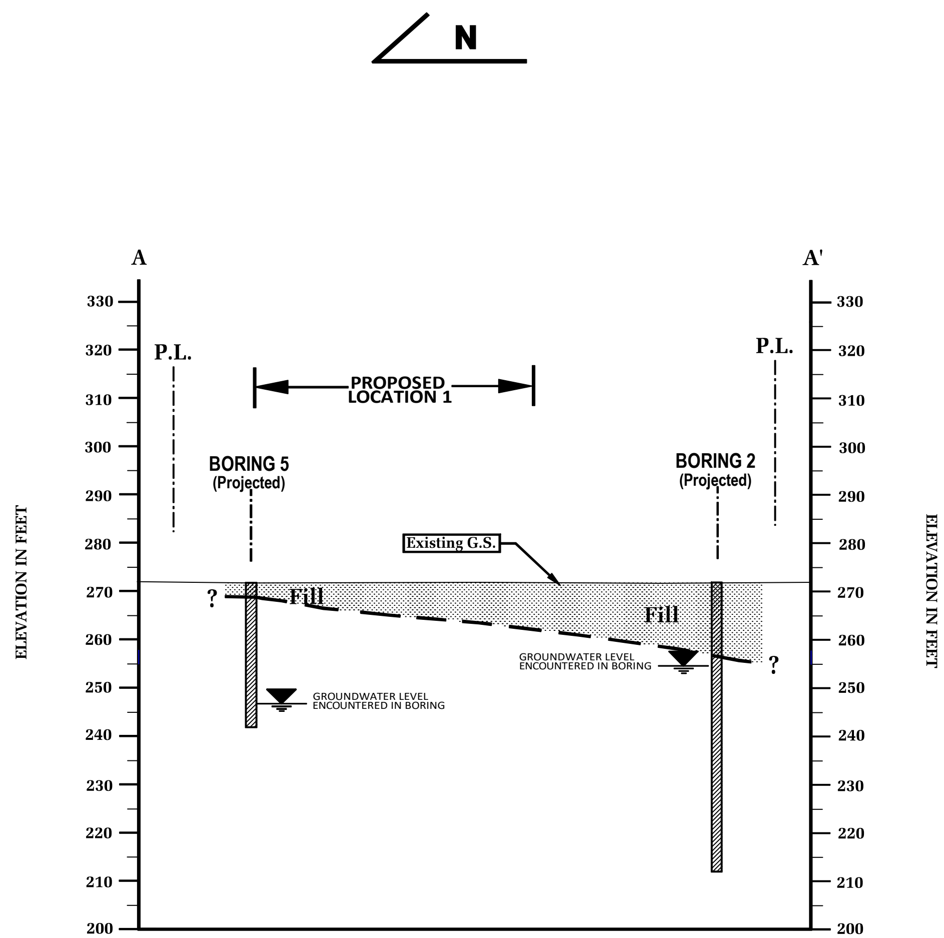
PLOT PLAN

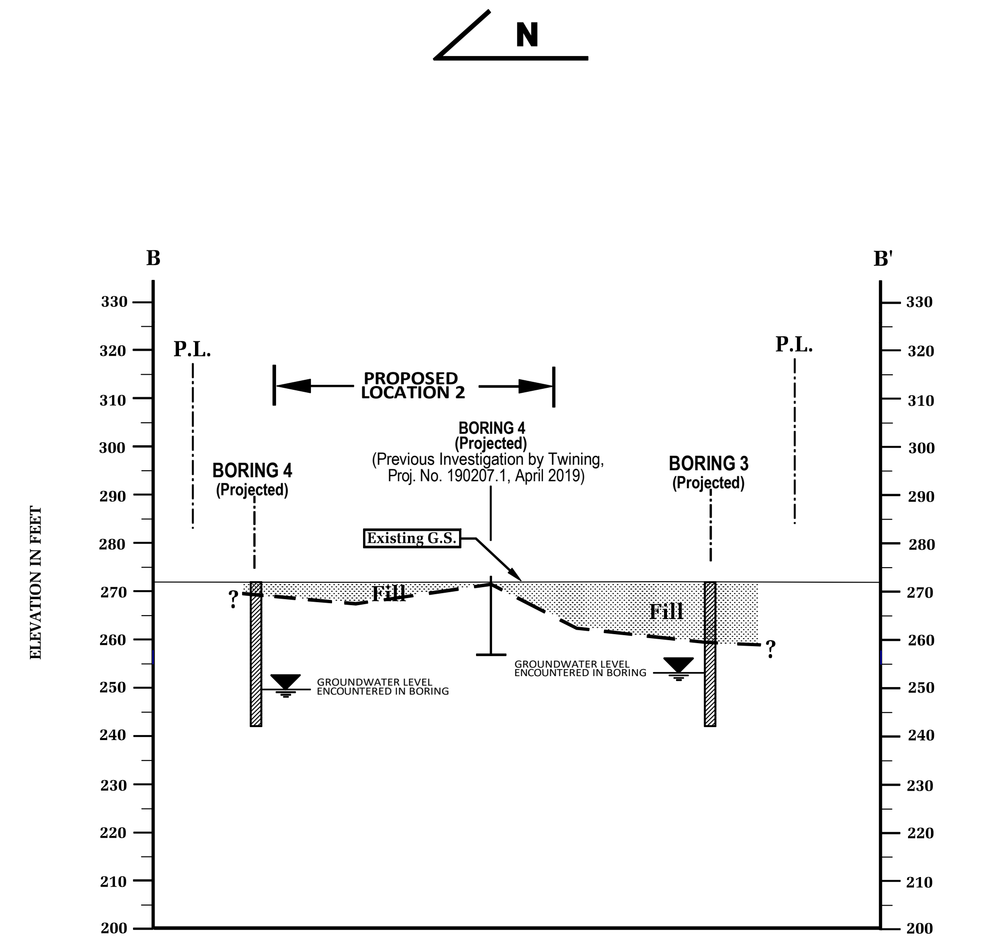
Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS
233 - 241 N. WESTMORELAND AVE., LOS ANGELES

FILE No. 21536 | DRAWN BY: TC

DATE: December 2019





CROSS-SECTIONS A-A' & B-B'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS
233 - 241 N. WESTMORELAND AVE., LOS ANGELES

FILE No. 21536	DRAWN BY: TC
DATE: January 2020	

BORING LOG NUMBER 1

Value Schools | **Date: 12/29/17**

File No. 21536 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete for Loading Dock
				0 --		7-inch Concrete over 3-inch Base
				-		
				1 --		
				-		FILL: Silty Sand, dark brown, moist, medium dense, fine
				2 --		grained, with cobbles
2.55	36	16.9	110.1	-		
				3 --		Silty Sand to Silty Clay, dark and yellowish brown, moist,
				-		medium dense, fine grained, stiff
				4 --		
				-		
5	17	16.3	113.6	5 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense,
				6 --		fine grained, stiff
				-		
				7 --		
7.5	22	17.9	110.5	-		
				8 --	SM/CL	Silty Sand to Silty Clay, dark and yellowish brown, moist,
				-		medium dense, fine grained, stiff
				9 --		
				-		
10	79	12.8	118.1	10 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, very
				11 --		dense, fine to medium grained
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	72	15.0	112.6	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/8"	16.1	108.2	20 --		
				-	SP	Sand, dark brown, moist, very dense, fine grained
				21 --		
				-		NOTE: The stratification lines represent the approximate
				22 --		boundary between earth types; the transition may be gradual.
				-		
				23 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				24 --		Modified California Sampler used unless otherwise noted
				-		
25	100/8"	17.2	107.2	25 --		
				-		Total Depth 25 feet; Water at 22 feet; Fill to 5 feet

BORING LOG NUMBER 2

Value Schools | **Date: 12/28/17**

File No. 21536 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 5-inch Base
				-		
				1 --		
				-		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium
				2 --		dense, fine grained, stiff
2.5	18	18.8	105.2	-		
				3 --		Silty Sand to Sandy Silt, dark brown, moist, medium dense,
				-		fine grained, stiff, minor asphalt fragments
				4 --		
				-		
5	13	17.6	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	12	21.6	98.6	-		
				8 --		Sandy to Clayey Silt, dark brown, moist
				-		
				9 --		
				-		
10	6	32.1	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	23	23.0	102.7	-		
				13 --		Clayey Silt to Silty Clay, dark and gray, moist, stiff
				-		
				14 --		
				-		
15	26	18.1	SPT	15 --		
				-	SM	Silty Sand, dark gray, very moist, medium dense, fine grained
				16 --		
				-		
				17 --		
17.5	46	20.5	105.1	-		
				18 --		Silty Sand to Sand, dark brown, moist, medium dense, fine
				-		grained
				19 --		
				-		
20	78	15.8	SPT	20 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, very dense, fine grained
				21 --		
				-		
				22 --		
22.5	35	21.0	104.0	-		
	50/4"			23 --		Silty Sand to Sand, dark brown, moist, very dense, fine grained
				-		
				24 --		
				-		
25	80	19.7	SPT	25 --		
				-		

BORING LOG NUMBER 2

Value Schools

File No. 21536

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	46	7.6	120.4	-		
	50/3"			28 --		Silty Sand to Sand, dark gray, moist, very dense, fine grained, minor tar
				-		
				29 --		
				-		
30	29	12.3	SPT	30 --		
	50/5"			-		
				31 --		
				-		
				32 --		
32.5	100/9"	9.2	117.4	-		
				33 --	SM	Silty Sand, dark and gray, moist, very dense, fine grained, odor
				-		
				34 --		
				-		
35	35	13.8	SPT	35 --		
	50/4"			-	SM/SP	Silty Sand to Sand, dark gray, moist, very dense, fine grained
				36 --		
				-		
				37 --		
37.5	100/9"	18.8	105.9	-		
				38 --		
				-		
				39 --		
				-		
40	40	22.7	SPT	40 --		
	50/4"			-	SM/ML	Silty Sand to Sandy Silt, dark and grayish brown, moist, very dense, fine grained, very stiff
				41 --		
				-		
				42 --		
42.5	38	9.7	116.2	-		
	50/5"			43 --		
				-		
				44 --		
				-		
45	40	15.6	SPT	45 --		
	50/4"			-	SM/SP	Silty Sand to Sand, dark and gray, moist, very dense, fine grained
				46 --		
				-		
				47 --		
47.5	100/8"	16.8	109.1	-		
				48 --		
				-		
				49 --		
				-		
50	32	15.2	SPT	50 --		
	50/3"			-		

BORING LOG NUMBER 2

Value Schools

File No. 21536

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	100/8"	14.4	113.9			
				53		
				54		
55	30 50/2"	16.0	SPT	55		
				56		
				57		
57.5	100/8"	11.6	112.5			
				58		
				59		
60	40 50/5"	17.7	SPT	60		
				61		Total Depth 60 feet Water at 17 feet Fill to 15 feet
				62		
				63		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				64		
				65		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				66		
				67		SPT=Standard Penetration Test
				68		
				69		
				70		
				71		
				72		
				73		
				74		
				75		

BORING LOG NUMBER 3

Value Schools **Date: 12/28/17**

File No. 21536 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	42	24.0	91.4	-		
				3 --		Sandy Silt, dark and gray, moist, stiff, with rock fragments
				-		
				4 --		
				-		
5	19	23.7	90.2	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	16	28.6	83.5	-		
				8 --		
				-		
				9 --		
				-		
10	19	30.0	86.9	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	27	13.7	115.4	-		
				13 --	ML	Sandy Silt, dark brown, moist, stiff
				-		
				14 --		
				-		
15	46	15.6	107.8	15 --		
				-	SM/SP	Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	79	14.4	112.9	20 --		
				-		Silty Sand to Sand, dark brown, moist, very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	100/8"	8.0	103.9	25 --		
				-		Silty Sand, dark brown, moist, very dense, odor

BORING LOG NUMBER 3

Value Schools

File No. 21536

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
				-		Total Depth 30 feet
				31 --		Water at 19 feet
				-		Fill to 12½ feet
				32 --		
				-		
				33 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				34 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				35 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 4

Value Schools **Date: 12/28/17**

File No. 21536 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 5-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	18	19.4	100.7	-		
				3 --	SM/CL	Silty Sand to Silty Clay, dark brown, moist, stiff
				-		
				4 --		
				-		
5	26	17.6	105.5	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	37	12.6	109.0	-		
				8 --	ML/SP	Sandy to Clayey Silt to Sand, dark to yellowish brown, moist, medium dense, fine grained, stiff
				-		
				9 --		
				-		
10	52	8.9	112.1	10 --		
				-	SP/SM	Silty Sand to Sand, dark and yellowish brown, moist, medium dense to dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	78	17.1	113.3	15 --		
				-	SP	Sand, dark brown, moist, very dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/9"	19.7	104.2	20 --		
				-		Sand, dark brown and gray, moist, very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	100/10"	6.1	122.2	25 --		
				-	SM/SP	Silty Sand to Sand, dark gray, moist, very dense, fine grained

BORING LOG NUMBER 4

Value Schools

File No. 21536

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
				-		Total Depth 30 feet
				31 --		Water at 22½ feet
				-		Fill to 2½ feet
				32 --		
				-		
				33 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				34 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				35 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 5

Value Schools **Date: 12/28/17**

File No. 21536 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		5-inch Asphalt over 5-inch Base
				-		
				1 --		FILL: Sandy to Clayey Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	34	18.2	106.5	-		
				3 --		
				-	ML/SM	Silty Sand to Sandy Silt, dark brown, moist, stiff
				4 --		
				-		
5	18	14.9	107.6	5 --		
				-	SM	Silty Sand, dark and yellowish brown, moist, medium dense, fine grained
				6 --		
				-		
				7 --		
7.5	25	17.1	96.8	-		
				8 --		
				-		
				9 --		
				-		
10	23	24.5	101.6	10 --		
				-	ML	Sandy Silt, dark brown, moist, stiff
				11 --		
				-		
				12 --		
12.5	50	24.5	101.7	-		
				13 --	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
				-		
				14 --		
				-		
15	48	18.4	104.7	15 --		
				-	ML	Sandy to Clayey Silt, dark brown to yellowish brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	80	25.7	97.7	20 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	100/9"	12.4	94.1	25 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, very dense, fine grained

BORING LOG NUMBER 5

Value Schools

File No. 21536

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
				-		Total Depth 30 feet
				31 --		Water at 25 feet
				-		Fill to 3 feet
				32 --		
				-		
				33 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				34 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				35 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

LOG OF TEST PIT NUMBER 1

Value Schools | **Drilling Date: 12/28/17**

File No. 21536 | **Method: Hand Dug Test Pit**

km

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete Slab
			0 --		9-inch Concrete over 4½-inch Concrete, No Base
1	22.4	102.7	1 --		FILL: Sandy Silt to Silty Sand, dark brown, moist, stiff
			2 --		
3	11.9	111.8	3 --		Silty Sand to Silty Clay, dark brown, moist, stiff
			4 --		
5	12.1	111.0	5 --		Sandy Silt, dark brown, moist, stiff, minor cobbles
			6 --		
			7 --		
7.5	12.9	115.7			Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained
			8 --		
			9 --		
10	34.2	88.6	10 --		Clayey Silt to Silty Clay, dark brown, moist, stiff
			11 --		
			12 --		
			13 --	SM/ML	Silty Sand to Sandy Silt, gray to dark gray, moist, medium dense, fine grained, stiff
			14 --		
15	17.7	112.1	15 --		
			16 --		
			17 --		
			18 --	SM	Silty Sand, dark brown, moist, medium dense to dense, fine grained
			19 --	SP	Sand, dark brown, moist, medium dense, fine to medium grained
20	14.1	115.9	20 --		Total Depth 20 feet No Water Fill to 13 feet
			21 --		
			22 --		
			23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			24 --		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			25 --		

LOG OF TEST PIT NUMBER 2

Value Schools — **Drilling Date: 12/28/17**

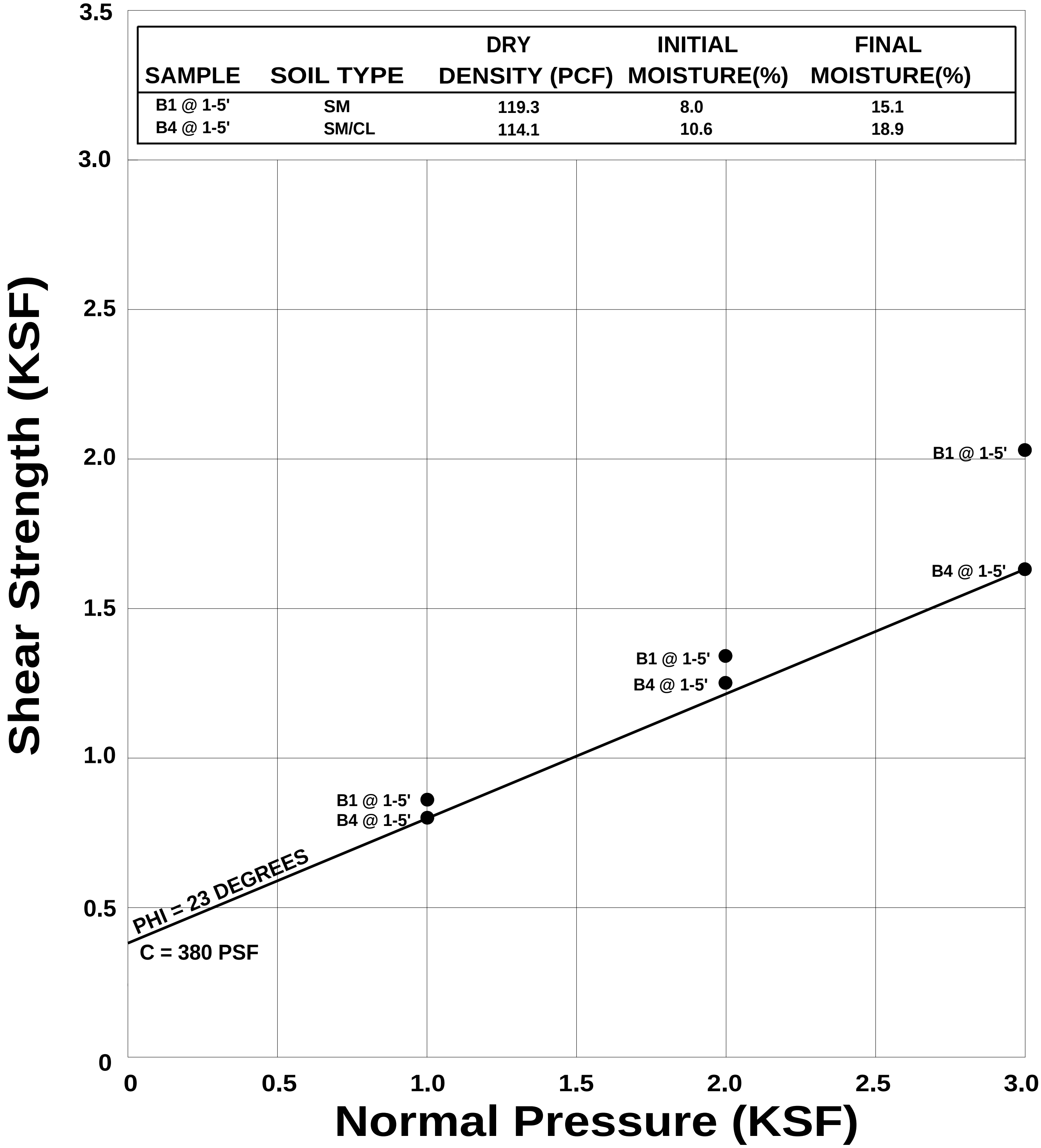
File No. 21536 — **Method: Hand Dug Test Pit**

km

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete Slab
			0		8-inch Concrete over 3-inch Base
			1		
2	10.0	105.7	2		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
			3		
4	11.8	108.3	4		
			5		
			6		
7	15.7	109.9	7		
			8		Sandy Silt, dark brown, moist stiff, minor rock fragments
			9		
10	28.1	92.5	10		
			11		
			12		
			13		
			14		
15	17.9	105.3	15		
			16	SM/SP	Silty Sand to Sand, dark gray, moist, medium dense, fine grained
			17		
			18		
			19		
20	16.7	105.4	20	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
			21		Total Depth 20 feet Water at 16½ feet Fill to 15 feet
			22		
			23		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			24		
			25		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	SM	119.3	8.0	15.1
B4 @ 1-5'	SM/CL	114.1	10.6	18.9

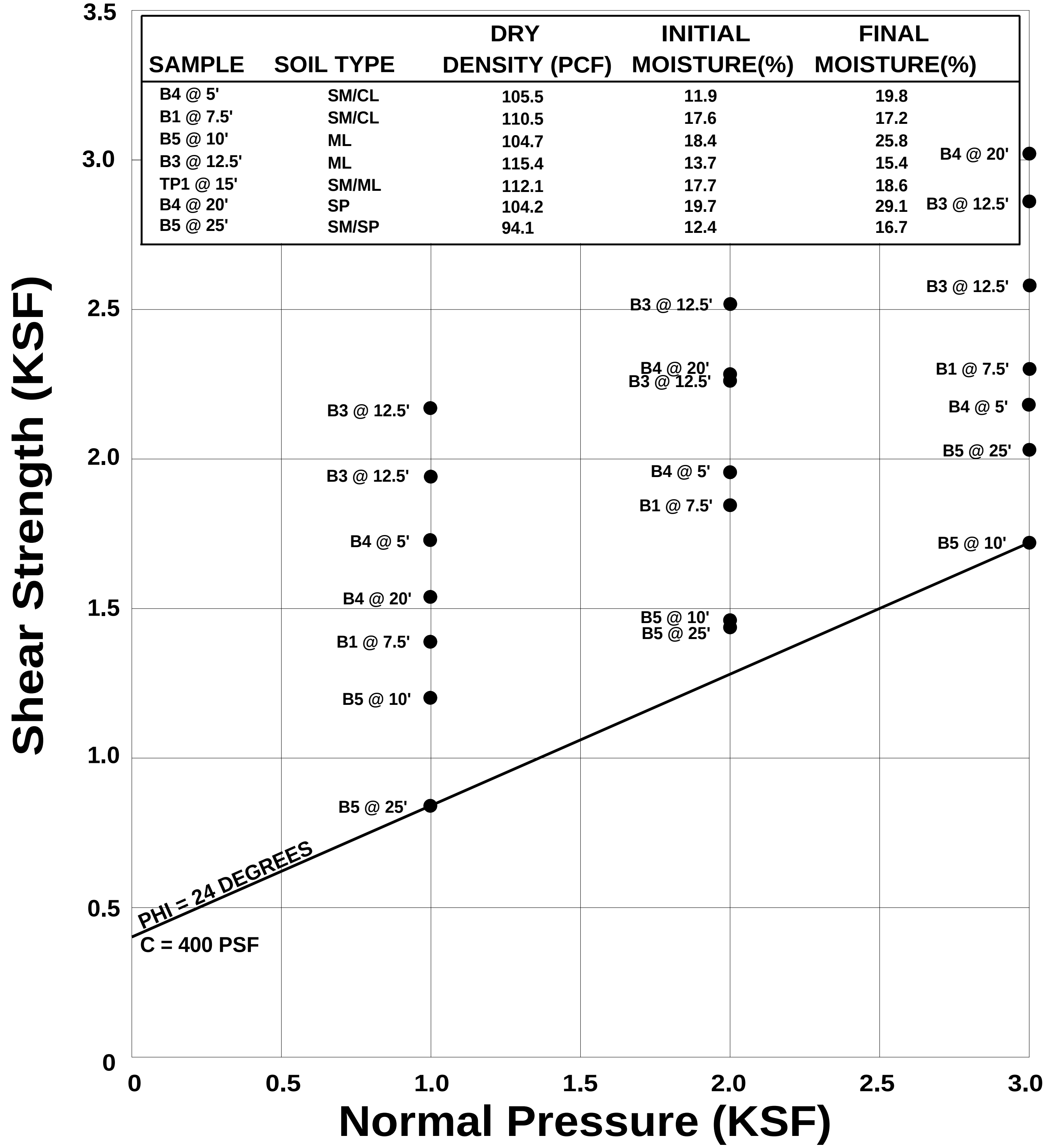


● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS	
FILE NO. 21536	PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B4 @ 5'	SM/CL	105.5	11.9	19.8
B1 @ 7.5'	SM/CL	110.5	17.6	17.2
B5 @ 10'	ML	104.7	18.4	25.8
B3 @ 12.5'	ML	115.4	13.7	15.4
TP1 @ 15'	SM/ML	112.1	17.7	18.6
B4 @ 20'	SP	104.2	19.7	29.1
B5 @ 25'	SM/SP	94.1	12.4	16.7

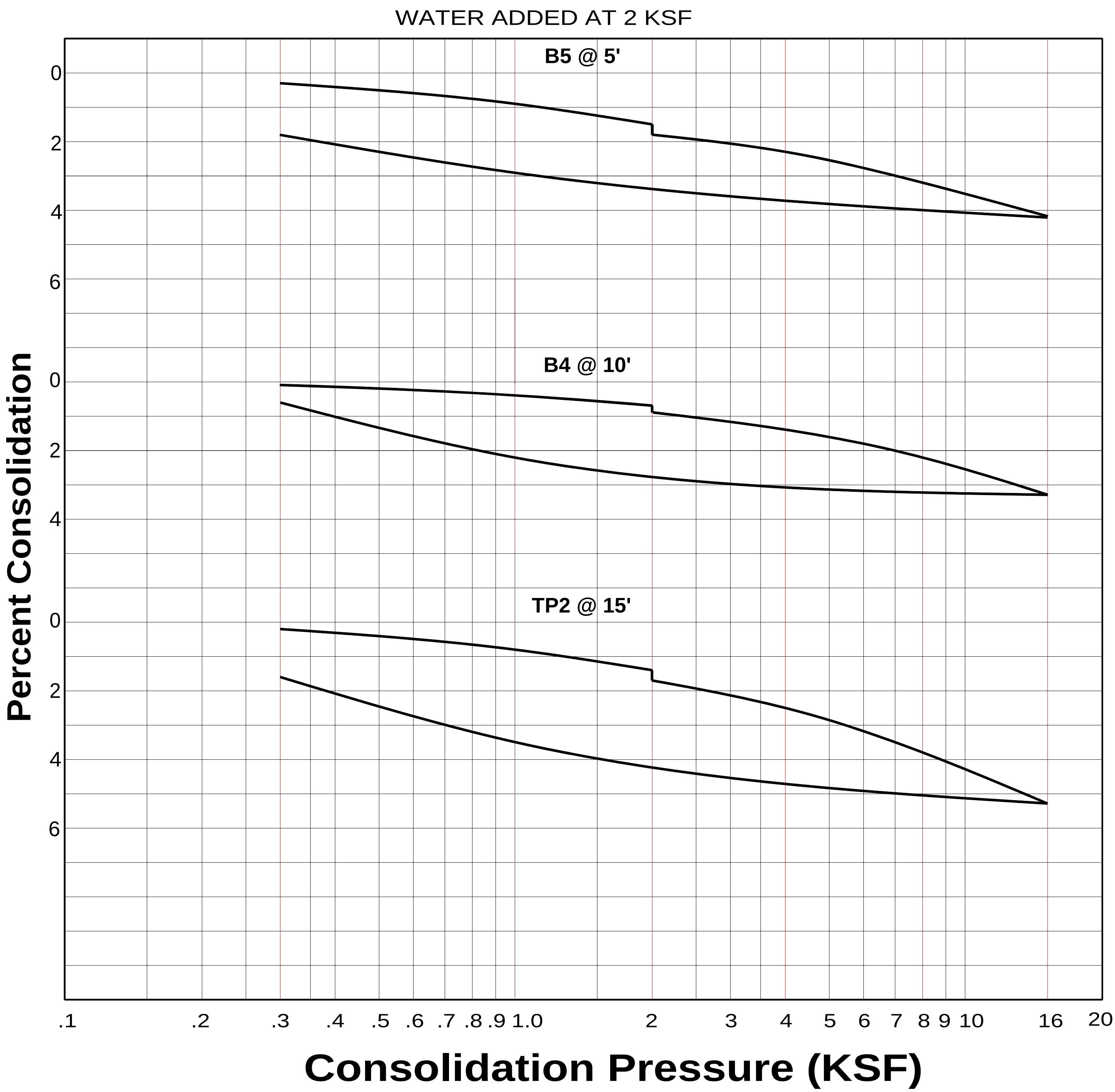
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS

FILE NO. 21536 | PLATE: B-2

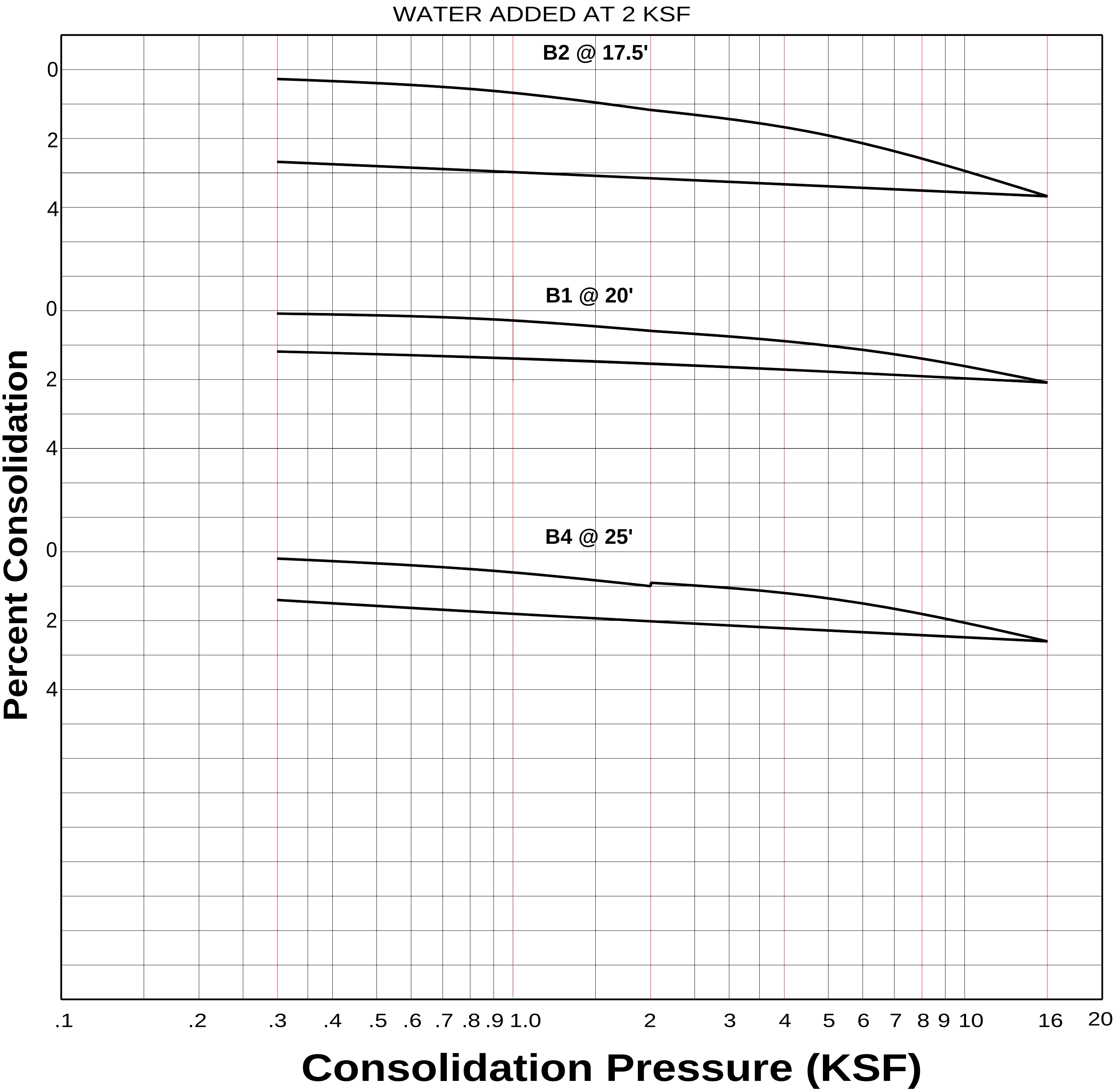


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS

FILE NO. 21536	**PLATE: C-1**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS

FILE NO. 21536 | **PLATE: C-2**

ASTM D-1557

SAMPLE	B1 @ 1-5'	B4 @ 1-5'
SOIL TYPE:	SM	SM/CL
MAXIMUM DENSITY pcf.	132.6	126.8
OPTIMUM MOISTURE %	8.0	10.6

ASTM D 4829

SAMPLE	B1 @ 1-5'	B4 @ 1-5'
SOIL TYPE:	SM	SM/CL
EXPANSION INDEX UBC STANDARD 18-2	43	82
EXPANSION CHARACTER	LOW	MODERATE

SULFATE CONTENT

SAMPLE	B1 @ 1-5'	B4 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10%	> 0.20%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

VALUE SCHOOLS

FILE NO. 21536

PLATE: D

Geotechnologies, Inc.

Project: Value Schools
File No.: 21536
Description: Liquefaction Analysis (10% Exceedance in 50 Years)
Boring No: 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.7
Peak Ground Horizontal Acceleration, PGA (g):	0.64
Calculated Mag.Wtg.Factor:	1.234
GROUNDWATER INFORMATION:	
Current Groundwater Level (ft):	17.0
Historically Highest Groundwater Level* (ft):	17.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y
LIQUEFACTION BOUNDARY:	
Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc} (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	124.9	Unsaturated	Unsaturated	13	5	0.0	0	124.9	124.9	29.5	1.00	0.418	0.619	Non-Liq.	0.00
2	124.9	Unsaturated	Unsaturated	13	5	0.0	0	249.8	249.8	29.5	1.00	0.416	0.619	Non-Liq.	0.00
3	124.9	Unsaturated	Unsaturated	13	5	0.0	0	374.7	374.7	29.5	1.00	0.415	0.619	Non-Liq.	0.00
4	124.9	Unsaturated	Unsaturated	13	5	0.0	0	499.6	499.6	29.5	0.99	0.413	0.615	Non-Liq.	0.00
5	124.9	Unsaturated	Unsaturated	13	5	0.0	0	624.5	624.5	29.3	0.99	0.412	0.607	Non-Liq.	0.00
6	124.9	Unsaturated	Unsaturated	13	5	0.0	0	749.4	749.4	27.7	0.99	0.410	0.507	Non-Liq.	0.00
7	124.9	Unsaturated	Unsaturated	13	5	0.0	0	874.3	874.3	26.0	0.98	0.409	0.431	Non-Liq.	0.00
8	119.9	Unsaturated	Unsaturated	13	5	0.0	0	994.2	994.2	24.7	0.98	0.407	0.383	Non-Liq.	0.00
9	119.9	Unsaturated	Unsaturated	13	5	0.0	0	1114.1	1114.1	25.0	0.97	0.405	0.394	Non-Liq.	0.00
10	119.9	Unsaturated	Unsaturated	6	10	0.0	0	1234.0	1234.0	10.7	0.97	0.403	0.160	Non-Liq.	0.00
11	119.9	Unsaturated	Unsaturated	6	10	0.0	0	1353.9	1353.9	10.2	0.96	0.401	0.154	Non-Liq.	0.00
12	119.9	Unsaturated	Unsaturated	6	10	0.0	0	1473.8	1473.8	9.8	0.96	0.399	0.149	Non-Liq.	0.00
13	126.3	Unsaturated	Unsaturated	6	10	0.0	0	1600.1	1600.1	9.4	0.96	0.397	0.144	Non-Liq.	0.00
14	126.3	Unsaturated	Unsaturated	6	10	0.0	0	1726.4	1726.4	9.0	0.95	0.395	0.140	Non-Liq.	0.00
15	126.7	Unsaturated	Unsaturated	26	15	0.0	0	1853.1	1853.1	47.8	0.95	0.393	2.000	Non-Liq.	0.00
16	126.7	Unsaturated	Unsaturated	26	15	0.0	0	1979.8	1979.8	46.9	0.94	0.391	2.000	Non-Liq.	0.00
17	126.7	Unsaturated	Unsaturated	26	15	0.0	0	2106.5	2106.5	46.2	0.93	0.389	2.000	Non-Liq.	0.00
18	126.7	Saturated	Saturated	26	15	0.0	0	2233.2	2170.8	45.8	0.93	0.398	2.000	5.0	0.00
19	126.7	Saturated	Saturated	26	15	0.0	0	2359.9	2235.1	45.5	0.92	0.406	2.000	4.9	0.00
20	125.9	Saturated	Saturated	78	20	0.0	0	2485.8	2298.6	135.4	0.92	0.413	2.000	4.8	0.00
21	125.9	Saturated	Saturated	78	20	0.0	0	2611.7	2362.1	134.4	0.91	0.420	2.000	4.8	0.00
22	125.9	Saturated	Saturated	78	20	0.0	0	2737.6	2425.6	133.5	0.91	0.426	2.000	4.7	0.00
23	125.9	Saturated	Saturated	78	20	0.0	0	2863.5	2489.1	132.6	0.90	0.432	2.000	4.6	0.00
24	125.9	Saturated	Saturated	78	20	0.0	0	2989.4	2552.6	131.7	0.90	0.436	2.000	4.6	0.00
25	125.9	Saturated	Saturated	80	25	0.0	0	3115.3	2616.1	134.2	0.89	0.441	2.000	4.5	0.00
26	125.9	Saturated	Saturated	80	25	0.0	0	3241.2	2679.6	133.4	0.88	0.445	2.000	4.5	0.00
27	125.9	Saturated	Saturated	80	25	0.0	0	3367.1	2743.1	132.5	0.88	0.448	2.000	4.5	0.00
28	129.6	Saturated	Saturated	80	25	0.0	0	3496.7	2810.3	138.6	0.87	0.452	2.000	4.4	0.00
29	129.6	Saturated	Saturated	80	25	0.0	0	3626.3	2877.5	137.8	0.87	0.454	2.000	4.4	0.00
30	129.6	Saturated	Saturated	50	30	0.0	0	3755.9	2944.7	85.6	0.86	0.456	2.000	4.4	0.00
31	129.6	Saturated	Saturated	50	30	0.0	0	3885.5	3011.9	85.1	0.85	0.458	2.000	4.4	0.00
32	129.6	Saturated	Saturated	50	30	0.0	0	4015.1	3079.1	84.6	0.85	0.460	2.000	4.3	0.00
33	128.2	Saturated	Saturated	50	30	0.0	0	4143.3	3144.9	84.1	0.84	0.461	2.000	4.3	0.00
34	128.2	Saturated	Saturated	50	30	0.0	0	4271.5	3210.7	83.7	0.84	0.463	2.000	4.3	0.00
35	125.8	Saturated	Saturated	50	35	0.0	0	4397.3	3274.1	83.2	0.83	0.463	2.000	4.3	0.00
36	125.8	Saturated	Saturated	50	35	0.0	0	4523.1	3337.5	82.8	0.82	0.464	2.000	4.3	0.00
37	125.8	Saturated	Saturated	50	35	0.0	0	4648.9	3400.9	82.4	0.82	0.465	2.000	4.3	0.00
38	125.8	Saturated	Saturated	50	35	0.0	0	4774.7	3464.3	82.0	0.81	0.465	2.000	4.3	0.00
39	125.8	Saturated	Saturated	50	35	0.0	0	4900.5	3527.7	81.6	0.80	0.465	2.000	4.3	0.00
40	127.4	Saturated	Saturated	50	40	0.0	0	5027.9	3592.7	81.2	0.80	0.465	2.000	4.3	0.00
41	127.4	Saturated	Saturated	50	40	0.0	0	5155.3	3657.7	80.8	0.79	0.465	2.000	4.3	0.00
42	127.4	Saturated	Saturated	50	40	0.0	0	5282.7	3722.7	80.5	0.79	0.464	2.000	4.3	0.00
43	127.4	Saturated	Saturated	50	40	0.0	0	5410.1	3787.7	80.1	0.78	0.463	2.000	4.3	0.00
44	127.4	Saturated	Saturated	50	40	0.0	0	5537.5	3852.7	79.7	0.77	0.463	2.000	4.3	0.00
45	127.4	Saturated	Saturated	50	45	0.0	0	5664.9	3917.7	79.4	0.77	0.462	2.000	4.3	0.00
46	127.4	Saturated	Saturated	50	45	0.0	0	5792.3	3982.7	79.0	0.76	0.461	2.000	4.3	0.00
47	127.4	Saturated	Saturated	50	45	0.0	0	5919.7	4047.7	78.7	0.76	0.460	1.994	4.3	0.00
48	127.4	Saturated	Saturated	50	45	0.0	0	6047.1	4112.7	78.4	0.75	0.458	1.982	4.3	0.00
49	127.4	Saturated	Saturated	50	45	0.0	0	6174.5	4177.7	78.1	0.74	0.457	1.971	4.3	0.00
50	127.4	Saturated	Saturated	50	50	0.0	0	6301.9	4242.7	77.7	0.74	0.456	1.960	4.3	0.00
51	127.4	Saturated	Saturated	50	50	0.0	0	6429.3	4307.7	77.4	0.73	0.454	1.949	4.3	0.00
52	127.4	Saturated	Saturated	50	50	0.0	0	6556.7	4372.7	77.1	0.73	0.453	1.938	4.3	0.00
53	130.3	Saturated	Saturated	50	50	0.0	0	6687.0	4440.6	76.8	0.72	0.451	1.927	4.3	0.00
54	130.3	Saturated	Saturated	50	50	0.0	0	6817.3	4508.5	76.5	0.71	0.449	1.916	4.3	0.00
55	130.3	Saturated	Saturated	50	55	0.0	0	6947.6	4576.4	76.2	0.71	0.447	1.905	4.3	0.00
56	130.3	Saturated	Saturated	50	55	0.0	0	7077.9	4644.3	75.9	0.70	0.446	1.894	4.2	0.00
57	130.3	Saturated	Saturated	50	55	0.0	0	7208.2	4712.2	75.6	0.70	0.444	1.883	4.2	0.00
58	125.6	Saturated	Saturated	50	55	0.0	0	7333.8	4775.4	75.4	0.69	0.442	1.874	4.2	0.00
59	125.6	Saturated	Saturated	50	55	0.0	0	7459.4	4838.6	75.1	0.69	0.440	1.864	4.2	0.00
60	125.6	Saturated	Saturated	50	60	0.0	0	7585.0	4901.8	74.8	0.68	0.438	1.855	4.2	0.00
											Total Liquefaction Settlement, S =			**0.00 inches**	

Geotechnologies, Inc.

Project: Value Schools
File No.: 21536
Description: Liquefaction Analysis (2% Exceedance in 50 Years)
Boring No: 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	0.96
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	17.0
Historically Highest Groundwater Level* (ft):	17.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc} (psf)	Effective Vert. Stress σ_{vc}' (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	124.9	Unsaturated	Unsaturated	13	5	0.0	0	124.9	124.9	29.5	1.00	0.627	0.587	Non-Liq.	0.00
2	124.9	Unsaturated	Unsaturated	13	5	0.0	0	249.8	249.8	29.5	1.00	0.625	0.587	Non-Liq.	0.00
3	124.9	Unsaturated	Unsaturated	13	5	0.0	0	374.7	374.7	29.5	1.00	0.623	0.587	Non-Liq.	0.00
4	124.9	Unsaturated	Unsaturated	13	5	0.0	0	499.6	499.6	29.5	0.99	0.621	0.583	Non-Liq.	0.00
5	124.9	Unsaturated	Unsaturated	13	5	0.0	0	624.5	624.5	29.3	0.99	0.619	0.576	Non-Liq.	0.00
6	124.9	Unsaturated	Unsaturated	13	5	0.0	0	749.4	749.4	27.7	0.99	0.616	0.481	Non-Liq.	0.00
7	124.9	Unsaturated	Unsaturated	13	5	0.0	0	874.3	874.3	26.0	0.98	0.614	0.409	Non-Liq.	0.00
8	119.9	Unsaturated	Unsaturated	13	5	0.0	0	994.2	994.2	24.7	0.98	0.612	0.364	Non-Liq.	0.00
9	119.9	Unsaturated	Unsaturated	13	5	0.0	0	1114.1	1114.1	25.0	0.98	0.609	0.374	Non-Liq.	0.00
10	119.9	Unsaturated	Unsaturated	6	10	0.0	0	1234.0	1234.0	10.7	0.97	0.607	0.152	Non-Liq.	0.00
11	119.9	Unsaturated	Unsaturated	6	10	0.0	0	1353.9	1353.9	10.2	0.97	0.604	0.146	Non-Liq.	0.00
12	119.9	Unsaturated	Unsaturated	6	10	0.0	0	1473.8	1473.8	9.8	0.96	0.601	0.141	Non-Liq.	0.00
13	126.3	Unsaturated	Unsaturated	6	10	0.0	0	1600.1	1600.1	9.4	0.96	0.599	0.137	Non-Liq.	0.00
14	126.3	Unsaturated	Unsaturated	6	10	0.0	0	1726.4	1726.4	9.0	0.95	0.596	0.133	Non-Liq.	0.00
15	126.7	Unsaturated	Unsaturated	26	15	0.0	0	1853.1	1853.1	47.8	0.95	0.593	2.000	Non-Liq.	0.00
16	126.7	Unsaturated	Unsaturated	26	15	0.0	0	1979.8	1979.8	46.9	0.95	0.590	2.000	Non-Liq.	0.00
17	126.7	Unsaturated	Unsaturated	26	15	0.0	0	2106.5	2106.5	46.2	0.94	0.587	2.000	Non-Liq.	0.00
18	126.7	Saturated	Saturated	26	15	0.0	0	2233.2	2170.8	45.8	0.94	0.601	2.000	3.3	0.00
19	126.7	Saturated	Saturated	26	15	0.0	0	2359.9	2235.1	45.5	0.93	0.613	2.000	3.3	0.00
20	125.9	Saturated	Saturated	78	20	0.0	0	2485.8	2298.6	135.4	0.93	0.625	2.000	3.2	0.00
21	125.9	Saturated	Saturated	78	20	0.0	0	2611.7	2362.1	134.4	0.92	0.635	2.000	3.1	0.00
22	125.9	Saturated	Saturated	78	20	0.0	0	2737.6	2425.6	133.5	0.92	0.645	2.000	3.1	0.00
23	125.9	Saturated	Saturated	78	20	0.0	0	2863.5	2489.1	132.6	0.91	0.653	2.000	3.1	0.00
24	125.9	Saturated	Saturated	78	20	0.0	0	2989.4	2552.6	131.7	0.90	0.661	2.000	3.0	0.00
25	125.9	Saturated	Saturated	80	25	0.0	0	3115.3	2616.1	134.2	0.90	0.668	2.000	3.0	0.00
26	125.9	Saturated	Saturated	80	25	0.0	0	3241.2	2679.6	133.4	0.89	0.675	2.000	3.0	0.00
27	125.9	Saturated	Saturated	80	25	0.0	0	3367.1	2743.1	132.5	0.89	0.681	2.000	2.9	0.00
28	129.6	Saturated	Saturated	80	25	0.0	0	3496.7	2810.3	138.6	0.88	0.686	2.000	2.9	0.00
29	129.6	Saturated	Saturated	80	25	0.0	0	3626.3	2877.5	137.8	0.88	0.690	2.000	2.9	0.00
30	129.6	Saturated	Saturated	50	30	0.0	0	3755.9	2944.7	85.6	0.87	0.694	2.000	2.9	0.00
31	129.6	Saturated	Saturated	50	30	0.0	0	3885.5	3011.9	85.1	0.87	0.697	2.000	2.9	0.00
32	129.6	Saturated	Saturated	50	30	0.0	0	4015.1	3079.1	84.6	0.86	0.700	2.000	2.9	0.00
33	128.2	Saturated	Saturated	50	30	0.0	0	4143.3	3144.9	84.1	0.85	0.703	2.000	2.8	0.00
34	128.2	Saturated	Saturated	50	30	0.0	0	4271.5	3210.7	83.7	0.85	0.705	2.000	2.8	0.00
35	125.8	Saturated	Saturated	50	35	0.0	0	4397.3	3274.1	83.2	0.84	0.707	2.000	2.8	0.00
36	125.8	Saturated	Saturated	50	35	0.0	0	4523.1	3337.5	82.8	0.84	0.708	2.000	2.8	0.00
37	125.8	Saturated	Saturated	50	35	0.0	0	4648.9	3400.9	82.4	0.83	0.710	2.000	2.8	0.00
38	125.8	Saturated	Saturated	50	35	0.0	0	4774.7	3464.3	82.0	0.83	0.710	2.000	2.8	0.00
39	125.8	Saturated	Saturated	50	35	0.0	0	4900.5	3527.7	81.6	0.82	0.711	1.987	2.8	0.00
40	127.4	Saturated	Saturated	50	40	0.0	0	5027.9	3592.7	81.2	0.81	0.711	1.975	2.8	0.00
41	127.4	Saturated	Saturated	50	40	0.0	0	5155.3	3657.7	80.8	0.81	0.711	1.962	2.8	0.00
42	127.4	Saturated	Saturated	50	40	0.0	0	5282.7	3722.7	80.5	0.80	0.711	1.950	2.7	0.00
43	127.4	Saturated	Saturated	50	40	0.0	0	5410.1	3787.7	80.1	0.80	0.710	1.938	2.7	0.00
44	127.4	Saturated	Saturated	50	40	0.0	0	5537.5	3852.7	79.7	0.79	0.710	1.926	2.7	0.00
45	127.4	Saturated	Saturated	50	45	0.0	0	5664.9	3917.7	79.4	0.79	0.709	1.915	2.7	0.00
46	127.4	Saturated	Saturated	50	45	0.0	0	5792.3	3982.7	79.0	0.78	0.708	1.903	2.7	0.00
47	127.4	Saturated	Saturated	50	45	0.0	0	5919.7	4047.7	78.7	0.77	0.706	1.892	2.7	0.00
48	127.4	Saturated	Saturated	50	45	0.0	0	6047.1	4112.7	78.4	0.77	0.705	1.881	2.7	0.00
49	127.4	Saturated	Saturated	50	45	0.0	0	6174.5	4177.7	78.1	0.76	0.703	1.870	2.7	0.00
50	127.4	Saturated	Saturated	50	50	0.0	0	6301.9	4242.7	77.7	0.76	0.702	1.860	2.6	0.00
51	127.4	Saturated	Saturated	50	50	0.0	0	6429.3	4307.7	77.4	0.75	0.700	1.849	2.6	0.00
52	127.4	Saturated	Saturated	50	50	0.0	0	6556.7	4372.7	77.1	0.75	0.698	1.839	2.6	0.00
53	130.3	Saturated	Saturated	50	50	0.0	0	6687.0	4440.6	76.8	0.74	0.696	1.828	2.6	0.00
54	130.3	Saturated	Saturated	50	50	0.0	0	6817.3	4508.5	76.5	0.73	0.694	1.818	2.6	0.00
55	130.3	Saturated	Saturated	50	55	0.0	0	6947.6	4576.4	76.2	0.73	0.691	1.807	2.6	0.00
56	130.3	Saturated	Saturated	50	55	0.0	0	7077.9	4644.3	75.9	0.72	0.689	1.797	2.6	0.00
57	130.3	Saturated	Saturated	50	55	0.0	0	7208.2	4712.2	75.6	0.72	0.686	1.787	2.6	0.00
58	125.6	Saturated	Saturated	50	55	0.0	0	7333.8	4775.4	75.4	0.71	0.684	1.778	2.6	0.00
59	125.6	Saturated	Saturated	50	55	0.0	0	7459.4	4838.6	75.1	0.71	0.681	1.769	2.6	0.00
60	125.6	Saturated	Saturated	50	60	0.0	0	7585.0	4901.8	74.8	0.70	0.679	1.760	2.6	0.00

Total Liquefaction Settlement, S =	0.00 inches

Geotechnologies, Inc.

Project: Value Schools
File No.: 21536
Description: Foundation Pile Design

Drilled Friction Pile Capacity Calculation

Input Data:

Unit Weight of Overlying Soil Layer	γ_1	100 pcf
Thickness of Overlying Soil Layer	H_1	15 feet
Unit Weight of Bearing Strata	γ_2	120 pcf
Friction Angle of Bearing Strata	ϕ_2	24 degrees
Friction Angle between Pile and Soil	δ	18 degrees
Cohesion of Bearing Strata	c_2	400 psf
Adhesion	c_A	500 psf
Minimum Embedment into Bearing Strata	H_2	30 feet
Unit Weight of Water	γ_w	62.4 pcf
Depth to Groundwater from Pile Cap	H_w	0 feet

Pile Design:
Drilled <<Driven/Drilled
Circular <<Circular/Square Pile

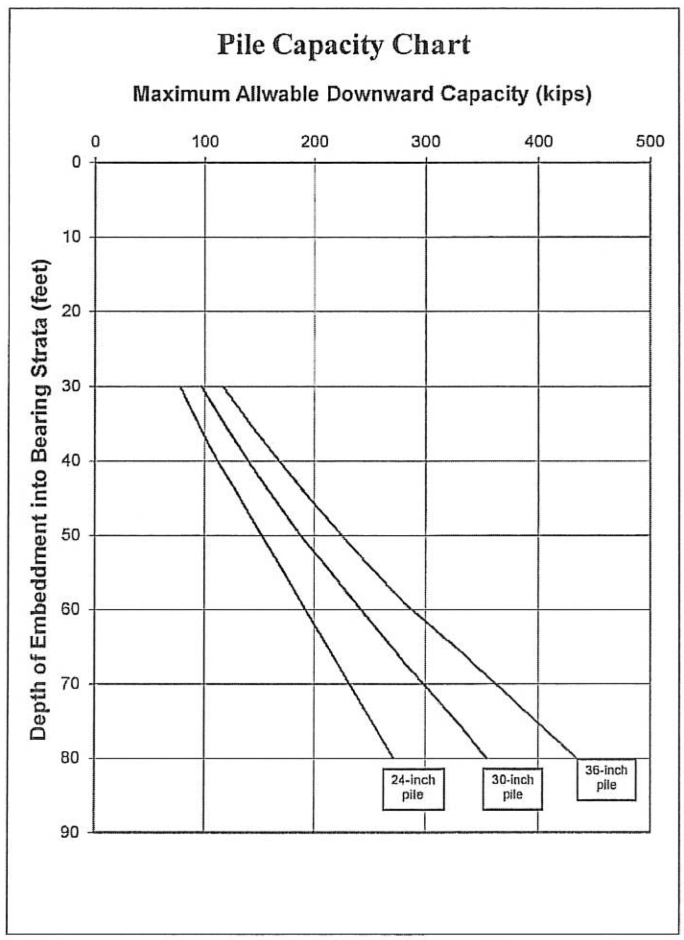
Pile Dimension:
24 inch diameter pile
30 inch diameter pile
36 inch diameter pile

Critical Depth Limit (Dc):
20 B

Lateral Earth Pressure Coefficient: $K_{HC} = 0.70$
Applied Factor of Safety: $FS = 2$
Factored Skin Friction $f_s/FS = [K_{HC} * \sigma'_v * (\tan \delta)]/FS$ <u>or</u> $fs/FS = c_A/FS$

Pile Capacity:

Total Depth of Pile (feet)	Depth of Embeddment into Bearing Strata (feet)	Maximum Allowable Downward Pile Capacity: Capacity of 24 inch diameter pile (kips)	Capacity of 30 inch diameter pile (kips)	Capacity of 36 inch diameter pile (kips)
45	30	77.7	97.2	116.6
46	31	81.0	101.2	121.4
47	32	84.2	105.3	126.4
48	33	87.5	109.4	131.3
49	34	90.9	113.6	136.3
50	35	94.3	117.9	141.4
51	36	97.7	122.2	146.6
52	37	101.2	126.5	151.8
53	38	104.7	130.9	157.1
54	39	108.3	135.3	162.4
55	40	111.9	139.8	167.8
56	41	115.9	144.4	173.3
57	42	119.9	149.0	178.8
58	43	123.9	153.7	184.4
59	44	128.0	158.4	190.0
60	45	132.0	163.1	195.7
61	46	136.0	167.9	201.5
62	47	140.0	172.8	207.3
63	48	144.1	177.7	213.2
64	49	148.1	182.7	219.2
65	50	152.1	187.7	225.2
66	51	156.1	193.2	231.3
67	52	160.2	198.8	237.4
68	53	164.2	204.3	243.6
69	54	168.2	209.8	249.9
70	55	172.2	215.4	256.2
71	56	176.2	220.9	262.6
72	57	180.3	226.5	269.0
73	58	184.3	232.0	275.6
74	59	188.3	237.6	282.1
75	60	192.3	243.1	288.8
76	61	196.4	248.6	296.0
77	62	200.4	254.2	303.3
78	63	204.4	259.7	310.6
79	64	208.4	265.3	317.8
80	65	212.5	270.8	325.1
81	66	216.5	276.4	332.4
82	67	220.5	281.9	339.7
83	68	224.5	287.5	346.9
84	69	228.5	293.0	354.2
85	70	232.6	298.5	361.5
86	71	236.6	304.1	368.7
87	72	240.6	309.6	376.0
88	73	244.6	315.2	383.3
89	74	248.7	320.7	390.5
90	75	252.7	326.3	397.8
91	76	256.7	331.8	405.1
92	77	260.7	337.3	412.3
93	78	264.8	342.9	419.6
94	79	268.8	348.4	426.9
95	80	272.8	354.0	434.2



<u>Note:</u> 1. Minimum pile embeddment depth of 30 feet
2. Uplift capacity may be designed using 50% of the downward capacity
3. Pile should be spaced a minimum of 3 diameters on center
4. See text of report for pile details and installation recommendations

Geotechnologies, Inc.

Project: Value SA=chools
File No.: 21536
Description: Foundation Pile Design

Drilled Friction Pile Capacity Calculation

Input Data:

Unit Weight of Overlying Soil Layer	γ_1	120	pcf
Thickness of Overlying Soil Layer	H_1	10	feet
Unit Weight of Bearing Strata	γ_2	120	pcf
Friction Angle of Bearing Strata	ϕ_2	24	degrees
Friction Angle between Pile and Soil	δ	18	degrees
Cohesion of Bearing Strata	c_2	400	psf
Adhesion	c_A	0	psf
Minimum Embedment into Bearing Strata	H_2	10	feet
Unit Weight of Water	γ_w	62.4	pcf
Depth to Groundwater from Pile Cap	H_w	20	feet

Pile Design:

Drilled <<Driven/Drilled
Circular <<Circular/Square Pile

Pile Dimension:

12 inch diameter pile
18 inch diameter pile
24 inch diameter pile

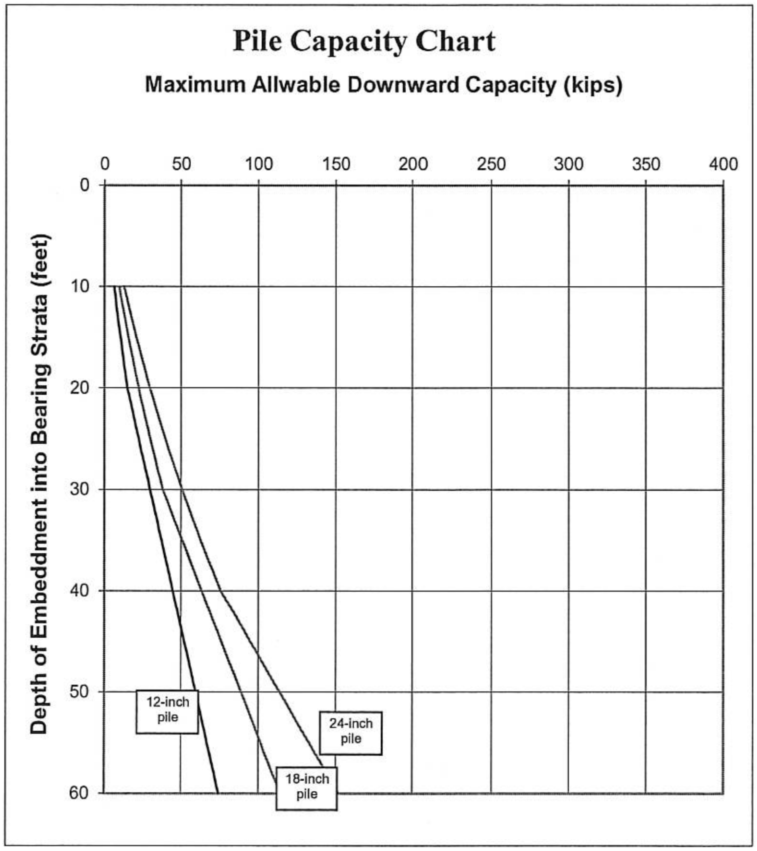
Critical Depth Limit (Dc):

20 B

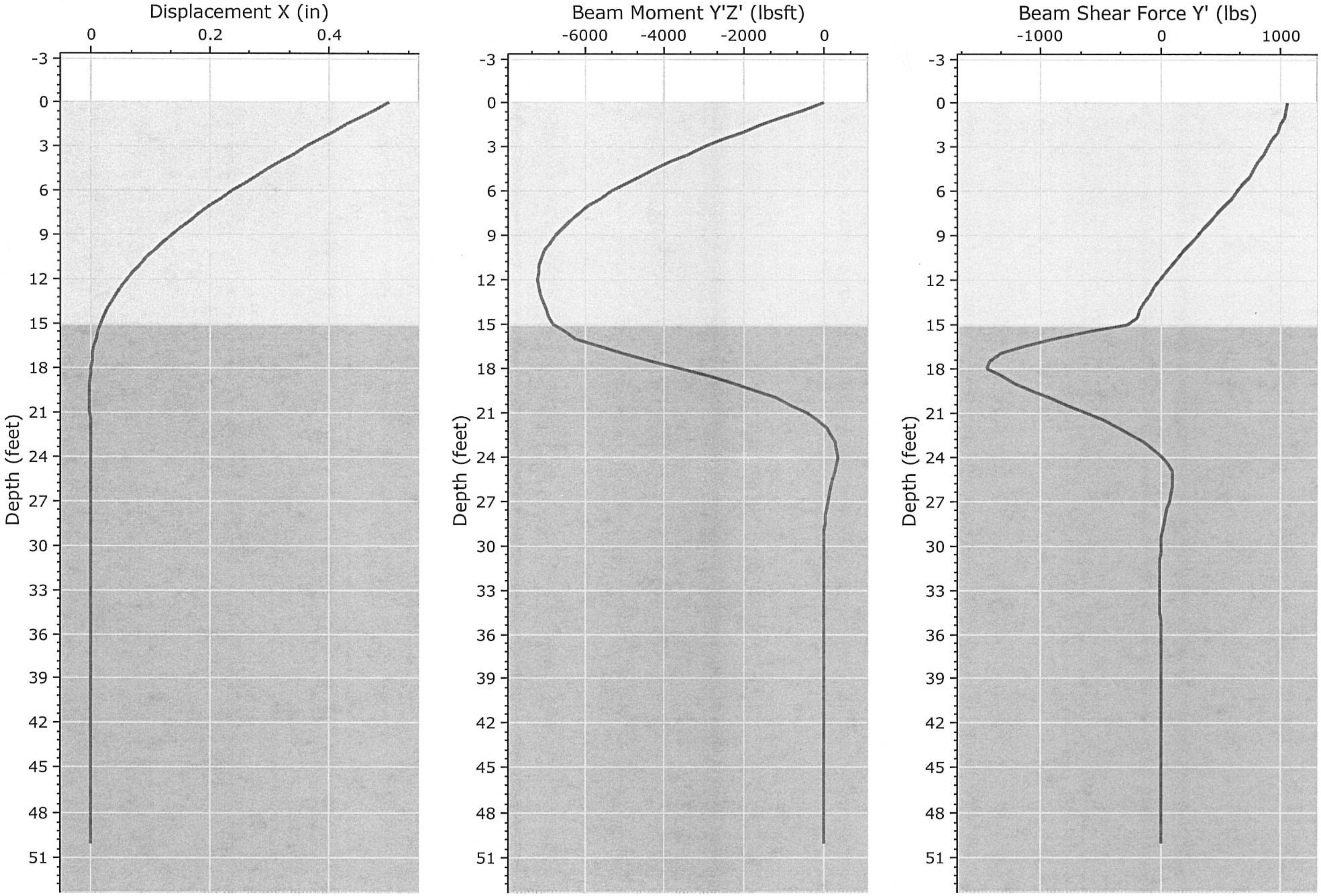
Lateral Earth Pressure Coefficient: $K_{HC} = 0.70$
Applied Factor of Safety: $FS = 2$
Factored Skin Friction $f_s/FS = [K_{HC}*\sigma'_v*(\tan\delta)]/FS$ <u>or</u> $f_s/FS = c_A/FS$

Pile Capacity:

Total Depth of Pile (feet)	Depth of Embedment into Bearing Strata (feet)	Maximum Allowable Downward Pile Capacity: Capacity of 12 inch diameter pile (kips)	Capacity of 18 inch diameter pile (kips)	Capacity of 24 inch diameter pile (kips)
20	10	6.4	9.6	12.9
21	11	7.2	10.8	14.4
22	12	8.0	11.9	15.9
23	13	8.8	13.1	17.5
24	14	9.6	14.4	19.2
25	15	10.4	15.6	20.8
26	16	11.3	16.9	22.6
27	17	12.2	18.2	24.3
28	18	13.1	19.6	26.1
29	19	14.0	21.0	28.0
30	20	14.9	22.4	29.8
31	21	16.4	23.8	31.8
32	22	17.9	25.3	33.7
33	23	19.4	26.8	35.7
34	24	20.9	28.3	37.8
35	25	22.4	29.9	39.9
36	26	23.9	31.5	42.0
37	27	25.4	33.1	44.2
38	28	26.9	34.8	46.4
39	29	28.3	36.5	48.6
40	30	29.8	38.2	50.9
41	31	31.3	40.7	53.3
42	32	32.8	43.3	55.6
43	33	34.3	45.8	58.1
44	34	35.8	48.4	60.5
45	35	37.3	50.9	63.0
46	36	38.8	53.5	65.6
47	37	40.3	56.0	68.1
48	38	41.8	58.6	70.8
49	39	43.3	61.1	73.4
50	40	44.8	63.7	76.1
51	41	46.3	66.2	79.9
52	42	47.7	68.8	83.8
53	43	49.2	71.3	87.6
54	44	50.7	73.9	91.4
55	45	52.2	76.4	95.2
56	46	53.7	78.9	99.0
57	47	55.2	81.5	102.8
58	48	56.7	84.0	106.6
59	49	58.2	86.6	110.4
60	50	59.7	89.1	114.2
61	51	61.2	91.7	118.0
62	52	62.7	94.2	121.8
63	53	64.2	96.8	125.6
64	54	65.6	99.3	129.4
65	55	67.1	101.9	133.2
66	56	68.6	104.4	137.1
67	57	70.1	107.0	140.9
68	58	71.6	109.5	144.7
69	59	73.1	112.1	148.5
70	60	74.6	114.6	152.3



<u>Note:</u>
1. Minimum pile embeddment depth of 10 feet
2. **Uplift capacity may be designed using 50% of the downward capacity**
3. Pile should be spaced a minimum of 3 diameters on center
4. See text of report for pile details and installation recommendations



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RSPILE 2.019

Project	Value Schools - Proposed Charter School (233-241 N. Westmoreland Ave, Los Angeles)
Analysis Description	Lateral Pile Capacity - 1.0 Foot Diameter Pile (Free Head)
Drawn By	S. Prince
Company	Geotechnologies, Inc. (FN 21536)
Date	1/13/2020, 3:46:46 PM
File Name	21536 - Value Schools, 1.0 Foot Dia, Free Head 2.rspile2

	Depth (feet)	Displacement X (in)	Beam Moment Y'Z' (lbsft)	Beam Shear Force Y' (lbs)
1	0.000	0.500	-1.511	1056.158
2	0.500	0.477	-525.058	1047.095
3	1.000	0.454	-1049.719	1032.119
4	1.500	0.431	-1552.723	1003.782
5	2.000	0.408	-2055.126	972.264
6	2.500	0.385	-2522.937	934.596
7	3.000	0.363	-2990.324	896.340
8	3.500	0.341	-3418.916	857.003
9	4.000	0.319	-3847.518	818.715
10	4.500	0.298	-4237.229	779.219
11	5.000	0.278	-4627.334	737.548
12	5.500	0.257	-4972.793	689.927
13	6.000	0.238	-5318.092	639.635
14	6.500	0.219	-5611.089	585.325
15	7.000	0.201	-5903.871	530.541
16	7.500	0.183	-6141.149	474.315
17	8.000	0.166	-6378.324	417.817
18	8.500	0.150	-6558.895	361.106
19	9.000	0.135	-6739.367	304.525
20	9.500	0.121	-6863.739	248.904
21	10.000	0.107	-6988.020	193.793
22	10.500	0.094	-7058.219	140.743
23	11.000	0.082	-7128.335	88.557
24	11.500	0.071	-7147.802	39.446
25	12.000	0.061	-7167.193	-8.479
26	12.500	0.051	-7140.652	-52.417
27	13.000	0.042	-7114.046	-94.883
28	13.500	0.035	-7047.362	-132.571
29	14.000	0.028	-6980.624	-168.544
30	14.500	0.021	-6880.627	-199.090
31	15.000	0.016	-6800.253	-281.469
32	15.500	0.012	-6522.475	-593.898
33	16.000	0.008	-6217.591	-914.626
34	16.500	0.005	-5639.590	-1140.132
35	17.000	0.002	-5060.657	-1334.014
36	17.500	0.001	-4341.042	-1421.498

	Depth (feet)	Displacement X (in)	Beam Moment Y'Z' (lbsft)	Beam Shear Force Y' (lbs)
37	18.000	-0.001	-3619.158	-1446.215
38	18.500	-0.001	-2939.364	-1337.320
39	19.000	-0.002	-2268.299	-1218.799
40	19.500	-0.002	-1730.184	-1071.420
41	20.000	-0.002	-1194.306	-920.387
42	20.500	-0.002	-810.470	-767.337
43	21.000	-0.002	-427.868	-616.762
44	21.500	-0.002	-189.443	-478.983
45	22.000	-0.001	48.304	-347.286
46	22.500	-0.001	164.823	-236.528
47	23.000	-0.001	281.061	-133.651
48	23.500	-0.000	306.579	-55.089
49	24.000	-0.000	331.916	15.503
50	24.500	-0.000	299.907	59.602
51	25.000	-0.000	267.811	92.496
52	25.500	-0.000	216.592	97.848
53	26.000	0.000	166.714	97.243
54	26.500	0.000	123.167	85.184
55	27.000	0.000	80.429	71.442
56	27.500	0.000	52.311	55.942
57	28.000	0.000	24.558	40.949
58	28.500	0.000	10.493	28.566
59	29.000	0.000	-3.492	17.412
60	29.500	0.000	-8.112	9.837
61	30.000	0.000	-12.788	3.419
62	30.500	0.000	-12.502	-0.087
63	31.000	0.000	-12.309	-2.805
64	31.500	0.000	-10.295	-3.729
65	32.000	0.000	-8.362	-4.241
66	32.500	-0.000	-6.331	-3.922
67	33.000	-0.000	-4.352	-3.459
68	33.500	-0.000	-2.945	-2.779
69	34.000	-0.000	-1.562	-2.099
70	34.500	-0.000	-0.819	-1.499
71	35.000	-0.000	-0.084	-0.952
72	35.500	-0.000	0.187	-0.569

RSPile Analysis Information

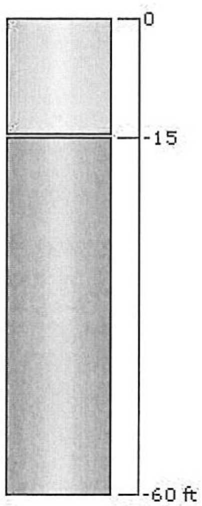
Value Schools - Proposed Charter School (233-241 N. Westmoreland Ave, Los Angeles)

Project Summary

Document Name	21536 - Value Schools, 1.0 Foot Dia, Free Head 2
Project Title	Value Schools - Proposed Charter School (233-241 N. Westmoreland Ave, Los Angeles)
Analysis	Lateral Pile Capacity - 1.0 Foot Diameter Pile (Free Head)
Author	S. Prince
Company	Geotechnologies, Inc. (FN 21536)
Date Created	1/13/2020, 3:46:46 PM
Last saved with RSPile version	2.019

Soil Layers

Layer Name	Color	Layer Type	Thickness [ft]	Depth [ft]
Soil Property 1		Sand	15	0
Soil Property 2		Sand	45	15



Soil Properties

Soil Property 1

Property	Value
Name	Soil Property 1
Color	
Soil Type	Sand
Unit Weight (lbs/ft3)	123.74
Sat. Unit Weight (lbs/ft3)	135
Friction Angle (degrees)	5
Kpy (lbs/ft3)	43200
Kpy Saturated (lbs/ft3)	34560

Soil Property 2

Property	Value
Name	Soil Property 2
Color	
Soil Type	Sand
Unit Weight (lbs/ft3)	126.6
Sat. Unit Weight (lbs/ft3)	135
Friction Angle (degrees)	24
Kpy (lbs/ft3)	388800
Kpy Saturated (lbs/ft3)	216000

Pile Properties

Pile Property 1

Property	Value
Name	Pile Property 1
Color	
Pile Type	Elastic
Pile Cross Section	Circle
Diameter (ft)	1
Young's Modulus (psf)	5.1912e+008

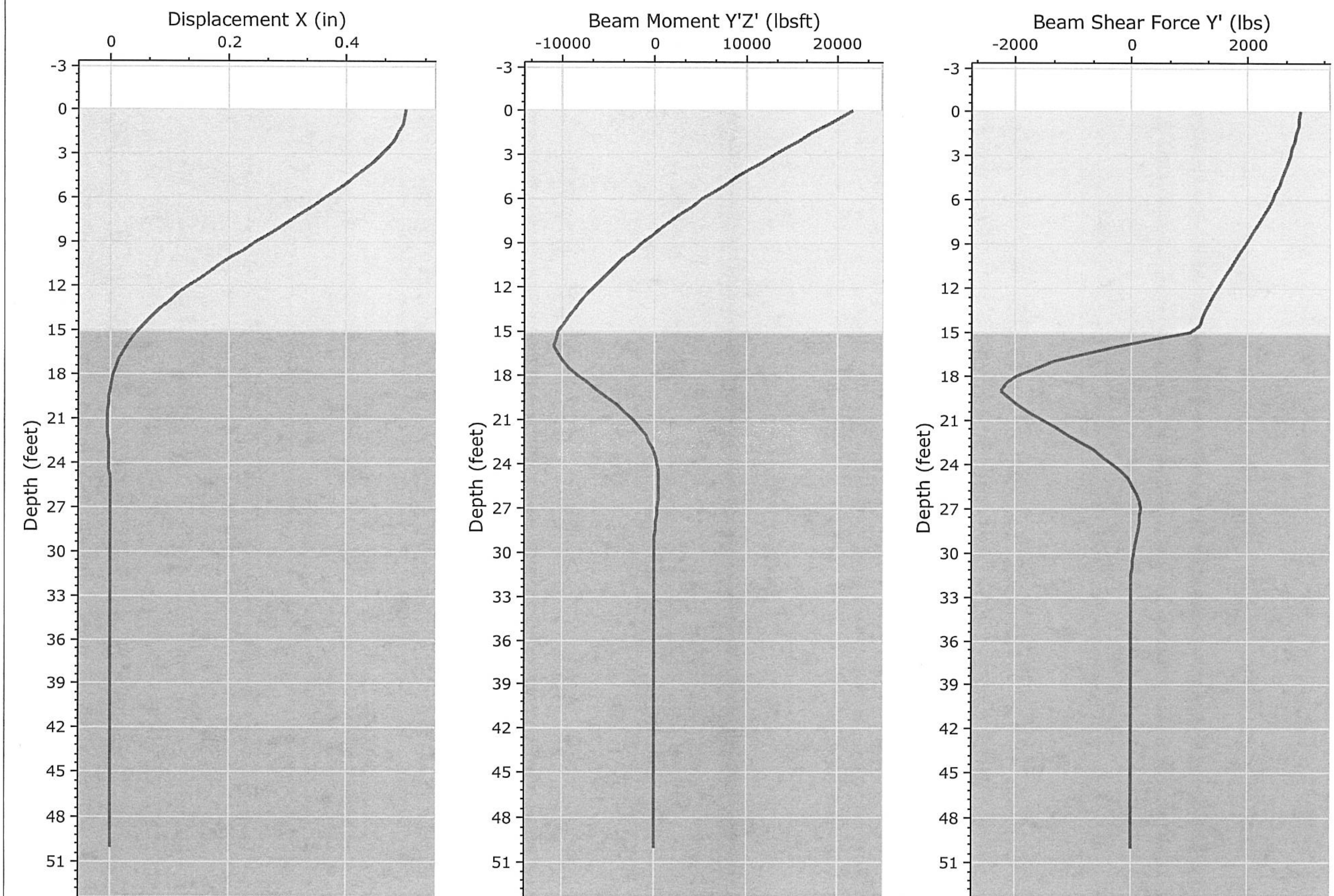
Pile Settings

Pile 1

General	
Property	Pile Property 1
Location	0, 0.127
Elevation:	0 (ft)
Length:	50 (ft)

Orientation	
Elevation (ft)	0
Length (ft)	50
Ground Slope Angle (°)	0
Alpha Angle (°)	0
Beta Angle (°)	90
Rotation Angle (°)	0

Loading	
Loading Type	Static
Load Factor Profile	None
Type	**Value**
Force Z, (lbs)	100000
Moment Y, (lbsft)	0
Deflection X, (ft)	0.0417



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RSPILE 2.019

Project	Value Schools - Proposed Charter School (233-241 N. Westmoreland Ave, Los Angeles)
Analysis Description	Lateral Pile Capacity - 1.0 Foot Diameter Pile (Fixed Head)
Drawn By	S. Prince
Company	Geotechnologies, Inc. (FN 21536)
Date	1/14/2020, 8:38:15 AM
File Name	21536 - Value Schools, 1.0 Foot Dia, Fixed Head.rspile2

	Depth (feet)	Displacement X (in)	Beam Moment Y'Z' (lbsft)	Beam Shear Force Y' (lbs)
1	0.000	0.500	21577.981	2910.551
2	0.500	0.499	20127.295	2901.371
3	1.000	0.495	18675.431	2886.156
4	1.500	0.489	17245.855	2856.796
5	2.000	0.482	15816.848	2823.889
6	2.500	0.472	14424.403	2783.671
7	3.000	0.460	13032.317	2742.458
8	3.500	0.447	11682.946	2698.243
9	4.000	0.433	10333.500	2654.282
10	4.500	0.417	9029.962	2606.430
11	5.000	0.400	7725.969	2555.359
12	5.500	0.383	6477.714	2494.956
13	6.000	0.365	5229.651	2430.636
14	6.500	0.346	4049.416	2359.301
15	7.000	0.326	2869.441	2286.691
16	7.500	0.306	1764.025	2210.183
17	8.000	0.286	658.737	2132.626
18	8.500	0.266	-367.814	2052.710
19	9.000	0.246	-1394.233	1972.270
20	9.500	0.226	-2339.828	1891.064
21	10.000	0.207	-3285.289	1809.870
22	10.500	0.188	-4149.980	1729.523
23	11.000	0.169	-5014.538	1649.728
24	11.500	0.151	-5800.524	1572.381
25	12.000	0.134	-6586.381	1496.115
26	12.500	0.117	-7297.974	1423.852
27	13.000	0.101	-8009.444	1353.178
28	13.500	0.086	-8652.973	1287.970
29	14.000	0.072	-9296.393	1224.819
30	14.500	0.060	-9880.052	1168.448
31	15.000	0.048	-10504.051	1014.281
32	15.500	0.038	-10735.233	382.815
33	16.000	0.029	-10915.708	-287.408
34	16.500	0.021	-10491.558	-826.435
35	17.000	0.015	-10066.987	-1320.645
36	17.500	0.009	-9216.324	-1678.622

	Depth (feet)	Displacement X (in)	Beam Moment Y'Z' (lbsft)	Beam Shear Force Y' (lbs)
37	18.000	0.005	-8364.832	-1991.051
38	18.500	0.002	-7273.996	-2157.309
39	19.000	-0.000	-6177.778	-2240.210
40	19.500	-0.002	-5104.040	-2112.349
41	20.000	-0.003	-4043.364	-1957.197
42	20.500	-0.003	-3164.849	-1748.025
43	21.000	-0.004	-2289.989	-1530.862
44	21.500	-0.004	-1637.379	-1303.522
45	22.000	-0.003	-986.734	-1077.762
46	22.500	-0.003	-555.155	-865.388
47	23.000	-0.002	-124.707	-660.407
48	23.500	-0.002	114.236	-482.378
49	24.000	-0.002	352.628	-314.933
50	24.500	-0.001	440.360	-181.059
51	25.000	-0.001	527.968	-58.978
52	25.500	-0.000	511.024	28.043
53	26.000	-0.000	493.741	104.411
54	26.500	-0.000	419.660	141.639
55	27.000	-0.000	346.248	163.230
56	27.500	0.000	266.799	153.713
57	28.000	0.000	189.159	138.413
58	28.500	0.000	131.521	113.708
59	29.000	0.000	74.821	88.511
60	29.500	0.000	42.393	65.166
61	30.000	0.000	10.285	43.543
62	30.500	0.000	-3.049	27.616
63	31.000	0.000	-16.399	13.761
64	31.500	0.000	-18.640	5.398
65	32.000	0.000	-21.025	-1.378
66	32.500	0.000	-18.518	-4.386
67	33.000	0.000	-16.160	-6.469
68	33.500	-0.000	-12.707	-6.577
69	34.000	-0.000	-9.361	-6.292
70	34.500	-0.000	-6.649	-5.306
71	35.000	-0.000	-3.995	-4.247
72	35.500	-0.000	-2.406	-3.175

RSPile Analysis Information

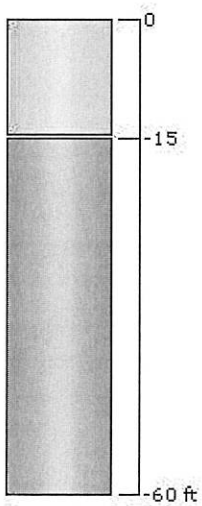
Value Schools - Proposed Charter School (233-241 N. Westmoreland Ave, Los Angeles)

Project Summary

Document Name	21536 - Value Schools, 1.0 Foot Dia, Fixed Head
Project Title	Value Schools - Proposed Charter School (233-241 N. Westmoreland Ave, Los Angeles)
Analysis	Lateral Pile Capacity - 1.0 Foot Diameter Pile (Fixed Head)
Author	S. Prince
Company	Geotechnologies, Inc. (FN 21536)
Date Created	1/14/2020, 8:38:15 AM
Last saved with RSPile version	2.019

Soil Layers

Layer Name	Color	Layer Type	Thickness [ft]	Depth [ft]
Soil Property 1		Sand	15	0
Soil Property 2		Sand	45	15



Soil Properties

Soil Property 1

Property	Value
Name	Soil Property 1
Color	
Soil Type	Sand
Unit Weight (lbs/ft3)	123.74
Sat. Unit Weight (lbs/ft3)	135
Friction Angle (degrees)	5
Kpy (lbs/ft3)	43200
Kpy Saturated (lbs/ft3)	34560

Soil Property 2

Property	Value
Name	Soil Property 2
Color	
Soil Type	Sand
Unit Weight (lbs/ft3)	126.6
Sat. Unit Weight (lbs/ft3)	135
Friction Angle (degrees)	24
Kpy (lbs/ft3)	388800
Kpy Saturated (lbs/ft3)	216000

Pile Properties

Pile Property 1

Property	Value
Name	Pile Property 1
Color	
Pile Type	Elastic
Pile Cross Section	Circle
Diameter (ft)	1
Young's Modulus (psf)	5.1912e+008

Pile Settings

Pile 1

General	
Property	Pile Property 1
Location	0, 0
Elevation:	0 (ft)
Length:	50 (ft)

Orientation	
Elevation (ft)	0
Length (ft)	50
Ground Slope Angle (°)	0
Alpha Angle (°)	0
Beta Angle (°)	90
Rotation Angle (°)	0

Loading	
Loading Type	Static
Load Factor Profile	None
Type	**Value**
Force Z, (lbs)	100000
Slope Y, (deg)	0
Deflection X, (ft)	0.0417

DATE DRILLED: 3/29/19	LOGGED BY: DHC	BORING NO. B-1	
DRIVE WEIGHT: 140 lbs.	DROP: 30 inches	DEPTH TO GROUNDWATER (ft.): 20	
DRILLING METHOD: 8" HSA	DRILLER: Baja Exploration	SURFACE ELEVATION (ft.): 273 ±(MSL)	

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0								4" Asphalt with no base
		Bulk				EI			**FILL:** Lean CLAY, brown, moist
268	5	Driven	7			#200, ATT		ML	**NATIVE:** Sandy SILT; medium stiff, brown, slightly moist
263	10	Driven	18	13.9	112.9	#200, ATT		CL	Lean CLAY; very stiff; brown; slightly moist; few gravel
		Driven	21	7.4	106.3			SM	Silty SAND; medium dense; orangish brown; slightly moist
258	15	Driven	30	15.2	110.1			SM	-- same
253	20	Driven	47	17.5	111.4	#200, ATT		SC	Clayey SAND; dense; dark brown; wet
248	25	Driven	50 for 5"	7.6	116.2			SM	Silty SAND; very dense; dark brown; wet; with micaceous flakes
									Total Depth = 26.5 feet Backfilled on 3/29/2019 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with concrete.
243	30								
238	35								

TWINING

LOG OF BORING

Value Schools
241 N. Westmoreland Avenue
Los Angeles, California

PROJECT NO.	REPORT DATE	
190207.1	April 2019	FIGURE A - 2

DATE DRILLED	3/29/19	LOGGED BY	DHC	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	Baja Exploration	SURFACE ELEVATION (ft.)	273 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										4" Asphalt over 2" base
									CL	**FILL:** Lean CLAY; dark brown; moist
268	5		X	10	13.4	111.7			CL	-- same; loose; pieces of asphalt
				5			#200, ATT		CL	**NATIVE:** Sandy Lean CLAY; medium stiff; light brown; some oxidation staining
263	10		X	12	15.8	111.2	C		CL	-- same; stiff
				13			#200, ATT		CL	-- same; stiff
258	15		X	29	16.6	113.2	DS		CL	--same; very stiff
253	20			11					CL	same; stiff; yellowish/orangish brown
										Total Depth = 21.5 feet Backfilled on 3/29/2019 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with concrete.
248	25									
243	30									
238	35									

TWINING

LOG OF BORING

Value Schools
241 N. Westmoreland Avenue
Los Angeles, California

PROJECT NO.	REPORT DATE	
190207.1	April 2019	FIGURE A - 3

DATE DRILLED 3/29/19 LOGGED BY DHC **BORING NO. B-3**

DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E

DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 273 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										5" Asphalt over 4" base
									CL	**FILL:** Lean CLAY; light brown; slightly moist
268	5			8			#200, ATT		ML	**NATIVE:** SILT; medium stiff; light brown; slightly moist
				12	19.3	104.2			CL	Lean CLAY; stiff; dark brown; slightly moist; few gravel
263	10			12					CL	-- same; stiff
				70	14.5	116.3			CL	-- same; hard; reddish brown
258	15			21			#200, ATT		ML	SILT; very stiff; light brown; slightly moist
253	20			37					SM	Silty SAND; dense; dark reddish brown; slightly moist; with micaceous flakes
										Total Depth = 21.5 feet Backfilled on 3/29/2019 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with concrete.
248	25									
243	30									
238	35									

TWINING

LOG OF BORING

Value Schools
241 N. Westmoreland Avenue
Los Angeles, California

PROJECT NO.	REPORT DATE	
190207.1	April 2019	FIGURE A - 4

DATE DRILLED	3/29/19	LOGGED BY	DHC	BORING NO.	B-4
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	Baja Exploration	SURFACE ELEVATION (ft.)	273 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										4" Asphalt over 4" base
									CL	**FILL:** Lean CLAY; brown; slightly moist
268	5		X	14	15.7	109.1			CL	-- same; stiff
				5			#200, ATT		CL	**NATIVE:** Lean CLAY; medium stiff; dark brown; slightly moist; some oxidation staining
263	10		X	25	14.3	111.4	DS		CL	-- same; very stiff
				10					SM	Silty SAND; loose; orangish brown; slightly moist
258	15		X	52	9.3	117.4			SM	-- same; very dense; some sharply fractured gravel
										Total Depth = 16.5 feet Backfilled on 3/29/2019 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with concrete.
253	20									
248	25									
243	30									
238	35									

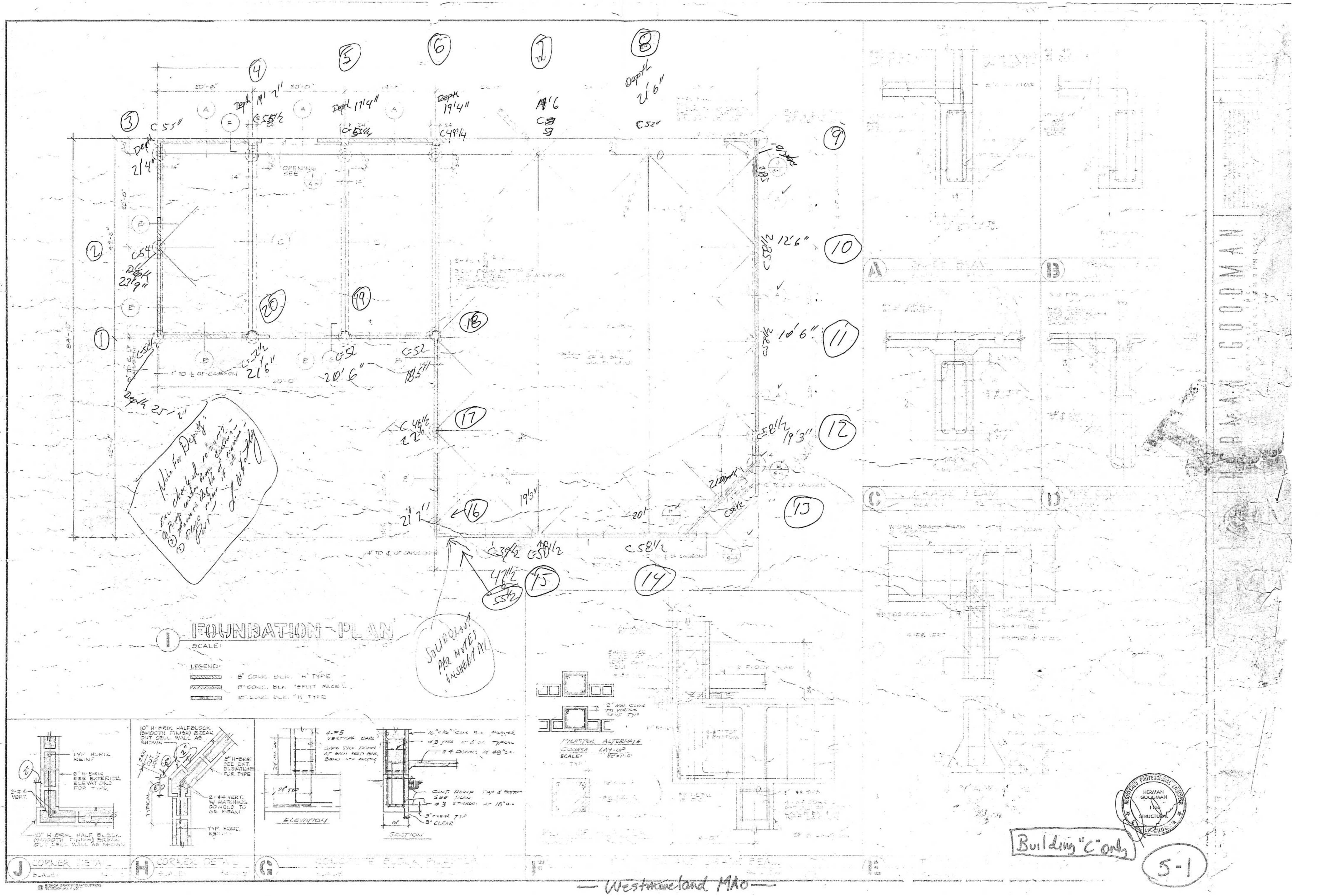
BORING LOG 190207.1 - VALUE SCHOOLS - WESTMORELAND AVE.GPJ TWINING LABS.GDT 4/11/19

TWINING

LOG OF BORING

Value Schools
241 N. Westmoreland Avenue
Los Angeles, California

PROJECT NO.	REPORT DATE	
190207.1	April 2019	FIGURE A - 5



FOUNDATION PLAN
SCALE:
LEGEND:
8" CONC. BLK. 'H' TYPE
8" CONC. BLK. SPLIT FACE
10" CONC. BLK. 'H' TYPE
4" TO ℄ OF CAISSON
OPENING SEE
Depth 21'4"
Depth 23'9"
Depth 25'-2"
Depth 19'4"
Depth 17'4"
Depth 21'6"
C 55"
C 54"
C 52"
C 49¼
C 58½
C=52
C 46½
21'6"
20'6"
18.5"
22'0"
21'2"
19'3"
12'6"
10'6"
19'3"
47½
55½
SOLID GROUT PER NOTES ON SHEET A1
10" H-BRIK HALFBLOCK (SMOOTH FINISH) BREAK OUT CELL WALL AS SHOWN
TYP. HORIZ. REINF.
2-#4 VERT.
4-#5 VERTICAL BARS
ELEVATION
SECTION
PILASTER ALTERNATE COURSE LAY-UP
CORNER DETAIL
Building "C" only
5-1
Westmoreland MAO