Appendix 9.14
Geotechnical Data

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DIN/CAL 4, Inc.
3411 Richmond Avenue, Fifth Floor
Houston, Texas 77046

Attention: Curtis Burnett

<u>Subject</u>: Geotechnical Engineering Investigation
Proposed Residential Complex
12850 Crenshaw Boulevard, Gardena, California

Dear Mr. Burnett:

This letter transmits the Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring 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.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL COMPLEX
12850 CRENSHAW BOULEVARD
GARDENA, CALIFORNIA

INTRODUCTION

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

This investigation included five 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 client, and the office of AO Architects. The proposed development consists of a new residential complex. The proposed structure will be eight stories in height, comprising of 5-story of wood frame or steel residential structure constructed over 3 concrete podium parking levels. The structure will be constructed at or near the existing site grades. Column loads are estimated to be between 600 and 800 kips. Wall loads are estimated to be between 5 and 10 kips per lineal foot. These loads reflect dead and live loads. Grading will consist of removal and recompaction of existing unsuitable soils.

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 site is located at 12850 Crenshaw Boulevard, in the City of Gardena, California. The site is bounded by a gas station to the north, by alleyway and the Dominguez Channel to the east, by one-story manufacturing development to the south, and by Crenshaw Boulevard to the west. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

At the time of exploration, the site was occupied by a 1-story near-grade commercial structure and associated parking lots. The existing structure will be demolished prior to construction of the proposed residential complex. The site is relatively level, with no pronounced highs or lows.

The neighboring developments consist of multi-story commercial and manufacturing structures. Vegetation on the site consists of shrubbery contained in planter areas, and patches of vegetation within the parking area. Drainage across the site appears to be by sheetflow to the city streets.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on December 2, 2019, by excavating five borings. The borings were excavated to depths of 30 and 60 feet below grade. The borings were excavated with the aid of a limited-access drilling machine, using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-5.

The location of exploratory excavations was determined from hardscaped features shown in the enclosed Plot Plan. The location of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in all exploratory excavations, to depths ranging between 2½ and 3 feet below the existing site grade. The fill consists of silty to clayey sand, sandy clay, and sandy silt, which are brown to dark brown in color, moist, medium dense and firm to stiff, fine to medium grained, with variable amounts of gravel and construction debris fragments.

The fill is in turn underlain by native alluvial soils, consisting of sandy to clayey silts, sandy to silty clays, and silty to clayey sands and sands. The native alluvial soils range from light brown to dark brown and olive brown to grayish dark brown in color, slightly moist to wet, medium dense to very dense, stiff to very stiff, and fine to medium grained, with variable amounts of gravel. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was encountered during exploration at depths ranging from 26½ feet to 28 feet below the ground surface. Groundwater was not encountered in Boring B3, conducted to a depth of 30 feet below the ground surface. The historically highest groundwater level is based on review of the Inglewood 7½ Minute Quadrangle Seismic Hazard Evaluation Report, Plate 1.2, Historically Highest Ground Water Contours (CDMG, 1998, Revised 2006). Review of this report indicates that the historically highest groundwater level at the site may be considered to be 25 feet below grade. A copy of this plate is included in the Appendix as Historically Highest Groundwater Levels Map.

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 drilling of the borings due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils 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 (Yerkes, 1965). The site is shown relative to local geology and topography on the enclosed Local Geologic Map.

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. It is bounded to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basing 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 based 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 alluvial sediments deposited by river and stream action, that are likely 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 Inglewood Quadrangle by the State of California (CDMG, 1999), does 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 copy of this map is provided in the Appendix of this report.

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 semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered during exploration in Boring B2 at a depth of 28 feet below the ground surface. According to the Seismic Hazard Zone Report of the Inglewood 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically highest groundwater level for the site is on the order of 25 feet below grade. The historic highest groundwater level was conservatively utilized for the enclosed liquefaction analysis.

The peak ground acceleration (PGA_M) and modal magnitude were obtained from the USGS websites, using the ASCE 7 Hazard Tool (https://asce7hazardtool.online/) and the Probabilistic Seismic Hazard Deaggregation program (USGS, 2014). A modal magnitude (MW) of 6.79 was obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014). A peak ground acceleration of 0.955g, which corresponds to the site's PGA_M, was obtained using the ASCE 7 Hazard tool. These parameters are used in the enclosed liquefaction analyses.

The enclosed "Liquefaction Evaluation" calculation sheet is based on Boring 2. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Sample of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory boring are presented on the enclosed E and F Plates. Based on CGS Special Publication 117A (CDMG, 2008), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Furthermore, cohesive soils with PI between 7 and 12 and moisture content greater than 85 percent of the liquid limit are susceptible to liquefaction.

The procedure presented in the SP 117A guidelines was followed in analyzing the liquefaction potential of the subject site. The SP 117A guidelines were developed based on a paper titled, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", by Bray and Sancio (2006). According to the SP 117A, soils having a Plastic Index greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered non-liquefiable, and the analysis of these soil layers was turned off in the liquefaction susceptibility column.

Based on CGS Special Publication 117A (CDMG, 2008), a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk where high-quality, site-specific penetration resistance and geotechnical laboratory data is collected. Utilizing the adjusted blow count data, and the results of laboratory testing, the enclosed liquefaction analysis indicated that the underlying soils would not be capable of liquefaction during the

Maximum Considered Earthquake ground motion, as set forth by ASCE 7-16 Standards and the most recent California Building Code.

Dynamic Dry Settlement

Seismic dry settlements were calculated utilizing Tokimatsu and Seed's procedure for soils encountered in Boring B2 (Tokimatsu and Seed, 1987). The calculations were performed for the soils below the proposed 5 foot removal and recompaction zone, to a depth of 30 feet (based on groundwater encountered at 28 feet below ground surface). Utilizing USGS U.S. Seismic Design Maps tool, a ground acceleration of 0.955g was utilized in the calculations. The acceleration is consistent with the $MCE_G PGA$ as determined by ASCE 7-16. The modal magnitude of 6.79, determined from the USGS Probabilistic Seismic Hazard Deaggregation program (2 percent in 50 years ground motion), was also utilized in the calculation (USGS, 2014).

Based on these parameters, the total seismically-induced dry sand settlement was calculated to be 0.22 inches for soils in Boring B2, below the proposed 5 foot removal and recompaction zone. Differential dynamic dry settlement would not be expected to exceed two-thirds of the total dynamic settlement, or 0.15 inches. The calculated settlements are expected to be within the tolerance of structures designed based on modern building codes.

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 County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990), indicates the site does not lie within the mapped tsunami inundation boundaries. The site is located approximately 5½ miles from the Pacific Ocean, and is considered far and/or high enough from the ocean or lakes such that it would not be prone to hazards of a tsunami or seiche.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Review of the County of Los Angeles Safety Element, (Leighton, 1990) the site does not lie within mapped inundation boundary due to a seiche or a breach an upgradient reservoir.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference 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 structure is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Fill materials were encountered during exploration to a maximum depth of 3 feet below the existing site grade. The existing fill materials are considered to be unsuitable for support of the proposed foundations, floor slabs, or additional fill, but may be reused for the preparation of a uniform compacted fill pad. Groundwater was encountered at depths ranging from 26½ to 28 feet below existing site grade. Historically highest groundwater is estimated at 25 feet below ground surface.

All existing fill materials shall be properly removed and recompacted for support of the proposed structure. For the construction of a uniform compacted fill pad, all existing fill materials and upper native soils shall be removed and recompacted to a minimum depth of 5 feet below the existing site grade. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations, or for a distance equal to the depth of fill below the foundation, whichever is greater. The proposed structure may be supported by conventional foundations bearing in the uniform compacted fill pad.

Based on correspondences with the project structural engineer, it is anticipated that some of the shear wall footings and elevator pits will extend to or below a depth of 5 feet. The deeper footings may bear in the compacted fill and/or the underlying dense native soils anticipated at or below a depth of 5 feet.

Foundations for small outlying structures, such as property line walls, planters, trach enclosures, and canopies, which are not to be tied-in to the proposed buildings, may be supported on conventional foundations bearing in native soils, and/or properly placed compacted fill.

The following statement is made in regard to Los Angeles County Code Sections 110 and 111: It is the opinion of the undersigned based on the findings of this investigation that provided the recommendations presented in this report are followed, the proposed development will be safe for its intended use against hazard from landsliding, settlement and slippage. The proposed development will have no adverse effect on the stability of the site or adjoining properties.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design, as outlined in this report, should be reviewed by this office as is standard practice. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

Shearwave Velocity Measurements

Geophysical measurements were performed at the site by GeoPentech to access the site characterization for ground motion analyses. Multi-Channel Analysis of Surface Waves (MASW) geophysical surveys were performed to collect the shear-wave velocity of subsurface material. The measurements were made from the existing ground surface. The resulting average shearwave velocity (V_{S30}) of 1,208 ft/s (368 m/s) was obtained, corresponding to Site Class C ("Very Dense Soil and Soft Rock") Profile.

Depth Range (feet)	Average Shearwave Velocity (ft/sec)
0-100	1,208

2019 CBC Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class C, which corresponds to a "Very Dense Soil or Soft Rock" Profile, according to Table 20.3-1 of ASCE 7-16. This information and the site coordinates were input into the ASCE 7 Hazard Tool (https://asce7hazardtool.online/) in order to calculate the ground motion parameters for the site. Ground motion parameters for the 2019 CBC are presented below.

2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	C
Mapped Spectral Acceleration at Short Periods (S_S)	1.841g
Site Coefficient (F_a)	1.2
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.209g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.473g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.649g
Site Coefficient (F_v)	1.4
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	0.909g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.606g

FILL SOILS

The maximum depth of fill encountered on the site was 3 feet. The existing fill soils are not suitable for support of newly proposed foundations, floor slabs or additional fill but may be reused as compacted fill. All existing fill materials shall be properly removed and recompacted for foundation and slab support.

EXPANSIVE SOILS

The onsite geologic materials are in the very low to low expansion range. The Expansion Index was found to be 10 and 28 for representative remolded bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slabs-On-Grade" sections of this report.

SOIL CORROSIVITY

Representative samples of the onsite soils were transported to the office of HDR Engineering, Inc. for corrosivity testing. The results indicate that the electrical resistivities of the soils are in the moderately corrosive category with as-received moisture and at saturation. The soils pH value was 7.7, which is considered mildly alkaline. The soluble salt content was low. Chloride and sulfate were found at low concentrations. Nitrate concentration was high enough to be aggressive to copper. Ammonium was not detected.

In summary, the site soils are classified as moderately corrosive to ferrous metals and aggressive to copper. Sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and there are not restrictions on the cement type utilized for concrete foundations in contact with the site soils. Detailed results, discussion of results and recommended mitigating measures are provided in HDR Engineering, Inc.'s report dated December 24, 2019, which is provided in the Appendix.

GRADING GUIDELINES

Site Preparation

- Prior to excavation and/or mass grading, state law requires the contractor to contact Underground Service Alert, and to perform field potholing, if necessary, to confirm underground utilities and/or structures for construction safety. Abandoned utilities and/or structures located within the footprint of the proposed grading area should be removed 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.

Recommended Overexcavation

All existing fill materials shall be properly removed and recompacted for support of the proposed structure. For the construction of a uniform compacted fill pad, all existing fill materials and upper native soils shall be removed and recompacted to a minimum depth of 5 feet below the existing site grade. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations, or for a distance equal to the depth of fill below the foundation, whichever is greater. It is very important that the position of the proposed structure is accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Compaction

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 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 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. Materials larger than 6 inches in maximum dimension shall not be used in the fill. 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 50. 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 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 at the bottom of the excavation were locally above the optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill may require 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 1-inch gravel 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 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 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 2 and 10 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 suck 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 be 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.

FOUNDATION DESIGN

Conventional Foundation Design

The proposed structure may be supported on conventional foundations bearing in the uniform compacted fill pad and/or the underlying dense native soils anticipated at or below a depth of 5 feet.

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 100 pounds per square foot. The bearing capacity increase for each additional foot of depth is 250 pounds per square foot. The maximum recommended bearing capacity is 6,000 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.

Foundations may be designed utilizing a modulus of subgrade reaction of 250 pounds per cubic inch. 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 larger foundations.

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

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

<u>Miscellaneous Foundations</u>

Conventional foundations for structures such as privacy walls, trash enclosures or canopies, which will not be tied-in to the proposed structure, may bear in native soils, and/or a properly compacted fill pad. Continuous footings may be designed for a bearing capacity of 1,500 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. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Foundation Reinforcement

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 0.30 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 200 pounds per cubic foot with a maximum earth pressure of 2,500 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 not expected to exceed 1 inch and occur below the heaviest loaded columns. Differential settlement within 30 feet is not expected to exceed ½ inch.

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.

RETAINING WALL DESIGN

Cantilever Retaining Walls

Miscellaneous site retaining walls up to 6 feet may be required as part of the proposed development. Retaining walls supporting a level backslope may be designed utilizing 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.

In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to street, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by any adjacent buildings.

Dynamic (Seismic) Earth Pressure

Based on the 2019 California Building Code, retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. Miscellaneous retaining walls anticipated for the proposed project are not expected to exceed 6 feet in height. Therefore, the dynamic earth pressure may be omitted.

Waterproofing

Moisture affecting 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 Drainage

All retaining walls shall be provided with a subdrain in order to minimize the potential for future hydrostatic pressure buildup behind the proposed retaining walls. Subdrains may consist of four-inch diameter perforated pipes, placed with perforations facing down. The pipe shall be encased in at least 1-foot of gravel around the pipe. The gravel may consist of ¾-inch to 1 inch crushed rocks.

A compacted fill blanket or other seal shall be provided at the surface. Retaining walls may be backfilled with gravel adjacent to the wall to within 2 feet of the ground surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent of the maximum density as determined by the latest revision of ASTM D 1557.

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 placement of a standard perforated pipe and gravel drainage system. Under these circumstances, every other head joints may be left out, or 2-inch diameter weepholes may be placed at the 8 feet on center along the base of the wall. The wall shall be backfilled with a minimum of 1 foot of gravel above the base of the retaining wall. The gravel may consist of three-quarter inch to one inch crushed rocks.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. If a drainage system is not provided, the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure. In any event, it is recommended that retaining walls be waterproofed.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the ASTM Designation D 1557-02 method of compaction. Flooding should not be permitted. 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.

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Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was encountered during exploration at depths ranging from 26½ to 28 feet below the existing grade. The mapped historically highest groundwater level for the site is on the order of 25 feet below grade. The proposed structure will be constructed at or near the current site grade. Therefore, the only water which could affect the proposed retaining walls would be irrigation water and precipitation.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

TEMPORARY EXCAVATIONS

Excavations on the order of 5 to 6 feet in vertical height are anticipated for the recommended recompaction and foundation excavations. The excavations are expected to expose fill and stiff 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 or slot-cut.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 (h:v) slope gradient in their entirety, up to a maximum depth of 10 feet. 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 Adjacent to Buildings or Property Lines

Where foundation excavation will leave an adjacent foundation or property line unsupported the proposed foundations may be slot cut a maximum vertical height of 6 feet. The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The “A-B-C” slot-cutting procedure should be utilized. The initial excavation consists of excavating the “A” slots. Alternate "A" slot of 8 feet may be worked. The remaining earth buttresses ("B" and "C" slots) should each be 8 feet in width for a combined intervening length of 16 feet. The backfill shall be properly placed or the foundation should be poured in the "A" slots before the "B" slots are excavated. After completing the grading and/or foundation in the "B" slots, finally the "C" slots may be excavated.

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 Slabs-on-Grade

Concrete floor slabs should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Outdoor concrete flatwork should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

Slabs-on-grade and 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 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, 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.

All concrete slabs-on-grade should be supported on vapor retarder. 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.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

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 15 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.

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 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
Heavy Truck	6	9

Concrete paving may also be utilized for the project. For concrete paving, the following sections are recommended:

Service	Concrete Pavement Thickness Inches	Base Course Inches
Passenger Car and Medium Truck Traffic	6	4
Heavy Truck	7½	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 conform to Sections 200-2.2 or 200-2.4 of the “Standard Specifications for Public Works Construction”, (Green Book), latest edition.

For standard crack control, a maximum expansion joint spacing of 15 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. Concrete pavement should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

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

Introduction

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.

Percolation Testing

Percolation testing was conducted in Boring B4, following the procedure for boring percolation test provided in the Guidelines for Design, Investigation and Reporting Low Impact Development Stormwater Infiltration (GS200.2), dated June 30, 2017, presented in the Administrative Manual for the County of Los Angeles, Department of Public Works, Geotechnical and Material Engineering Division.

Boring B4 was drilled to a depth of 30 feet below the existing grade. At the completion of drilling, the borehole was backfilled to a depth of 20 feet and a 2-inch diameter casing was placed within the center of the borehole for the purpose of conducting percolation testing. The casing consisted of a solid PVC pipe from the ground surface to a depth of 10 feet, and slotted PVC pipe between depths of 10 and 20 feet. A sand pack consisting of #3 Monterey Sand was poured into the annular space around the slotted portion of the casing. A 1-foot thick, hydrated bentonite seal was placed over the sand and drill cuttings were placed to the ground surface.

Prior to testing, the borehole was filled with water for the purpose of pre-soaking for 3 hours. After presoaking, the borehole was refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 8 times or until a stabilized rate of drop was obtained, whichever occurred first.

The table below summarizes the results of the infiltration rate derived from the testing. This rate includes correction factors (RF_t, RF_v, and RF_s), as required by the County of Los Angeles procedure. Field readings and calculations for the percolation testing are included in the Appendix.

Boring No.	Depth of Boring Below Existing Ground Surface (ft.)	Percolation Testing Conducted Between Depths (ft.)	Infiltration Rate (in./hr.)
B4	30	10 to 20	0.03

At the completion of the percolation testing, the PVC casing was removed from the percolation testing well, and the resulting hole was backfilled with on-site soils to the ground surface.

The Proposed System

The location and design for potential stormwater disposal have not been specifically addressed on this site. Until the plan achieves more definition, and this office can address the impacts, stormwater infiltration is not recommended. Stormwater infiltration shall only occur on undisturbed native soils, and shall not be allowed within the fill materials.

Any proposed infiltration system shall be located outside the proposed structures. The edge of any proposed infiltration system shall maintain a minimum horizontal setback distance of 15 feet away from any foundation system, and a minimum of 5 feet from property lines.

Stormwater infiltration is not allowed within 10 feet (vertically) from the groundwater level. Groundwater was encountered during exploration at depths ranging from 26½ to 28 feet below the existing grade. As explained in the "Groundwater" Section of this report, the historically highest groundwater levels published by the State of California indicate a historical highest groundwater level of 25 feet below ground surface. Based on these considerations, it is the recommendation of this firm that the bottom of any proposed infiltration system does not extend below a depth of 15 feet below the existing site grade.

The proposed infiltration systems should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.

The proposed stormwater infiltration systems will not be located in a hillside area. The onsite soils are in the very low to low expansion range, and are not susceptible to significant hydroconsolidation.

Due to the dense consistency of the underlying natural alluvial soils, liquefaction potential for the site was remote. It is the opinion of this firm that the any proposed infiltration of stormwater will not materially impact the liquefaction potential of the site.

It is recommended that the design team, including the structural engineer, waterproofing consultant, plumbing engineer, environmental engineer and landscape architect be consulted in regard to the design and construction of filtration systems.

The design and construction of stormwater infiltration systems is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:

- All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.
- All connections associated with stormwater infiltration systems should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.
- Excavations proposed for the installation of stormwater systems should comply with the "Temporary Excavations" sections of this geotechnical engineering investigation, as well as CalOSHA Regulations where applicable.

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 affect 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. 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.

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. Results are presented on Plate D of this report.

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. Results are presented on Plate D of this 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. The most recent revisions 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. The grain size distributions are presented in the E-Plate of this report.

Atterberg Limits

ASTM D 4318 is used to determine the liquid limits, plastic limits, and plasticity index of the soil. These test methods are used to characterize the fine grained fractions of the soil. Results from Atterberg Limits tests are presented in the F-Plate of this report.

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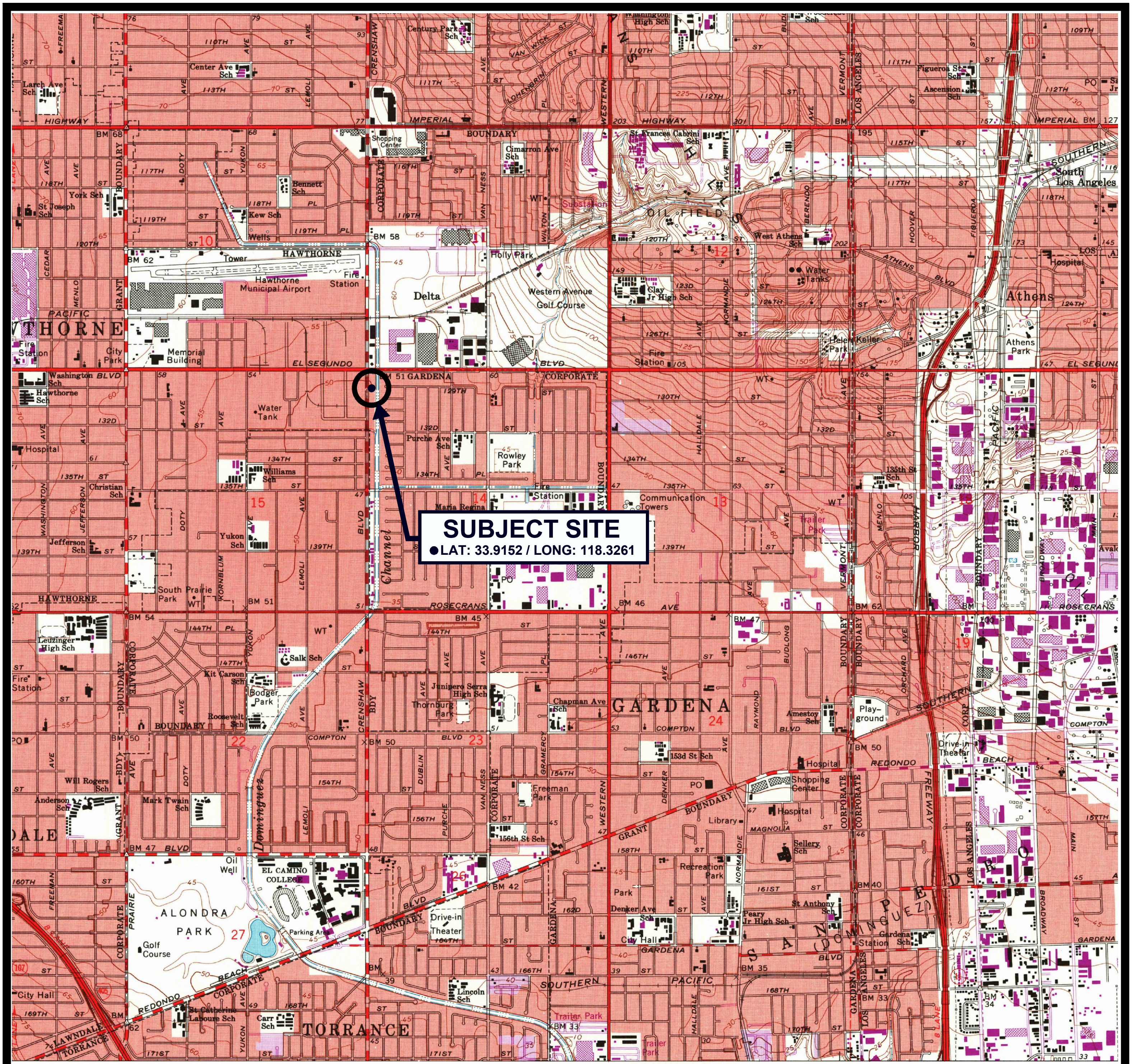


SCALE 1:24 000

1 MILE · 1000 0 1000 2000 3000 4000 5000 6000 7000 FEET · 1 KILOMETER

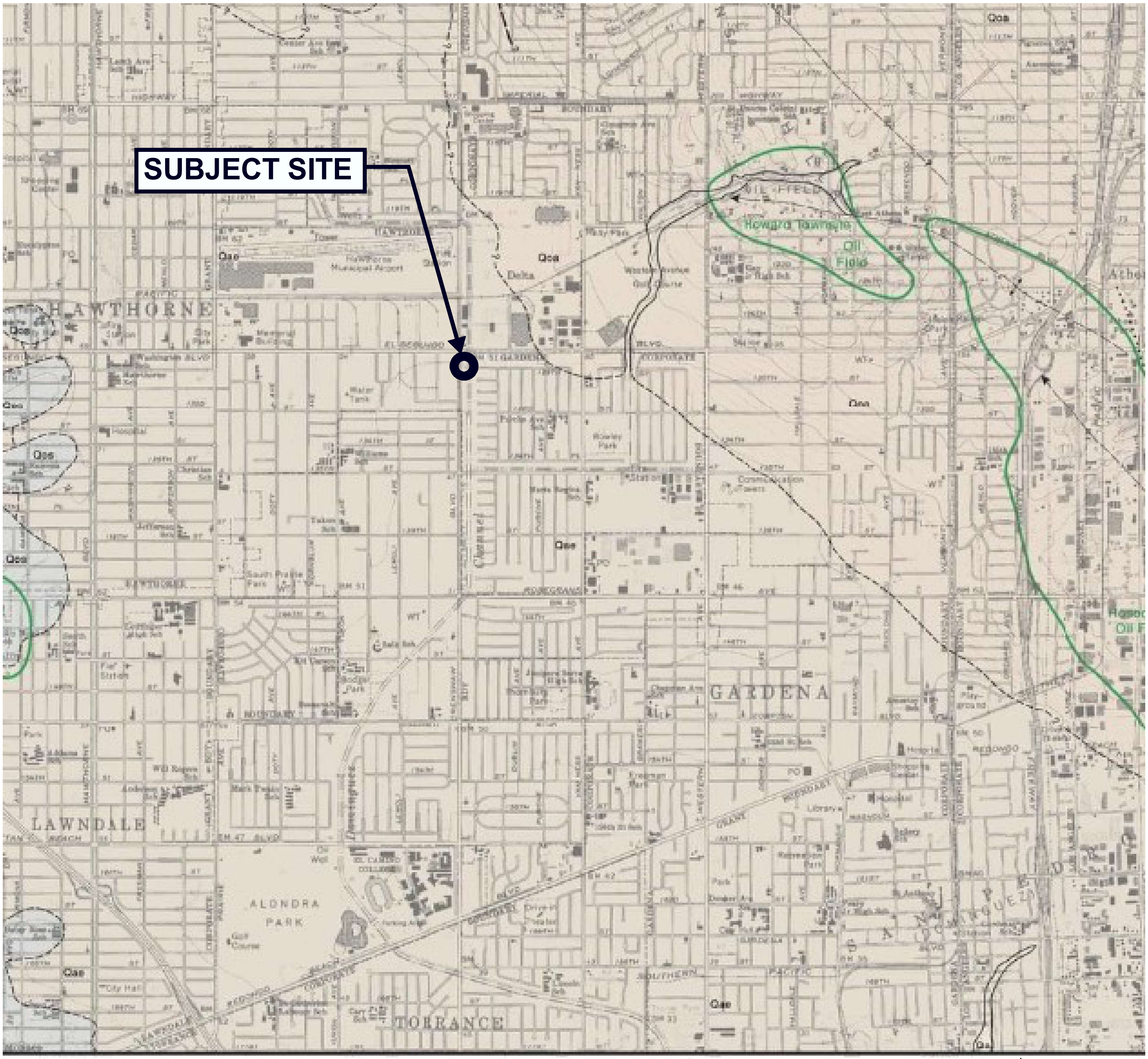
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, INGLEWOOD, CA QUADRANGLE

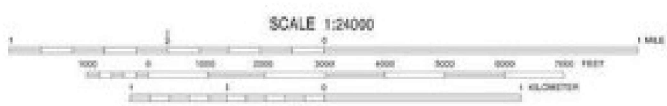
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

DIN/CAL 4, INC.

FILE NO. 21911





LEGEND

Qos: Older Surficial Sediments - old sand dune deposits

Qae: Older Surficial Sediments - alluvial gravel, sand, and clay, slightly elevated and dissected

Qoa: Older Surficial Sediments - older alluvium of gray to light brown pebble-gravel, sand, and silty-clay, elevated and dissected; in Baldwin Hills designated as Baldwin Hills sandy gravel by Weber et al., 1982, where it is much dissected and eroded

REFERNCE: DIBBLEE, T.W., (2007), MAP #DF-322, GEOLOGIC MAP OF THE VENICE AND INGLEWOOD QUADRANGLES

N

LOCAL GEOLOGIC MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

DIN/CAL 4, INC.

FILE NO. 21911

SCALE 1:24,000

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

N

LIQUEFACTION AREA

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

SEISMIC HAZARD ZONE MAP

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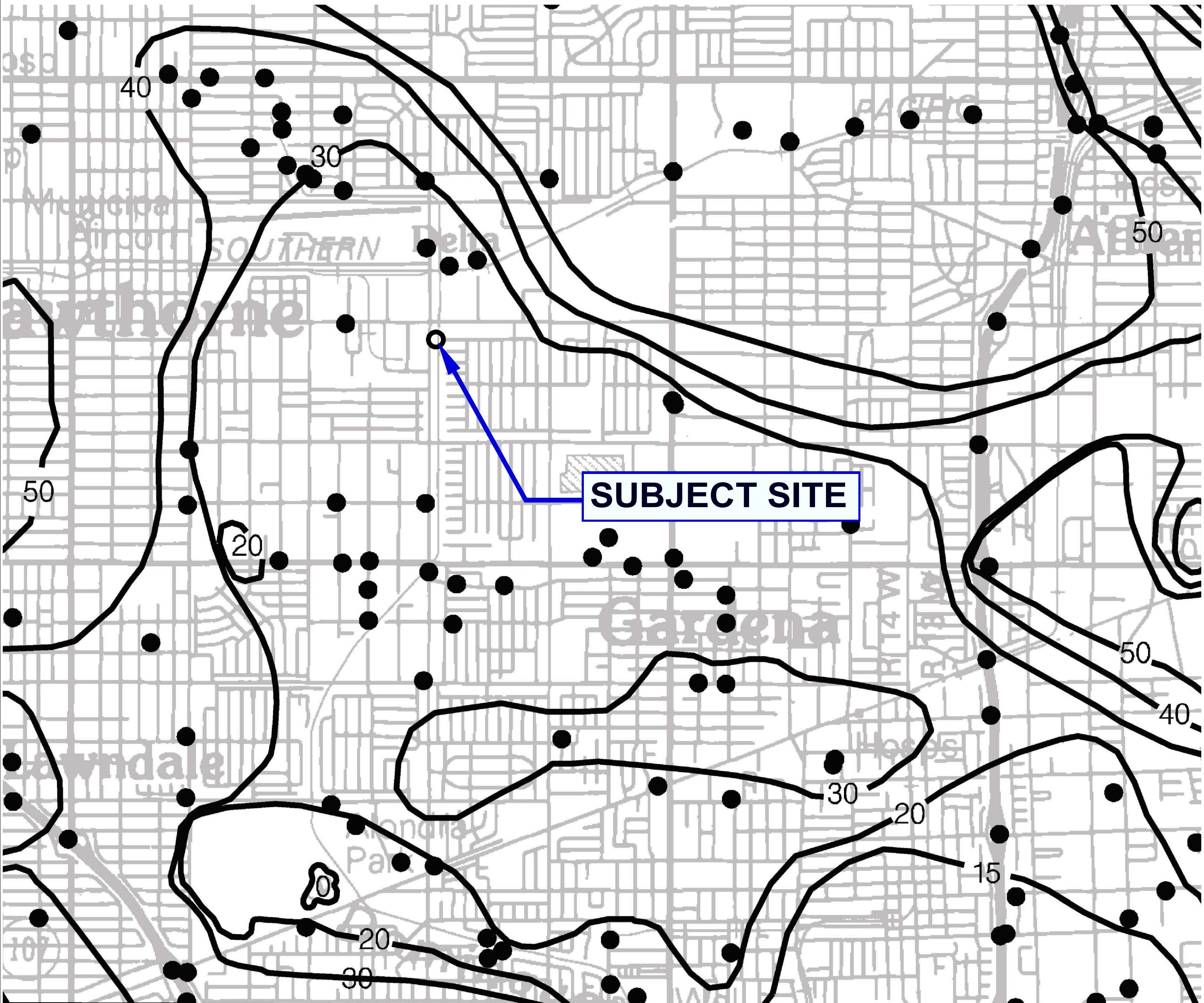
FILE NO. 21911



ONE MILE

SCALE

N

30 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 027
INGLEWOOD 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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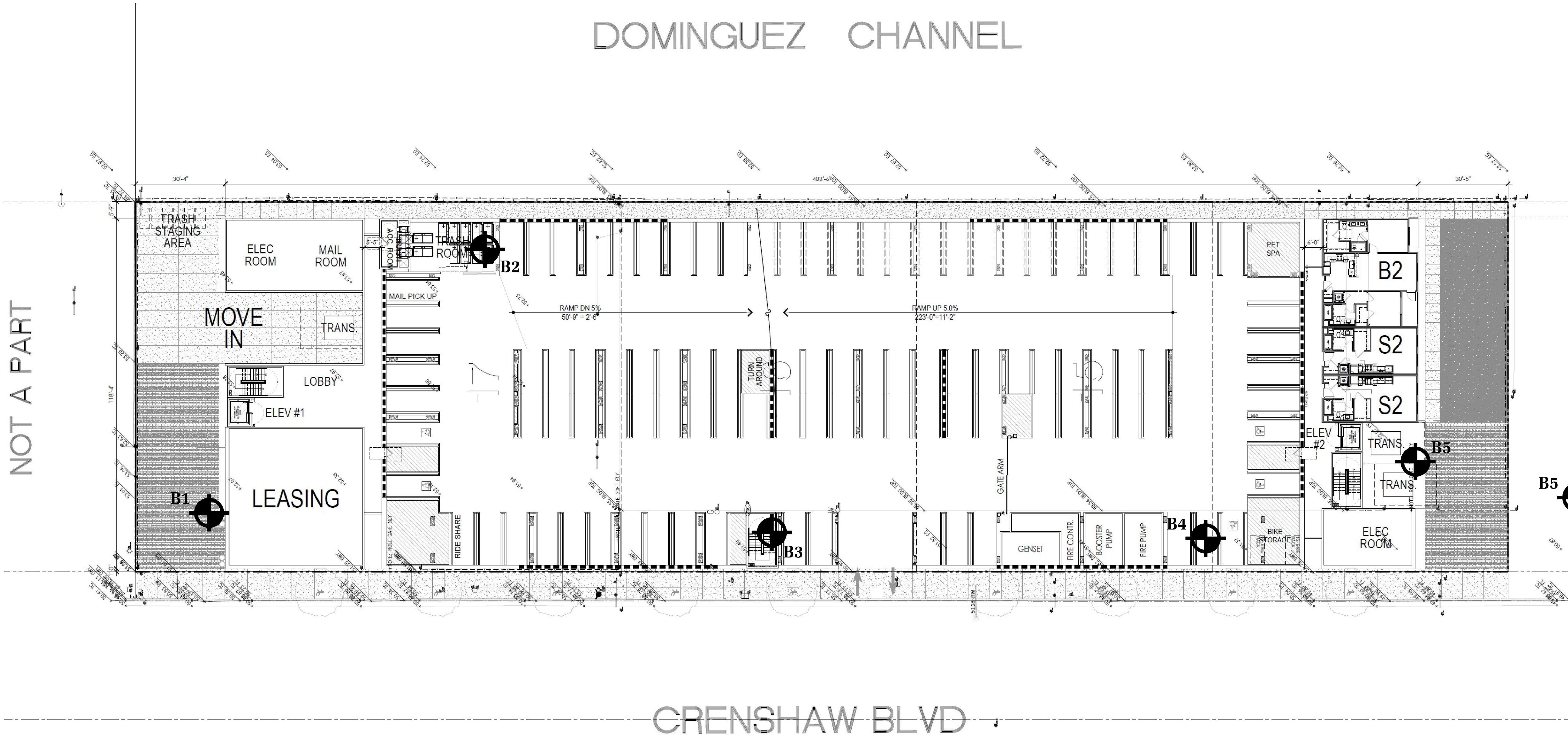
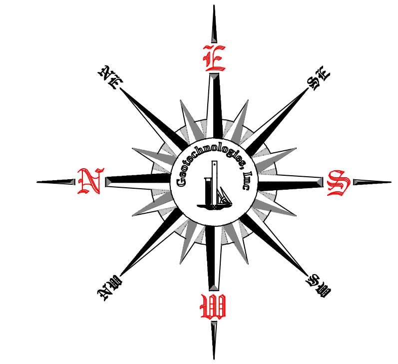
FILE NO. 21911





SCALE IN FEET

0 40 80

LEGEND

B5 LOCATION & NUMBER OF BORING

REFERENCE: GROUND LEVEL (BUILDING PLANS) PROVIDED BY AO ARCHITECTS DATED JANUARY 21, 2020

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

DIN/CAL 4, INC.
12850 CRENSHAW BLVD., GARDENA

FILE No. 21911 | DRAWN BY: TC

DATE: January 2020

BORING LOG NUMBER 1

The DIN/CAL 4, INC | **Date: 12/02/2019**

File No. 21911 | **Method: 8-inch diameter Hollow Stem Auger**

dy

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 --		2-inch Asphalt over 2-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, stiff, medium dense, fine grained
				-		
				2 --		
2.5	52	10.8	123.3	-		
				3 --		
				-	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt, dark to yellowish brown, moist, dense, fine grained, very stiff
				4 --		
				-		
5	81	17.3	118.3	5 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, very stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	29 50/5	9.7	130.7	10 --		
				-	SM	Silty Sand, dark and grayish brown, moist, very dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	72	14.3	121.9	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, gray to dark brown, moist, dense, fine grained, very stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	70	27.7	97.0	20 --		
				-	ML	Sandy to Clayey Silt, dark and yellowish brown, moist, very stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	58	15.1	118.4	25 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown and gray, moist, stiff, dense, fine grained

BORING LOG NUMBER 1

The DIN/CAL 4, INC

File No. 21911

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	32	17.6	115.4	30 --		
				- 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -		Total Depth 30 feet Water at 27½ feet Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 2

The DIN/CAL 4, Inc. **Date: 12/02/19**

File No. 21911 **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 Parking Lot
				0 --		2-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Clayey Sand, brown, moist, medium dense, fine to medium grained, gravel and concrete debris fragments
				-		
				2 --		
2.5	13	10.1	120.1	-		
				3 --	ML	ALLUVIUM: Sandy Silt, dark brown, moist, stiff, minor gravel
				-		
				4 --		
				-		
5	23	12.4	SPT	5 --		
				-	SC	Clayey Sand, brown, moist, medium dense, fine grained
				6 --		
				-		
				7 --		
7.5	49	12.4	123.9	-		
				8 --		slight mottling, dense
				-		
				9 --		
				-		
10	35	6.9	SPT	10 --		
				-	SM/CL	Silty Sand to Sandy Clay, light brown with slight mottling, slightly moist, dense, stiff, fine grained
				11 --		
				-		
				12 --		
12.5	52	12.5	120.5	-		
				13 --	CL	Sandy Clay, mottled olive brown, moist, very stiff
				-		
				14 --		
				-		
15	27	14.4	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	58	19.6	111.2	-		
				18 --		
				-		
				19 --		
				-		
20	29	20.3	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	37	27.4	99.3	-		
				23 --	CL	Silty Clay, mottled olive brown, moist, very stiff
				-		
				24 --		
				-		
25	30	22.6	SPT	25 --		
				-		

BORING LOG NUMBER 2

The DIN/CAL 4, Inc.

File No. 21911

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	53	16.2	120.1			
				28	SC	Clayey Sand, mottled brown, moist, dense, fine grained
				29		
30	22	19.1	SPT	30		
						wet, medium dense
				31		
				32		
32.5	37	20.8	106.2			
				33	SM	Silty Sand, mottled olive brown, wet, medium dense, fine grained
				34		
35	21	22.7	SPT	35		
						some clay intermixed
				36		
				37		
37.5	32	20.9	110.4			
				38		
				39		
40	20	33.3	SPT	40		
					ML/CL	Sandy Silt to Silty Clay, mottled olive brown, wet, stiff
				41		
				42		
42.5	23	19.1	104.7			
				43		
				44		
45	30	18.2	SPT	45		
					SM/SP	Silty Sand to Sand, mottled brown, wet, medium dense, fine grained
				46		
				47		
47.5	38 50/4"	16.7	117.1			
				48		very dense
				49		
50	33	18.9	SPT	50		
					SP/CL	Sand to Silty Clay, mottled olive brown, wet, medium dense, stiff, fine grained

BORING LOG NUMBER 2

The DIN/CAL 4, Inc.

File No. 21911

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	59	20.6	106.8	53	SP	Sand, brown, wet, dense, fine grained, some silt intermixed
				54		
55	31	19.7	SPT	55		
				56	SP/CL	Sand to Sandy Clay, olive brown with slight mottling, wet, stiff, medium dense, fine grained
				57		
57.5	37	21.8	109.4	58	SC	Clayey Sand, mottled olive brown, wet, medium dense, fine grained
				59		
60	31	26.0	SPT	60		
				61		Total Depth 60 feet Water at 28 feet Fill to 2½ 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

The DIN/CAL 4, Inc. **Date: 12/02/19**

File No. 21911 **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 Driveway
				0 --		2-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Clay, brown, moist, firm, debris fragments
				-		
				2 --		Sandy Silt, dark brown, moist, firm, debris
2.5	46	16.2	115.6	-		
				3 --	CL	ALLUVIUM: Sandy Clay, brown, moist, very stiff, primarily fine grained sand
				-		
				4 --		
				-		
5	44	12.6	123.7	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	61	10.7	128.5	10 --		
				-		light brown, some medium sand grains intermixed
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	47	18.2	115.3	15 --		
				-	ML	Sandy to Clayey Silt, yellowish to light olive brown, moist, very stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	40	27.0	99.6	20 --		
				-	CL	Silty Clay, mottled olive brown, moist, very stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	47	16.0	112.9	25 --		
				-	ML/CL	Sandy Silt to Silty Clay, olive to medium brown with slight mottling, moist, very stiff

BORING LOG NUMBER 3

The DIN/CAL 4, Inc.

File No. 21911

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29	ML	Sandy Silt, brown with slight mottling, moist, very stiff, some clay intermixed
30	53	19.7	112.2	30		
				31		Total Depth 30 feet No Water Fill to 2½ feet
				32		
				33		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				34		
				35		Used 8-inch diameter Hollow-Stem Auger 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

The DIN/CAL 4, Inc. **Date: 12/02/2019**

File No. 21911 **Method: 8-inch diameter Hollow Stem Auger**

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Slab
				0 --		3-inch Asphalt No Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, stiff, medium dense, fine grained
				-		
2.5	52	13.5	121.4	2 --		
				-		
				3 --		
				-	SM/ML	ALLUVIUM: Silty Sand to Sandy Silty, dark to yellowish brown, moist, dense, fine grained, stiff
				4 --		
				-		
5	34	16.7	116.8	5 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	57	12.9	121.3	10 --		
				-		Sandy Silt, dark and grayish brown, moist, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	55	17.6	116.7	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark and yellowish brown, moist, dense, fine grained, very stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	36	25.5	98.2	20 --		
				-	ML	Sandy to Clayey Silt, dark and yellowish brown, moist, very stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	48	16.2	114.6	25 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark and yellowish brown, moist, very stiff, dense, fined grained

BORING LOG NUMBER 4

The DIN/CAL 4, Inc.

File No. 21911

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		Silty Sand, dark and yellowish brown, wet, medium dense, fine
				-	SM	grained
30	23	25.2	100.3	30 --		
				-		Total Depth 30 feet
				31 --		Water at 26½ 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 --		
				-		

BORING LOG NUMBER 5

The DIN.CAL 4, Inc. **Date: 12/02/2019**

File No. 21911 **Method: 8-inch diameter Hollow Stem Auger**

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete Slab
				0 --		6-inch Concrete, No Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, stiff, medium dense
				-		
				2 --		
2.5	43	15.5	119.8	-		
				3 --		
				-	ML	ALLUVIUM: Sandy to Clayey Silt, dark brown, moist, very stiff
				4 --		
				-		
5	28	15.2	119.4	5 --		
				-	ML/CL	Clayey Silt to Silt, Clay, dark and yellowish brown, moist, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	72	11.3	126.4	10 --		
				-	ML/SM	Clayey Silt to Silty Sand, dark and yellowish brown, moist, dense, fined grained, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	49	15.8	118.5	15 --		
				-	SM	Silty Sand, yellowish brown, moist, dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	83	9.0	115.7	20 --		
				-	SP	Sand, dark brown, moist, very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	84	15.6	119.9	25 --		
				-	ML	Sandy Silt, dark and yellowish brown, moist, very stiff

BORING LOG NUMBER 5

The DIN.CAL 4, Inc.

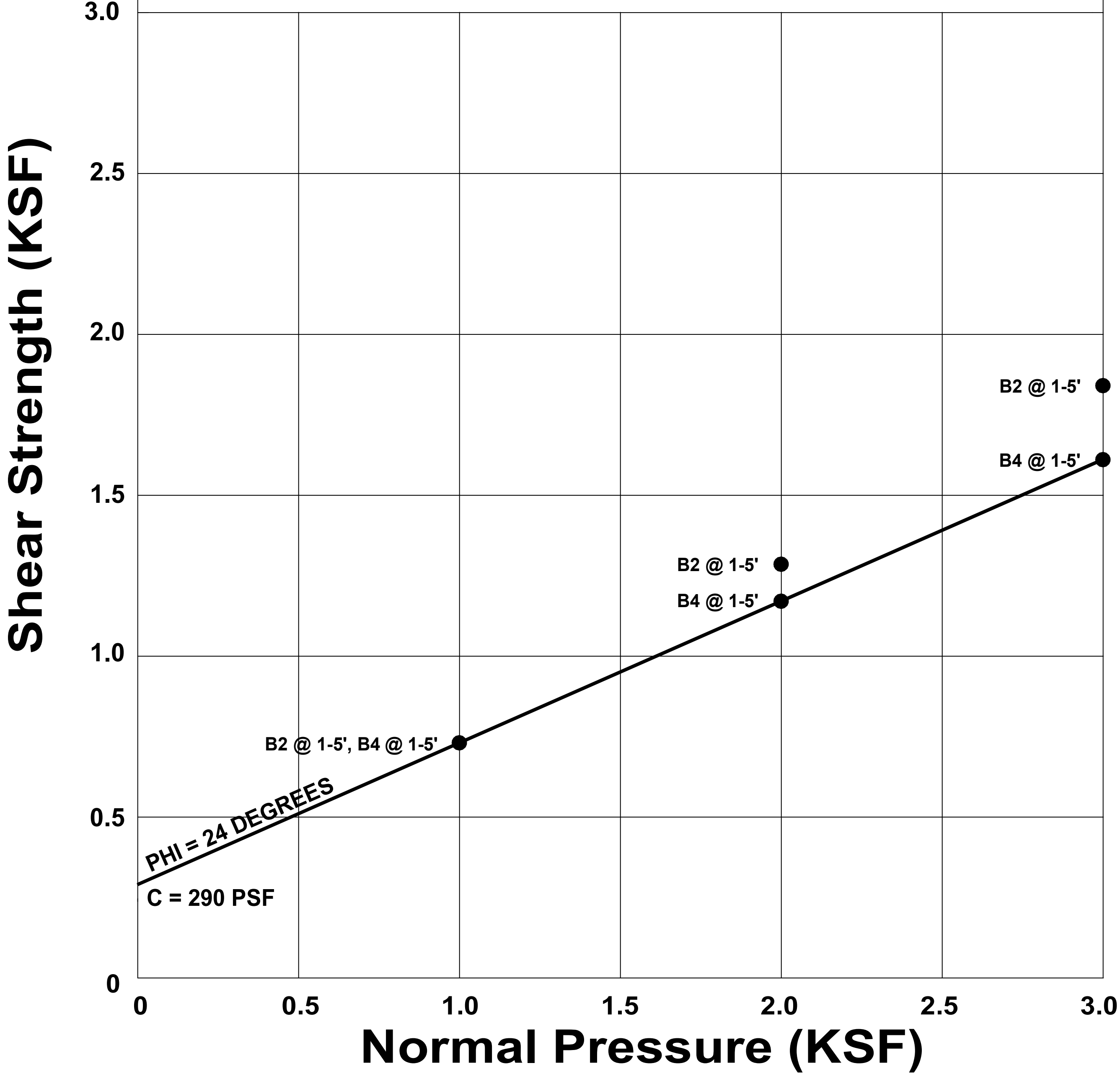
File No. 21911

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		Silty Sand, dark and yellowish brown, wet, medium dense,
				-	SM	fine grained
30	29	21.7	105.6	30 --		
				-		Total Depth 30 feet
				31 --		Water at 26½ 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 --		
				-		

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 1-5'	SM	133.3	8.5	14.7
B4 @ 1-5'	SM	131.9	8.9	14.7



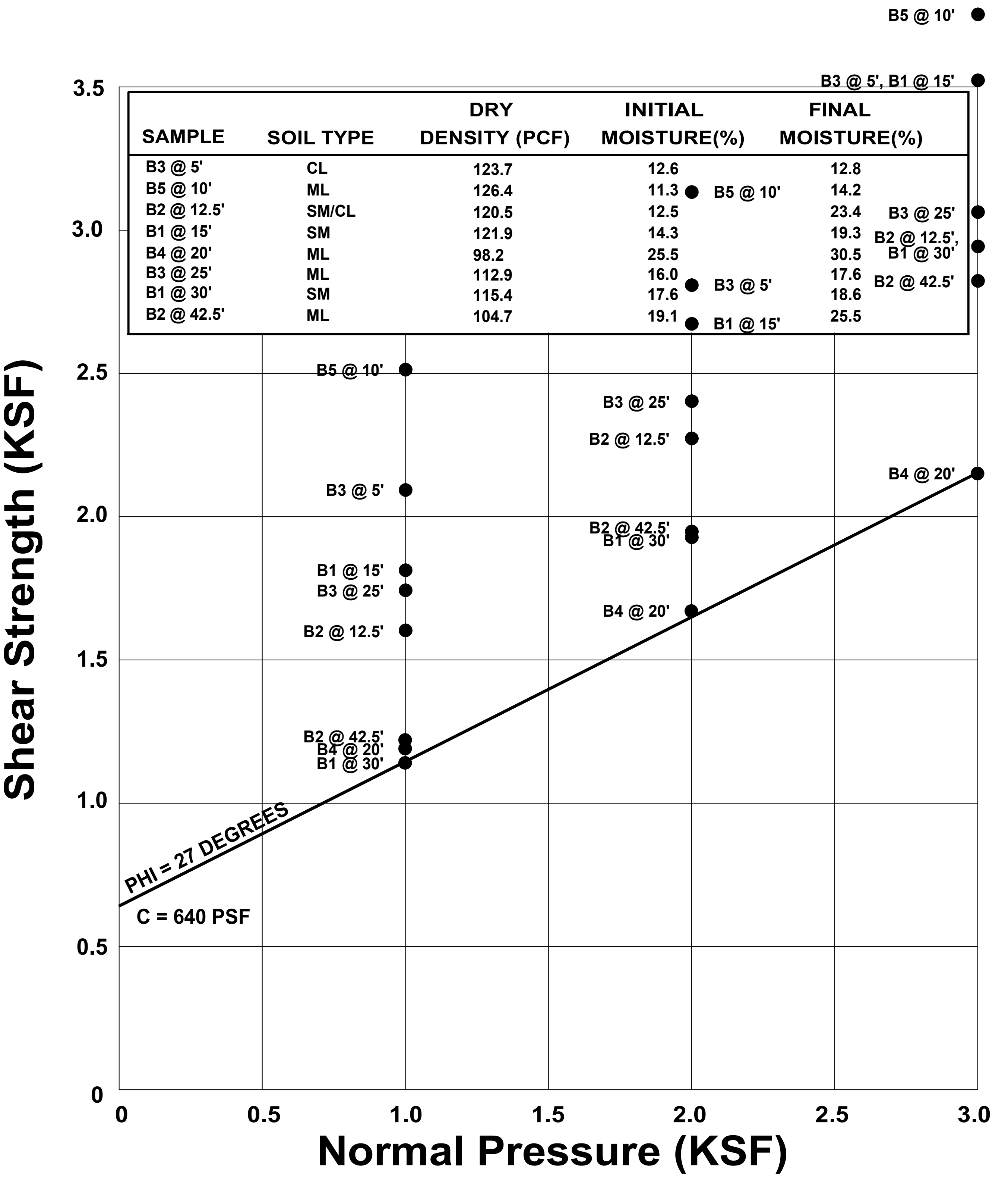
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

DIN/CAL 4, INC.

FILE NO. 21911 | PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B3 @ 5'	CL	123.7	12.6	12.8
B5 @ 10'	ML	126.4	11.3	14.2
B2 @ 12.5'	SM/CL	120.5	12.5	23.4
B1 @ 15'	SM	121.9	14.3	19.3
B4 @ 20'	ML	98.2	25.5	30.5
B3 @ 25'	ML	112.9	16.0	17.6
B1 @ 30'	SM	115.4	17.6	18.6
B2 @ 42.5'	ML	104.7	19.1	25.5

● Direct Shear, Saturated

SHEAR TEST DIAGRAM

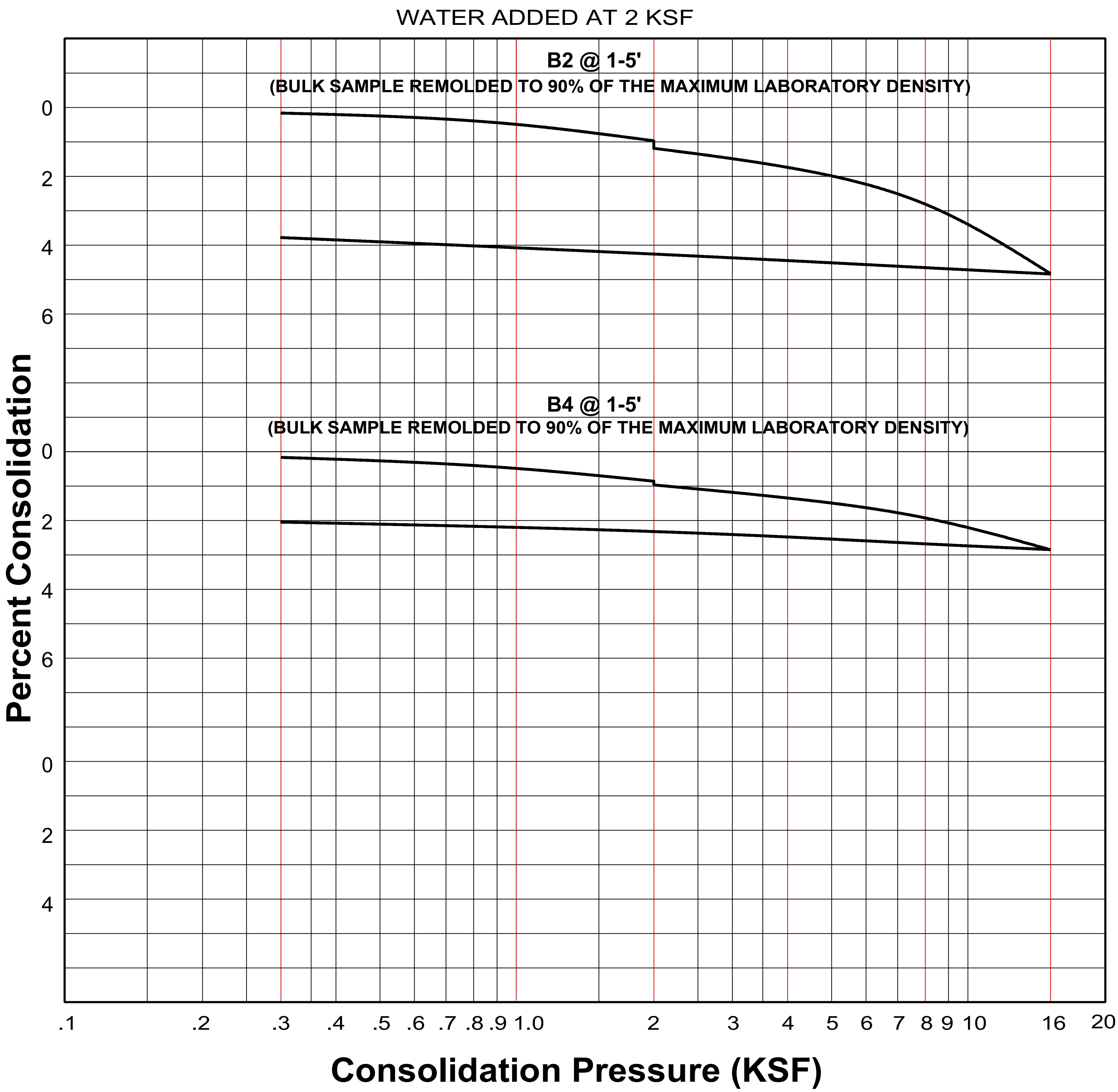
Geotechnologies, Inc.

Consulting Geotechnical Engineers

DIN/CAL 4, INC.

FILE NO. 21911

PLATE: B-2

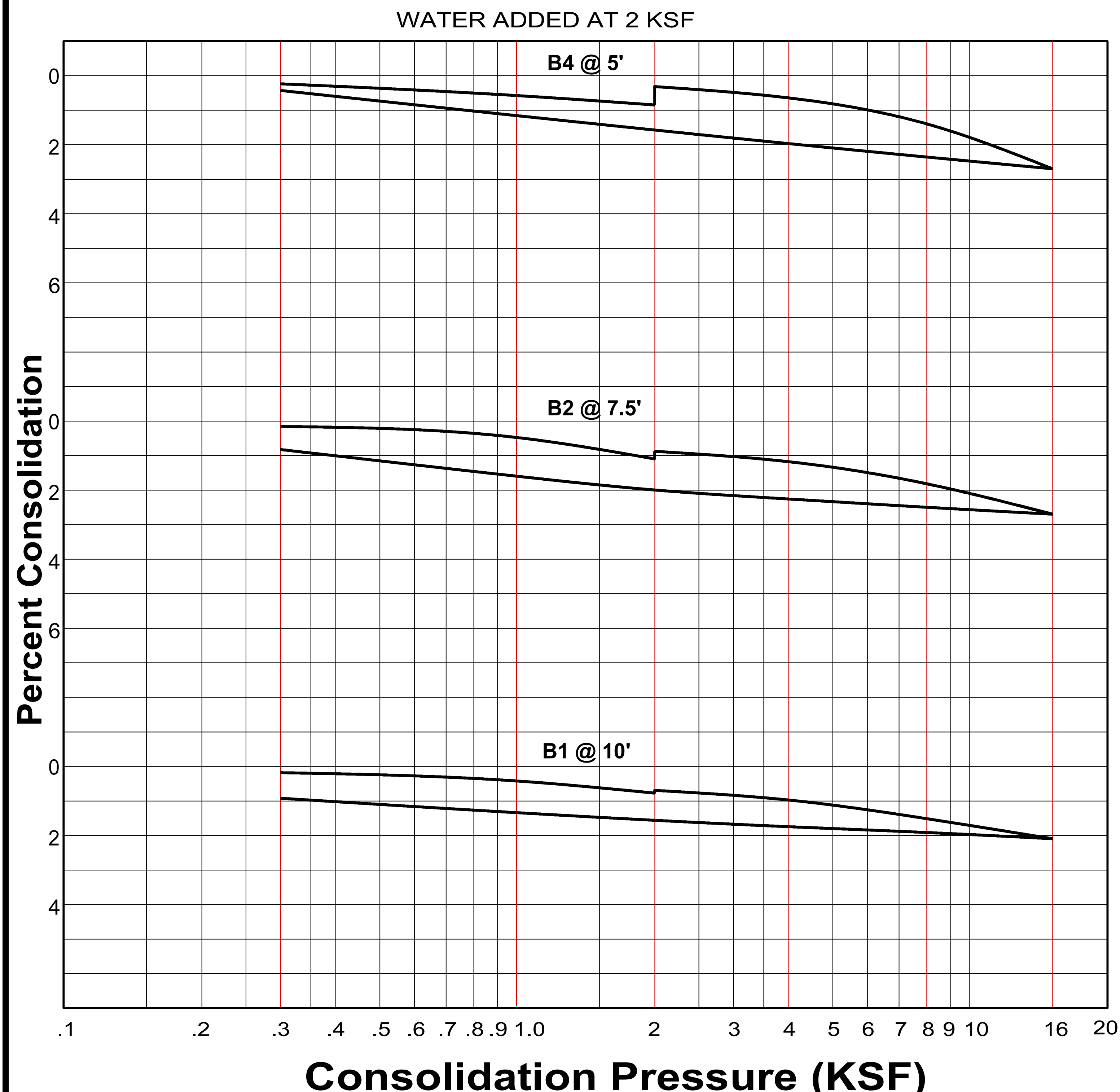


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

DIN/CAL 4, INC.

FILE NO. 21911 **PLATE: C-1**

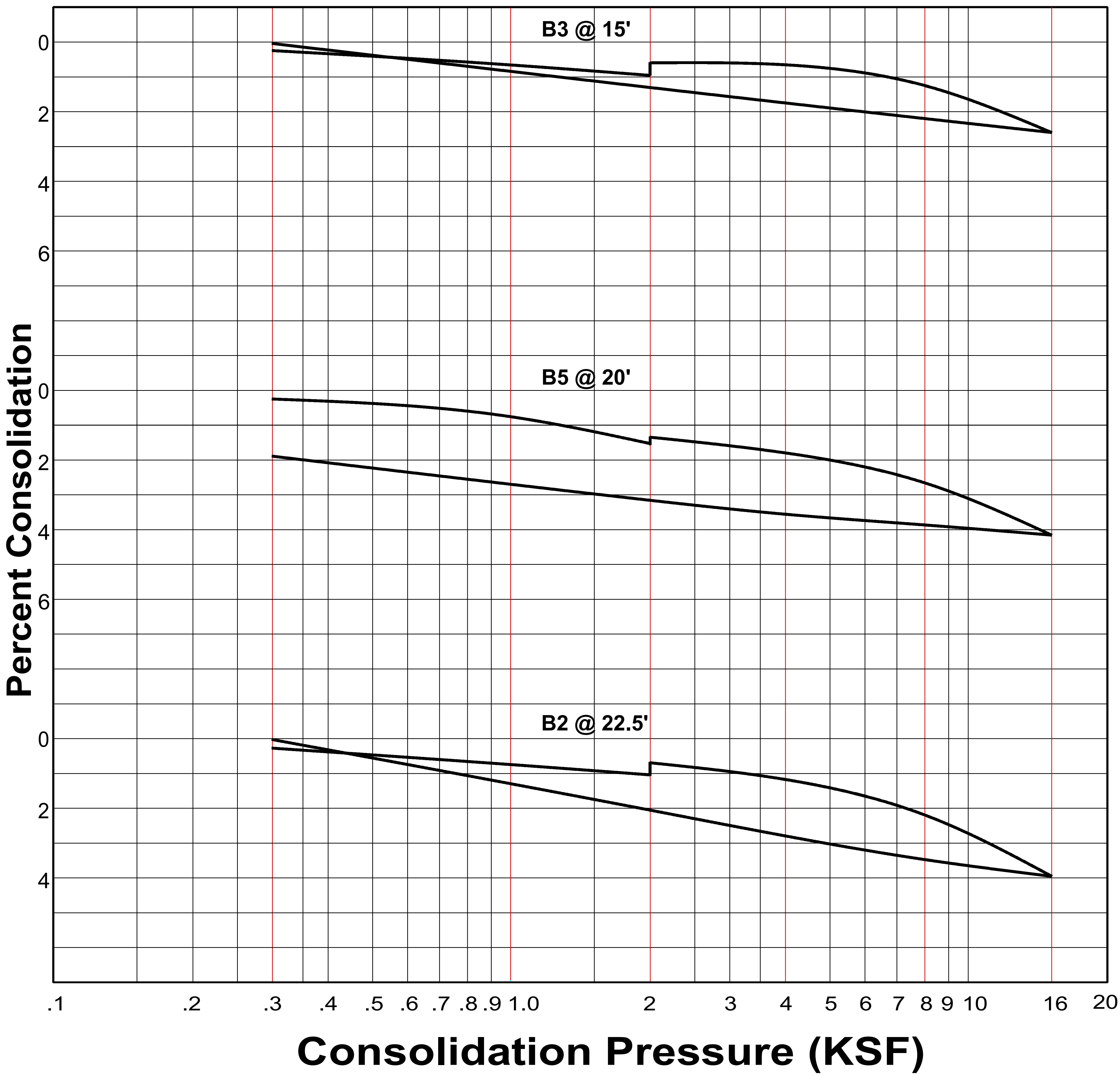


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

DIN/CAL 4, INC.	
FILE NO. 21911	**PLATE: C-2**

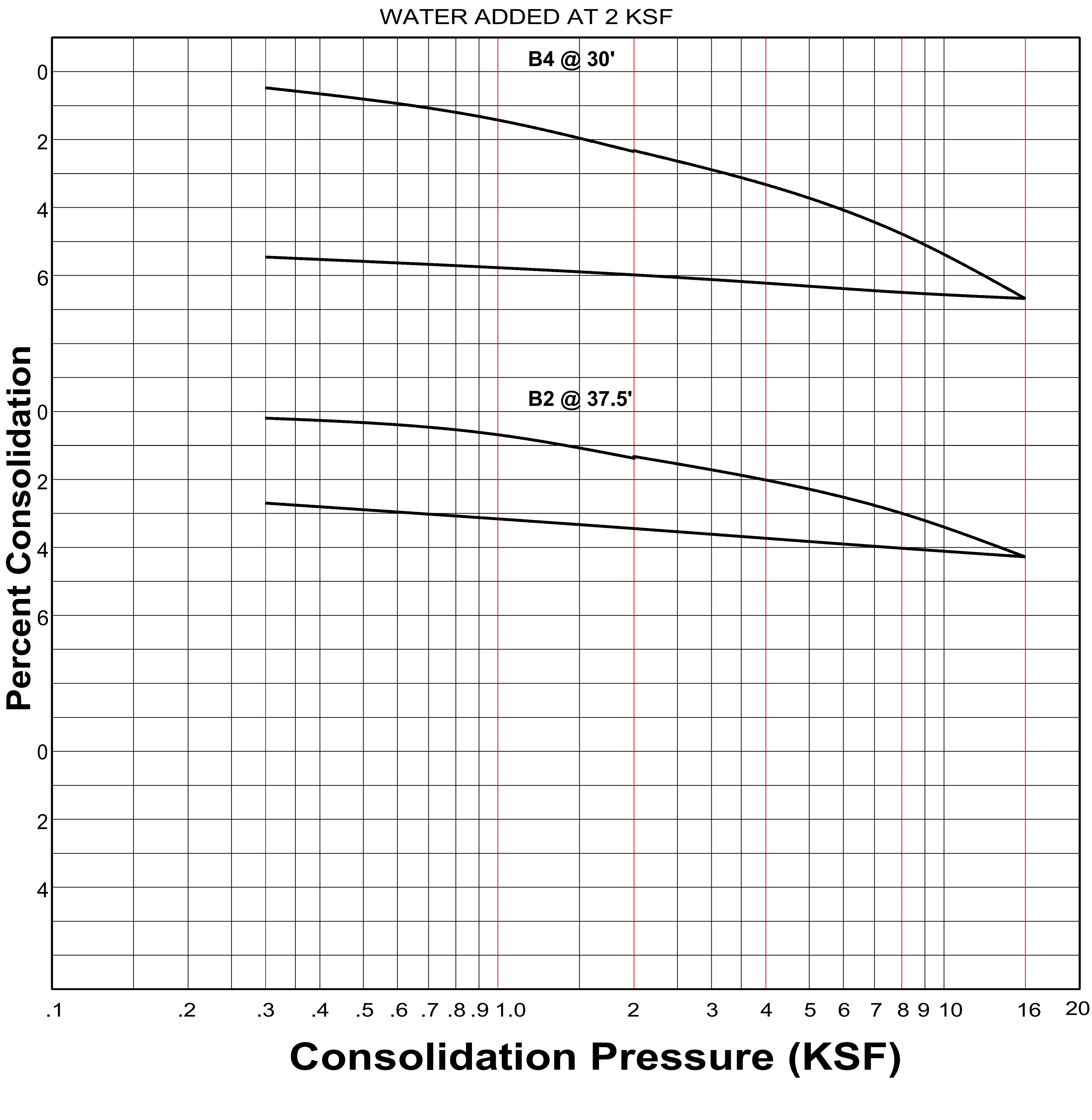
WATER ADDED AT 2 KSF



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

DIN/CAL 4, INC.	
FILE NO. 21911	PLATE: C-3



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

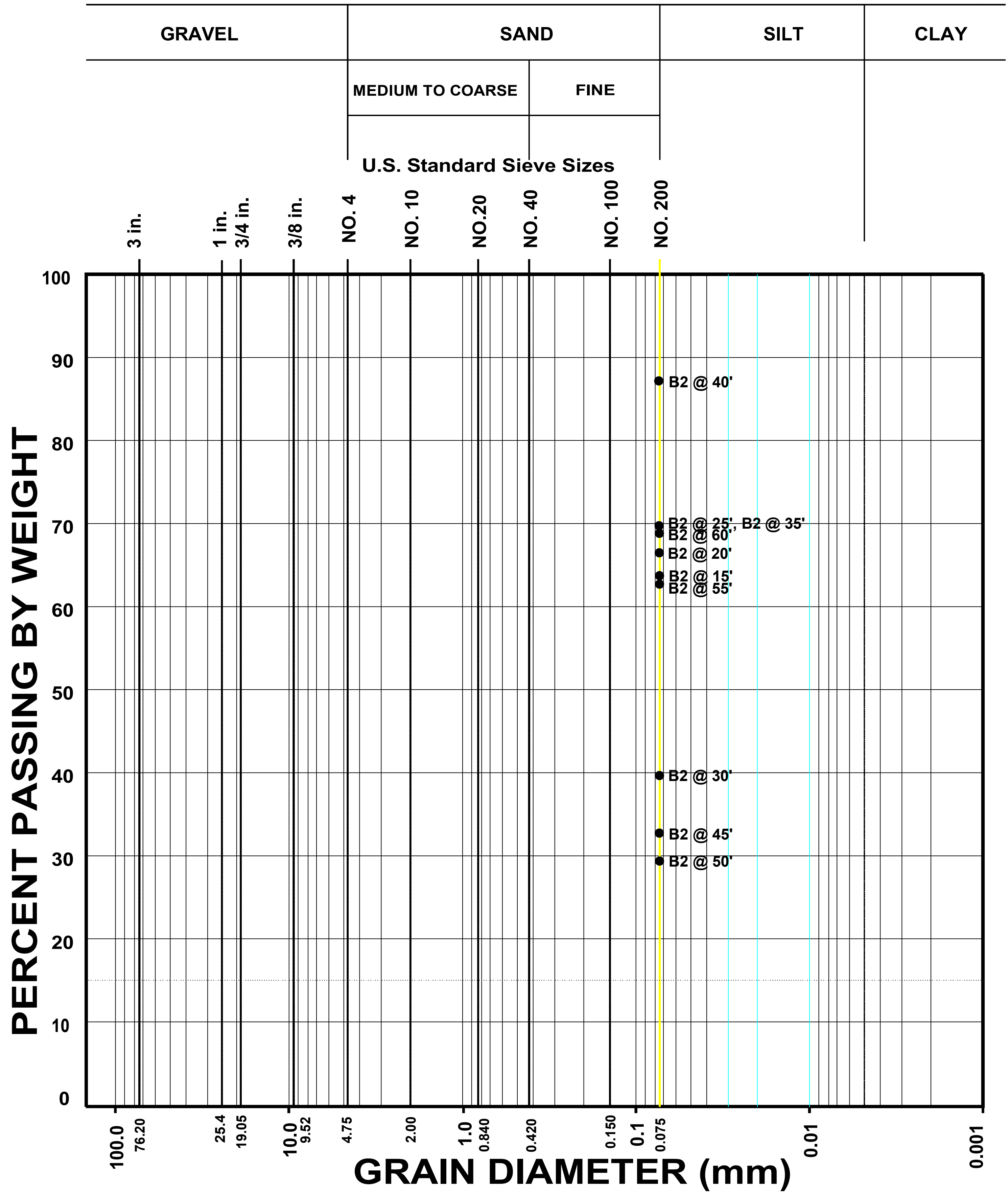
DIN/CAL 4, INC.	
FILE NO. 21911	**PLATE: C-4**

ASTM D-1557

SAMPLE	B2 @ 1- 5'	B4 @ 1-5'
SOIL TYPE:	SM	SM
MAXIMUM DENSITY pcf.	133.3	131.9
OPTIMUM MOISTURE %	8.5	8.9

ASTM D 4829-03

SAMPLE	B2 @ 1- 5'	B4 @ 1-5'
SOIL TYPE:	SM	SM
EXPANSION INDEX UBC STANDARD 18-2	10	28
EXPANSION CHARACTER	VERY LOW	LOW



GRAIN SIZE DISTRIBUTION

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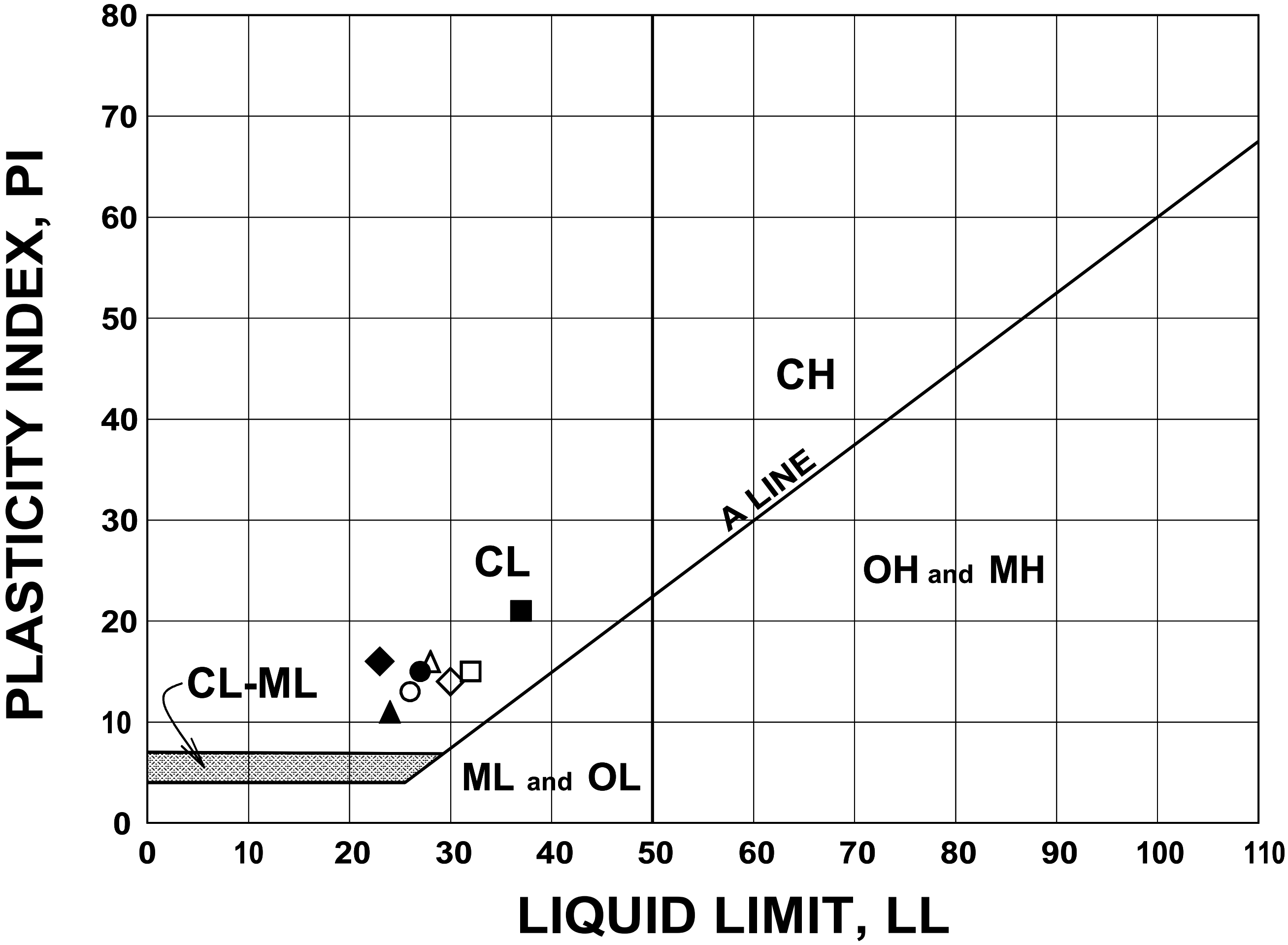
DIN/CAL 4, INC.

FILE NO. 21911	PLATE: E

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B2	15	○	26	13	13	CL
B2	20	●	27	12	15	CL
B2	25	△	28	12	16	CL
B2	30	▲	24	13	11	CL
B2	35	□	32	17	15	CL
B2	40	■	37	16	21	CL
B2	55	◇	30	16	14	CL
B2	60	◆	33	17	16	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
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DIN/CAL 4, INC.

FILE NO. 21911

PLATE: F

Geotechnologies, Inc.

Project: Din/Cal 4, Inc.
File No.: 21911
Description: Liquefaction Analysis
Boring Numbe 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.96
Calculated Mag.Wtg.Factor:	1.206

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	28.0
Historically Highest Groundwater Level* (ft):	25.0
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:	1.3

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	132.3	Unsaturated	Unsaturated	23	5	0.0	0	132.3	132.3	54.8	1.00	0.623	2.000	Non-Liq.	0.00
2	132.3	Unsaturated	Unsaturated	23	5	0.0	0	264.6	264.6	54.8	1.00	0.621	2.000	Non-Liq.	0.00
3	132.3	Unsaturated	Unsaturated	23	5	0.0	0	396.9	396.9	54.8	1.00	0.619	2.000	Non-Liq.	0.00
4	132.3	Unsaturated	Unsaturated	23	5	0.0	0	529.2	529.2	52.2	0.99	0.617	2.000	Non-Liq.	0.00
5	132.3	Unsaturated	Unsaturated	23	5	0.0	0	661.5	661.5	50.6	0.99	0.615	2.000	Non-Liq.	0.00
6	132.3	Unsaturated	Unsaturated	23	5	0.0	0	793.8	793.8	47.6	0.99	0.613	2.000	Non-Liq.	0.00
7	132.3	Unsaturated	Unsaturated	23	5	0.0	0	926.1	926.1	45.3	0.98	0.610	2.000	Non-Liq.	0.00
8	139.2	Unsaturated	Unsaturated	35	10	0.0	0	1065.3	1065.3	62.6	0.98	0.608	2.000	Non-Liq.	0.00
9	139.2	Unsaturated	Unsaturated	35	10	0.0	0	1204.5	1204.5	64.4	0.97	0.605	2.000	Non-Liq.	0.00
10	139.2	Unsaturated	Unsaturated	35	10	0.0	0	1343.7	1343.7	62.6	0.97	0.602	2.000	Non-Liq.	0.00
11	139.2	Unsaturated	Unsaturated	35	10	0.0	0	1482.9	1482.9	61.0	0.97	0.600	2.000	Non-Liq.	0.00
12	139.2	Unsaturated	Unsaturated	35	10	0.0	0	1622.1	1622.1	59.6	0.96	0.597	2.000	Non-Liq.	0.00
13	135.6	Unsaturated	Unsaturated	35	10	0.0	0	1757.7	1757.7	58.3	0.96	0.594	2.000	Non-Liq.	0.00
14	135.6	Unsaturated	Unsaturated	35	10	0.0	0	1893.3	1893.3	57.2	0.95	0.591	2.000	Non-Liq.	0.00
15	135.6	Unsaturated	Unsaturated	27	15	62.7	13	2028.9	2028.9	54.0	0.95	0.588	2.000	Non-Liq.	0.00
16	135.6	Unsaturated	Unsaturated	27	15	62.7	13	2164.5	2164.5	53.2	0.94	0.585	2.000	Non-Liq.	0.00
17	135.6	Unsaturated	Unsaturated	27	15	62.7	13	2300.1	2300.1	52.5	0.94	0.582	2.000	Non-Liq.	0.00
18	133.0	Unsaturated	Unsaturated	27	15	62.7	13	2433.1	2433.1	51.8	0.93	0.579	2.000	Non-Liq.	0.00
19	133.0	Unsaturated	Unsaturated	27	15	62.7	13	2566.1	2566.1	51.1	0.93	0.576	2.000	Non-Liq.	0.00
20	133.0	Unsaturated	Unsaturated	29	20	65.4	15	2699.1	2699.1	53.8	0.92	0.572	2.000	Non-Liq.	0.00
21	133.0	Unsaturated	Unsaturated	29	20	65.4	15	2832.1	2832.1	53.2	0.92	0.569	2.000	Non-Liq.	0.00
22	133.0	Unsaturated	Unsaturated	29	20	65.4	15	2965.1	2965.1	52.7	0.91	0.566	2.000	Non-Liq.	0.00
23	126.6	Unsaturated	Unsaturated	29	20	65.4	15	3091.7	3091.7	52.1	0.91	0.562	2.000	Non-Liq.	0.00
24	126.6	Unsaturated	Unsaturated	29	20	65.4	15	3218.3	3218.3	51.7	0.90	0.559	2.000	Non-Liq.	0.00
25	126.6	Unsaturated	Unsaturated	30	25	68.6	16	3344.9	3344.9	52.8	0.89	0.555	2.000	Non-Liq.	0.00
26	126.6	Unsaturated	Saturated	30	25	68.6	16	3471.5	3409.1	52.5	0.89	0.562	2.000	3.6	0.00
27	126.6	Unsaturated	Saturated	30	25	68.6	16	3598.1	3473.3	52.3	0.88	0.568	2.000	3.5	0.00
28	139.6	Unsaturated	Saturated	30	25	68.6	16	3737.7	3550.5	54.5	0.88	0.573	2.000	3.5	0.00
29	139.6	Saturated	Saturated	30	25	68.6	16	3877.3	3627.7	54.2	0.87	0.578	2.000	3.5	0.00
30	139.6	Saturated	Saturated	22	30	39.1	11	4016.9	3704.9	38.9	0.87	0.583	2.000	3.4	0.00
31	139.6	Saturated	Saturated	22	30	39.1	11	4156.5	3782.1	38.6	0.86	0.586	1.996	3.4	0.00
32	139.6	Saturated	Saturated	22	30	39.1	11	4296.1	3859.3	38.4	0.85	0.590	1.982	3.4	0.00
33	128.2	Saturated	Saturated	22	30	39.1	11	4424.3	3925.1	38.1	0.85	0.593	1.970	3.3	0.00
34	128.2	Saturated	Saturated	22	30	39.1	11	4552.5	3990.9	37.9	0.84	0.596	1.958	3.3	0.00
35	128.2	Saturated	Saturated	21	35	68.9	15	4680.7	4056.7	35.7	0.84	0.599	1.278	2.1	0.00
36	128.2	Saturated	Saturated	21	35	68.9	15	4808.9	4122.5	35.5	0.83	0.601	1.220	2.0	0.00
37	128.2	Saturated	Saturated	21	35	68.9	15	4937.1	4188.3	35.3	0.82	0.603	1.167	1.9	0.00
38	133.5	Saturated	Saturated	21	35	68.9	15	5070.6	4259.4	35.1	0.82	0.604	1.114	1.8	0.00
39	133.5	Saturated	Saturated	21	35	68.9	15	5204.1	4330.5	34.9	0.81	0.606	1.066	1.8	0.00
40	133.5	Saturated	Saturated	20	40	85.7	21	5337.6	4401.6	32.7	0.81	0.606	0.726	Non-Liq.	0.00
41	133.5	Saturated	Saturated	20	40	85.7	21	5471.1	4472.7	32.6	0.80	0.607	0.703	Non-Liq.	0.00
42	133.5	Saturated	Saturated	20	40	85.7	21	5604.6	4543.8	32.4	0.79	0.608	0.681	Non-Liq.	0.00
43	131.3	Saturated	Saturated	20	40	85.7	21	5735.9	4612.7	32.2	0.79	0.608	0.661	Non-Liq.	0.00
44	131.3	Saturated	Saturated	20	40	85.7	21	5867.2	4681.6	32.0	0.78	0.608	0.643	Non-Liq.	0.00
45	131.3	Saturated	Saturated	30	45	32.3	0	5998.5	4750.5	50.7	0.78	0.608	1.834	3.0	0.00
46	131.3	Saturated	Saturated	30	45	32.3	0	6129.8	4819.4	50.6	0.77	0.608	1.823	3.0	0.00
47	131.3	Saturated	Saturated	30	45	32.3	0	6261.1	4888.3	50.4	0.76	0.607	1.813	3.0	0.00
48	136.7	Saturated	Saturated	30	45	32.3	0	6397.8	4962.6	50.2	0.76	0.607	1.803	3.0	0.00
49	136.7	Saturated	Saturated	30	45	32.3	0	6534.5	5036.9	36.3	1.01	0.811	1.333	1.6	0.00
50	136.7	Saturated	Saturated	33	50	29.0	0	6671.2	5111.2	54.2	0.75	0.605	1.782	2.9	0.00
51	136.7	Saturated	Saturated	33	50	29.0	0	6807.9	5185.5	40.5	1.01	0.821	1.771	2.2	0.00
52	136.7	Saturated	Saturated	33	50	29.0	0	6944.6	5259.8	40.4	1.01	0.825	1.761	2.1	0.00
53	128.9	Saturated	Saturated	33	50	29.0	0	7073.5	5326.3	40.2	1.01	0.830	1.752	2.1	0.00
54	128.9	Saturated	Saturated	33	50	29.0	0	7202.4	5392.8	40.0	1.01	0.835	1.743	2.1	0.00
55	128.9	Saturated	Saturated	31	55	61.7	14	7331.3	5459.3	37.0	1.01	0.840	1.520	1.8	0.00
56	128.9	Saturated	Saturated	31	55	61.7	14	7460.2	5525.8	36.9	1.01	0.844	1.465	1.7	0.00
57	128.9	Saturated	Saturated	31	55	61.7	14	7589.1	5592.3	36.7	1.01	0.848	1.414	1.7	0.00
58	133.2	Saturated	Saturated	31	60	67.7	16	7722.3	5663.1	36.6	1.01	0.853	1.357	1.6	0.00
59	133.2	Saturated	Saturated	31	60	67.7	16	7855.5	5733.9	36.4	1.01	0.857	1.310	1.5	0.00
60	133.2	Saturated	Saturated	31	60	67.7	16	7988.7	5804.7	50.0	0.69	0.590	1.691	2.9	0.00

Total Liquefaction Settlement, S = 0.00 inches

GEOTECHNOLOGIES, INC.

FILE NO.: 21911
PROJECT: DIN/CAL 4, Inc.
BORING 2

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

INPUT:

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.8
Peak Horiz. Acceleration (g):	0.96

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Soil Type	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	Percent Passing 200 Sieve	ΔN for Fines Content	Fines Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] / [Gmax]	[geff]	[geff]*100%	Volumetric Strain [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
5.0	5.0	SC	2.5	132.3	0.17	0.11	0.103	-	-		-	-	-	-	-	-	-	-		5 FOOT REMOVAL / RECOMPACTION			0.00
10.0	5.0	SM/CL	7.5	139.2	0.52	0.35	0.322	35	1.3	100.0	1.33	60.3	0.0	0.0	60.3	1036.548	2.71E-04	1.55E-03	1.55E-01	1.18E-02	9.2675	0.0095	0.01
15.0	5.0	CL	12.5	135.6	0.85	0.57	0.518	27	1.3	92.7	1.09	38.2	62.7	5.5	43.7	1186.169	3.50E-04	1.41E-03	1.41E-01	3.80E-02	9.2675	0.0306	0.04
20.0	5.0	CL	17.5	133.0	1.16	0.78	0.701	29	1.3	89.5	0.95	35.6	65.4	5.5	41.1	1362.434	3.84E-04	1.28E-03	1.28E-01	3.95E-02	9.2675	0.0318	0.04
25.0	5.0	CL	22.5	126.6	1.42	0.95	0.842	30	1.3	87.5	0.86	33.5	68.6	5.5	39.0	1481.294	3.98E-04	1.13E-03	1.13E-01	3.90E-02	9.2675	0.0314	0.04
30.0	5.0	SC	27.5	139.6	1.76	1.18	1.014	22	1.3	71.4	0.79	22.6	39.1	5.5	28.1	1474.355	4.55E-04	1.45E-03	1.45E-01	9.90E-02	9.2675	0.0797	0.10

*Groundwater Encountered @ 28 Feet

Total Calculated Dynamic Dry Settlement (inches) 0.22

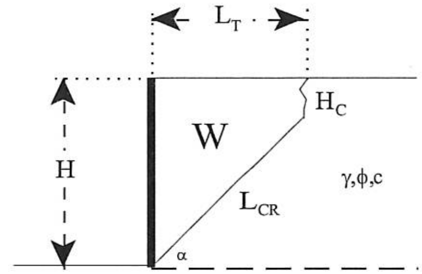
Geotechnologies, Inc.

Project: Din/Cal 4, Inc.
File No.: 21911
Description: Retaining Walls up to 6 feet

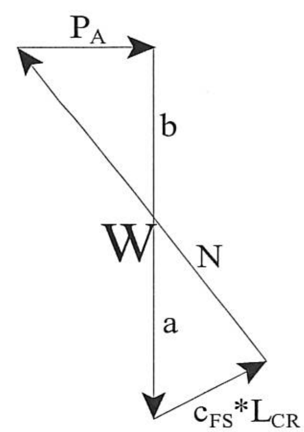
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	6.00 feet
Unit Weight of Retained Soils	(γ)	130.0 pcf
Friction Angle of Retained Soils	(ϕ)	24.0 degrees
Cohesion of Retained Soils	(c)	290.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	16.5 degrees
	(c_{FS})	193.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.2	9	1177.0	2.5	973.4	203.7	110.4
46	4.2	9	1167.2	2.5	957.4	209.7	118.5
47	4.1	9	1151.8	2.6	938.2	213.6	125.7
48	4.1	9	1131.9	2.6	916.5	215.4	131.8
49	4.0	9	1108.2	2.6	892.9	215.3	137.0
50	4.0	8	1081.2	2.6	867.8	213.4	141.1
51	4.0	8	1051.5	2.6	841.6	209.9	144.1
52	4.0	8	1019.4	2.5	814.4	205.0	146.0
53	4.0	8	985.2	2.5	786.5	198.7	146.9
54	4.0	7	949.3	2.5	758.0	191.3	146.6
55	4.0	7	911.8	2.4	729.0	182.8	145.3
56	4.0	7	873.0	2.4	699.6	173.5	142.8
57	4.0	6	832.9	2.3	669.7	163.3	139.3
58	4.1	6	791.8	2.3	639.3	152.4	134.7
59	4.1	6	749.5	2.2	608.5	141.0	129.1
60	4.1	5	706.3	2.1	577.1	129.2	122.4
61	4.2	5	662.1	2.1	545.1	117.0	114.8
62	4.3	5	617.0	2.0	512.4	104.6	106.4
63	4.3	4	570.9	1.9	478.7	92.2	97.0
64	4.4	4	523.8	1.8	444.0	79.8	87.0
65	4.5	4	475.6	1.6	408.0	67.6	76.3
66	4.6	3	426.3	1.5	370.5	55.8	65.2
67	4.7	3	375.7	1.4	331.3	44.4	53.8
68	4.9	2	323.8	1.2	290.0	33.8	42.5
69	5.0	2	270.3	1.1	246.2	24.1	31.4
70	5.2	2	215.0	0.9	199.4	15.6	21.1



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 146.89 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 8.2 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project: Din/Cal 4, Inc.
File No.: 21911
Description: Slot Cut

Slot Cut Calculation

Input:

Height of Slots	(H)	6.0 feet
Unit Weight of Soils	(γ)	130.0 pcf
Friction Angle of Soils	(ϕ)	24.0 degrees
Cohesion of Soils	(c)	290.0 psf
Factor of Safety	(FS)	1.25

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.5

Surcharge Pressure:

Line Load	(q_L)	2000.0 psf
Distance Away from Edge of Excavation	(X)	2.0 feet

Design Equations

$b = H/(\tan\alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin\alpha)*(\cos\alpha)$

$F_2 = d*L$

$R_1 = d*[W*(\cos^2\alpha)*(\tan\phi)+(c*b)]$

$R_2 = 2*\Delta F$

$\Delta F = A*[1/3*\gamma*H*K_o*(\tan\phi)+c]$

FS = Resistance Force/Driving Force

$FS = (R_1+R_2)/(F_1+F_2)$

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
45	6.0	18	2340.0	2170.0	2706.1	6261.8	1971.1
46	5.8	17	2259.7	2128.6	2595.5	6047.0	185.4
47	5.6	17	2182.1	2085.9	2488.6	5839.3	98.3
48	5.4	16	2106.9	2042.2	2385.4	5638.2	67.4
49	5.2	16	2034.1	1997.4	2285.6	5443.3	51.6
50	5.0	15	1963.5	1951.6	2189.1	5254.3	42.0
51	4.9	15	1894.9	1904.9	2095.8	5070.7	35.5
52	4.7	14	1828.2	1857.2	2005.5	4892.3	31.0
53	4.5	14	1763.3	1808.8	1918.0	4718.6	27.5
54	4.4	13	1700.1	1759.5	1833.3	4549.5	24.9
55	4.2	13	1638.5	1709.5	1751.3	4384.6	22.7
56	4.0	12	1578.3	1658.9	1671.8	4223.7	21.0
57	3.9	12	1519.6	1607.7	1594.8	4066.5	19.6
58	3.7	11	1462.2	1555.9	1520.1	3912.8	18.4
59	3.6	11	1406.0	1503.7	1447.8	3762.5	17.4
60	3.5	10	1351.0	1451.0	1377.6	3615.3	16.6
61	3.3	10	1297.1	1398.0	1309.5	3471.0	15.8
62	3.2	10	1244.2	1344.8	1243.5	3329.5	15.2
63	3.1	9	1192.3	1291.3	1179.5	3190.6	14.7
64	2.9	9	1141.3	1237.7	1117.4	3054.1	14.2
65	2.8	8	1091.2	1184.0	1057.2	2919.9	13.8
66	2.7	8	1041.8	1130.3	998.7	2787.9	13.5
67	2.5	8	993.3	1076.6	942.0	2658.0	13.2
68	2.4	7	945.4	1023.0	887.0	2529.9	12.9
69	2.3	7	898.2	969.7	833.6	2403.7	12.7
70	2.2	7	851.7	916.5	781.8	2279.1	12.5

* Width of Slots to achieve a minimum o f 1.5 Factor of Saferty, with a Maximum Allowable Slot Width of 8-feet.

Critical Slot Width with Factor of Safety equal or exceeding 1.5:

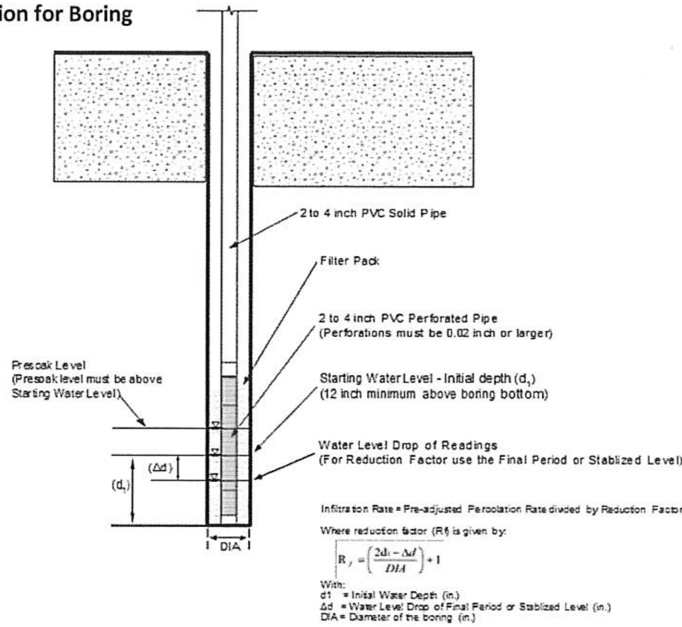
d_{allow} 12.5 feet

The proposed excavation may be made using the A-B-C Slot-Cutting Method with a Maximum Allowable Slot Width of 8 Feet, and up to 6 Feet in Height, with a Factor of Safety Equal or Exceeding 1.5.

Test Date: 2-Dec-19
File No. 21911
File Name : Din/Cal 4, Inc.

Infiltration Rate Calculation for Boring

Testing Boring Number		4
Boring Diameter (DIA)		8 inches
Depth of Boring		30 feet
Ground surface elevation	N.A.	feet
Length of Casing (dc)		20 feet
Top of Casing elevation	N.A.	feet
finish floor elevation	N.A.	feet
Elevation Bottom of Casing	#VALUE!	feet
Pre-soak Time		3 hours
Measured By	H.C.	



Terms

Initial water depth (d1) =dc-di
Water level drop (Δd) = di-df

Reading Number	Clock Time	Elapsed Time	Water Measurement (d_i) and (d_f)	Percolation Rate	Preadjusted Percolation Rate		Initial Water depth (d1)	Water level Drop (Δd)	Raw Percolation Rate	Percolation Rate Variation
							d1 = dc-di	Δd = di-df	Vol. H2o / Bor. Surface Area	
		Min	feet	ft/min	in/hour		in	in	in/hr	Percent
1	14:28		10.00				120			
	14:58	30	10.17	0.01	4.00			2.0	0.1	
2	15:00		10.00				120			
	15:30	30	10.17	0.01	4.00			2.0	0.1	0.0
3	15:31		10.00				120			
	16:01	30	10.17	0.01	4.00			2	0.1	0.0
4										
				#DIV/0!	#DIV/0!			0	#DIV/0!	#DIV/0!
5										
				#DIV/0!	#DIV/0!			0	#DIV/0!	#DIV/0!
6										
				#DIV/0!	#DIV/0!			0	#DIV/0!	#DIV/0!
7										
				#DIV/0!	#DIV/0!			0	#DIV/0!	#DIV/0!

Note: Calculation based on County of Los Angeles, Administrative Manual, Low Impact Development Best Management PracticeGuideline for Design, Investigation, and Reporting, dated 6/30/17.
LA County Minimum 0.3 Inches per hour

Measured Percolation Rate= 0.07 in/hr
R_f= 2.0
CF_v= 1
CF_s= 1

Design Infiltration Rate = **0.03 in/hr**

December 24, 2019 via email: stang@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Ave.
Glendale, CA 91201

Attention: Stanley Tang

Re: Soil Corrosivity Study
The Dinerstein Company
Gardena, CA
HDR #19-0881, Your #21911

Introduction

Laboratory tests have been completed for the Dinerstein Company project. Laboratory tests have been completed on one soil sample provided to HDR for the referenced project. The purpose of these tests was to determine if the soil might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR) assumes that the sample provided is representative of the most corrosive soils at the site.

The proposed structure has eight stories and no subterranean levels. The site is located at 12850 Crenshaw Boulevard in Gardena, California, and the water table is reportedly greater than 80 feet below ground level.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

Laboratory Soil Corrosivity Tests

The electrical resistivity of the sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivity is at about its lowest value when the soil is saturated. The pH of the saturated sample was measured per ASTM G51. A 5:1 water:soil extract from the sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B[1]. Laboratory test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[2]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivity was in the moderately corrosive category with as-received moisture and at saturation.

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[1] American Public Health Association (APHA). 2012. Standard Methods of Water and Wastewater. 22nd ed. American Public Health Association, American Water Works Association, Water Environment Federation publication. APHA, Washington D.C.

[2] Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.

The soil pH value was 7.7. This value is mildly alkaline.[3] This value does not particularly increase soil corrosivity.

The soluble salt content of the sample was low. Chloride and sulfate were found at low concentrations.

The nitrate concentration was high enough to be aggressive to copper. Ammonium was not detected.

Tests were not made for sulfide and oxidation-reduction (redox) potential because this sample did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as moderately corrosive to ferrous metals and aggressive to copper.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and the possible future application of cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:

    a. At each end of the pipeline.

---

[3] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

b. At each end of all casings.

c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:

   a. Dissimilar metals.

   b. Dissimilarly coated piping (cement-mortar vs. dielectric).

   c. Above ground steel pipe.

   d. All existing piping.

   Insulated joints should be placed above grade or in vaults where possible. Wrap all buried insulators with wax tape per AWWA C217.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 *or*

      ii. Extruded polyethylene per AWWA C215 *or*

      iii. A tape coating system per AWWA C214 *or*

      iv. Hot applied coal tar enamel per AWWA C203 *or*

      v. Fusion bonded epoxy per AWWA C213.

   b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

As an alternative to dielectric coating and possible future cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete three inches thick, using any type of ASTM C150 cement. Joint bonds, test stations, and insulated joints are still recommended for this alternative.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevators

1. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders with a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 or

      ii. Extruded polyethylene per AWWA C215 or

      iii. A tape coating system per AWWA C214 or

      iv. Hot applied coal tar enamel per AWWA C203 or

      v. Fusion bonded epoxy per AWWA C213.

   b. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.

   c. Apply cathodic protection to hydraulic cylinders as per NACE SP0169.

   OPTION 2

   As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

2. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating.

   b. Electrically isolate the pipeline.

   c. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   Place the oil line in a PVC casing pipe with solvent-welded joints and sealed at both ends to prevent contact with soil and moisture.

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future application of cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:

   a. At each end of the pipeline.

   b. At each end of any casings.

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable coating intended for underground use such as:

i. Polyethylene encasement per AWWA C105; *or*

ii. Epoxy coating; *or*

iii. Polyurethane; *or*

iv. Wax tape.

NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Although it is customary to cathodically protect coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

As an alternative to the coating systems described in Option 1 and possible future cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement.

NOTE: Some iron piping systems, such as for fire water piping, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.

2. It is not necessary to bond the pipe joints or apply cathodic protection.

3. Provide 6 inches of clean sand backfill all around the pipe.

Clean Sand Backfill

1. Clean sand backfill must have the following parameters:

    a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*

    b. pH between 6.0 and 8.0.

2. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

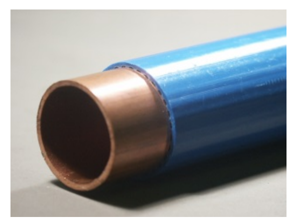
1. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.

2. Electrically insulate cold water piping from hot water piping systems.

3. Protect buried copper tubing by one of the following measures:

    a. Prevention of soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing using PVC pipe with solvent-welded joints.

    b. Installation of a factory-coated copper pipe with a minimum 25-mil thickness such as Kamco's Aqua Shield™, Mueller's Streamline Protec™, or equal. The coating must be continuous with no cuts or defects.



    c. Installation of 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE SP0169.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately sized cathodic protection per NACE SP0169.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, from 0 to 0.10 percent.[4,5,6]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with this soil due to the low chloride concentration[7] found onsite. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

Post-Tensioned Slabs: Unbonded Single-Stranded Tendons and Anchors

Soil is considered an aggressive environment for post-tensioning strands and anchors. Protect post-tensioning strands and anchors against corrosion by implementing all the following measures:[8,9,10]

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[4] *2015 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[5] *2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[6] *2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

[8] *Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.*

[9] *PTI M10.2-00: Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2000.*

[10] *ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Concrete Institute (ACI), 2001*

1. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.06 percent by weight of cement.

2. All tendons should be designed to prevent ingress of moisture. A corrosion-inhibiting coating should be incorporated into the tendon sheaths.

3. Use non-shrink grout mixes for all post-tensioning pockets.

4. Prior to grouting the pocket, apply a corrosion protection cap filled with corrosion protection material that provides a watertight seal for the strand end and wedge cavity, such as Tiger Industries' PocketCap or equal. Ensure the cap fully seats against the face of the standard anchor at the live end.

5. All components exposed to the job site should be protected within one working day after their exposure during installation.

6. Ensure the minimum concrete cover over the tendon tail is one inch, or greater if required by the applicable building code.

7. Caps should be installed within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.
8. Limit the access of direct runoff onto the anchorage area by designing proper drainage. Do not allow water to pond against anchors.

9. Provide at least two inches of space between finish grade and the anchorage area, or more if required by applicable building codes.

Expanded Analysis

Because only a single sample was submitted for soil corrosivity analysis, recommendations are based on a worst-case scenario. The owner may find it advantageous to consider retesting the site more extensively in order to allow for the appropriate scaling of mitigative measures to match the corrosivity of the various regions of the site, thereby removing the alternate need of applying the worst-case corrosivity to the entire site.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory sample. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

REGISTERED PROFESSIONAL ENGINEER
SEAN O'CASEY HOSS
79964
CIVIL
STATE OF CALIFORNIA

Steven Pierce
Corrosion Coordinator

Sean Hoss, PE

Enc: Table 1

HDR

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
The Dinerstein Company
Your #21911, HDR Lab #19-0881SCS
20-Dec-19

Sample ID			B 3 @ 1'-5' SM
Resistivity		**Units**	
as-received		ohm-cm	6,400
saturated		ohm-cm	2,240
pH			7.7
Electrical Conductivity		mS/cm	0.13
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	30
magnesium	Mg^{2+}	mg/kg	7.1
sodium	Na^{1+}	mg/kg	128
potassium	K^{1+}	mg/kg	2.4
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	229
fluoride	F^{1-}	mg/kg	8.6
chloride	Cl^{1-}	mg/kg	10
sulfate	SO_4^{2-}	mg/kg	86
phosphate	PO_4^{3-}	mg/kg	8.3
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	86
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

January 9, 2019
Project No. 19094A

Mr. Stan Tang
Geotechnologies, Inc.
439 Western Ave.
Glendale, CA 91201

**SUBJECT: SURFACE WAVE GEOPHYSICAL SURVEY RESULTS
12850 CRENSHAW BOULEVARD
GARDENA, CALIFORNIA**

Dear Mr. Tang,

Per your request and in accordance with the provisions of our proposal, dated November 20, 2019, GeoPentech performed surface wave geophysical measurements along three survey lines (SW19-1 through SW19-3) at the subject property located at 12850 Crenshaw Boulevard in Gardena, California. The locations of the geophysical measurements are shown on Figure 1.

Project Understanding

We understand that the proposed project includes the development of an eight-story structure at grade. We also understand that a surface-wave geophysical investigation was necessary to measure the shear-wave velocity profile at the site to evaluate the site V_{S30}. This letter summarizes the results of the surface wave surveys and the evaluation of V_{S30}.

Surface Wave Geophysics Methods

An active surface seismic wave survey was performed at the site using Multi-channel Analysis of Surface Waves (MASW) methods. A detailed description of MASW is provided in Park et al. (1999)[1]. In general, the MASW surface wave method records Rayleigh waves generated by striking the ground surface with a sledgehammer (active source). In a layered medium, Rayleigh surface waves of different frequencies (or wavelengths) propagate at different velocities, referred to as phase velocity. This phase velocity primarily depends on the material stiffness properties (e.g. S-wave velocity) over a depth approximately equal to one wavelength. Consequently, lower frequency, longer wavelength surface wave energy will provide samples to greater survey depths than higher frequency, shorter wavelength energy. Because surface waves of different frequencies (wavelengths) sample different depths, they travel at different velocities (dispersion) in a layered medium. Surface wave geophysical surveys measure the dispersive nature of the geologic medium and produce dispersion curves, which show the variation of Rayleigh wave phase velocity as a function of frequency (or wavelength).

[1] Park, C, Miller, R., and Xia, J. (1999). Multichannel analysis of surface waves: Geophysics, v. 64, no. 3, pp. 800-808.

After the dispersion curve is generated, the dispersion curve picks are then iteratively fitted to a horizontally layered, laterally continuous, homogeneous-isotropic, S-wave velocity model that would account for the measured surface wave velocity dispersion. The results provide a representative average estimate of the one-dimensional S-wave velocity profile under the array.

Surface Wave Geophysics Procedures

The MASW investigations were performed at the site on December 2, 2019. As shown on Figure 1, the surface wave measurements were performed along three lines, two lines along the sidewalk on the western edge of the property (SW19-1 and SW19-2) and one line on the north side of the property (SW19-3). These measurements were collected using a Geometrics S12 seismograph with 12 channels connected to a linear array of twelve 4.5-Hz geophones.

For the MASW measurements, the active seismic source consisted of a sledgehammer blow to a ground plate. For lines SW19-1 and SW19-2, two separate sets of MASW measurements were collected along each line with geophones linearly spaced at 10-foot and 20-foot intervals. Because of space constraints, the widest spacing that could be achieved for line SW19-3 was 12 feet. Consequently, only one set of MASW measurements was collected along line SW19-3 (using 12-foot spacing). For each measurement set, shots were performed at equal station intervals behind the first geophone with the station intervals equal to the geophone spacing (both 10 and 20 feet for each of lines SW19-1 and SW19-2, and 12 feet for SW19-3). The stations ranged from 60 to 0 feet behind the first geophone for the 12-foot line (six stations) and 20-foot lines (four stations) and 50 to 0 feet behind the first geophone for the 10-foot line (six stations).

At each shot location, the sledgehammer was hit five times, and the resultant waveform was stacked. A 1,024-millisecond long record (0.5 millisecond sample interval) was recorded at each shot location. The recorded MASW data for each shot location were subsequently processed using the program SurfSeis by Kansas Geological Survey. This program performs a wavefield transformation to convert the seismic data from time-distance space to frequency-phase velocity space. The highest amplitude energy in the frequency-phase velocity space was selected for the dispersion curve.

The dispersion curves generated from each shot location were combined to form one dispersion curve for each survey line. A best fit polynomial curve was created from the combined dispersion curve data points for modeling and iteratively fit to a one-dimensional S-wave velocity model using the SurfSeis software. The results provide a one-dimensional vertical profile of S-wave velocity as a function of depth averaged beneath the extent of the line.

Surface Wave Geophysics Results

The results of the MASW surface wave measurements are shown in Figures 2 through 4 for lines SW19-1 through SW19-3, respectively. These figures present the individual MASW data points and best fit surface wave dispersion curves and the corresponding representative S-wave velocity models. The investigation depths modelled were approximately 109 feet (SW19-1), 100 feet (SW19-2), and 72 feet (SW19-3) beneath the survey lines.

The results of the geophysical measurements are summarized in Figure 5. Figure 5 shows the following: (1) the S-wave velocity models for SW19-1 through SW19-3 plotted as a function of depth, (2) the site average S-wave velocity for all the measurements calculated at 1-foot depth increments, and (3) the site average V_{S30} as a function of depth.

Based on the results shown in Figure 5, the V_{s30} was calculated based on the procedures outlined in the National Earthquake Hazards Reduction Program (NEHRP) and UBC. The V_{s30} was calculated from the following equation from these references:

$$v_s = \frac{\sum_{i=1}^{n} di}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where:

i = distinct different soil and/or rock layer between 1 and n
v_{si} = shear wave velocity in feet per second of layer i
d_i = thickness of any layer within the 100-foot interval
$\sum_{i=1}^{n} d_i$ = 100 feet

Based on this procedure, the V_{S30} was calculated below existing ground surface (i.e. between a depth of 0 and 100 feet bgs) for SW19-1, SW19-2, and the site average S-wave velocity. It is noted that the investigation depth of SW19-3 did not extend to 100 feet; therefore, this line was only used to evaluate the site average S-wave velocity. The results are shown on Figure 5 (Site Average V_{S30}) and summarized on Table 1. As summarized on Table 1, the site V_{S30} below existing ground surface falls within NEHRP Site Class C, very dense soil and soft rock sites (1,200 < V_{S30} <2,500 ft/s).

TABLE 1
SITE V_{S30} BELOW EXISTING GROUND SURFACE

ID	V_{s30} (ft/sec)	V_{s30} (m/sec)	NEHRP Site Class
SW19-1	1,204	367	C
SW19-2	1,217	371	C
Site Average S-wave Velocity	1,208	368	C

Limitations

The technical results and professional judgments presented herein are based on limited observations, geophysical measurements (as described above), and our general experience in the field of geophysics. GeoPentech does not guarantee the performance of the project in any respect, only that the information provided meets the standard of care of the profession at this time under the same scope limitations imposed by the project.

We trust the contents of this letter will meet your current needs. If you have questions or require additional information, please call.

Very Truly Yours,

GeoPentech

REGISTERED GEOPHYSICIST
STEVE DUKE
No. 1013
Exp. 7-31-21
STATE OF CALIFORNIA

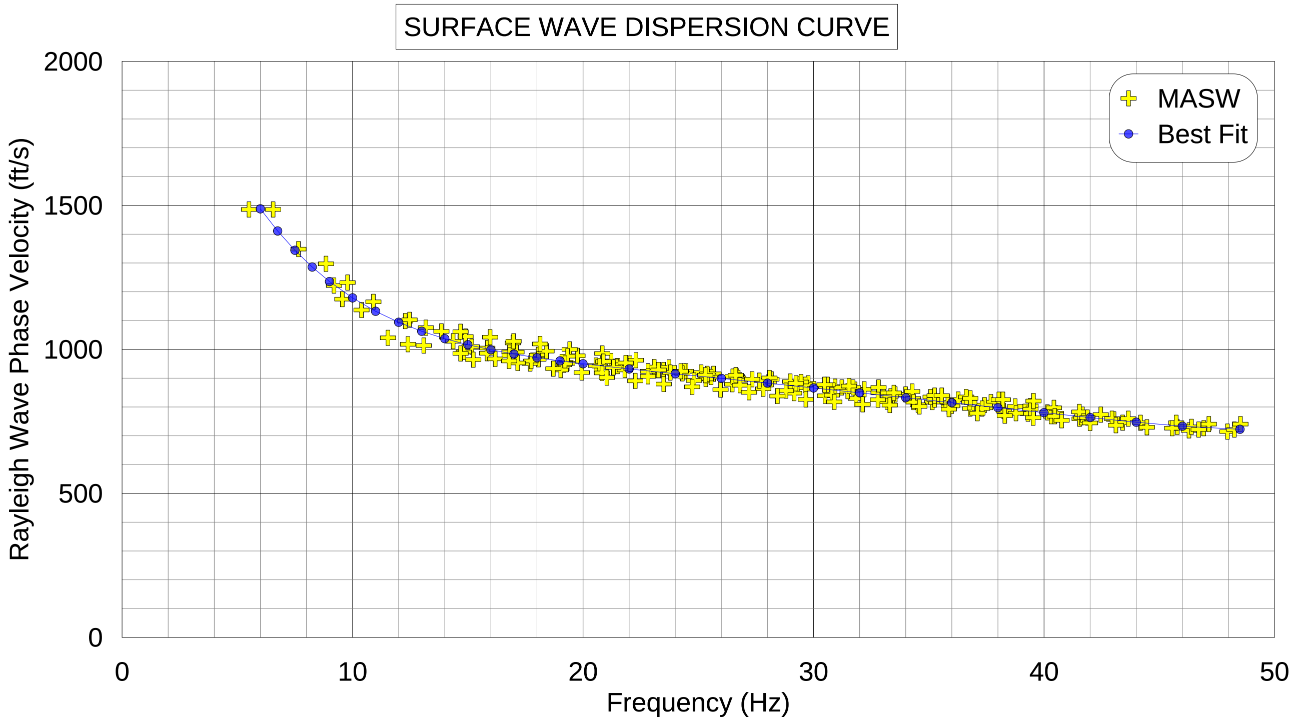
Steven K. Duke
Geophysicist
GP 1013

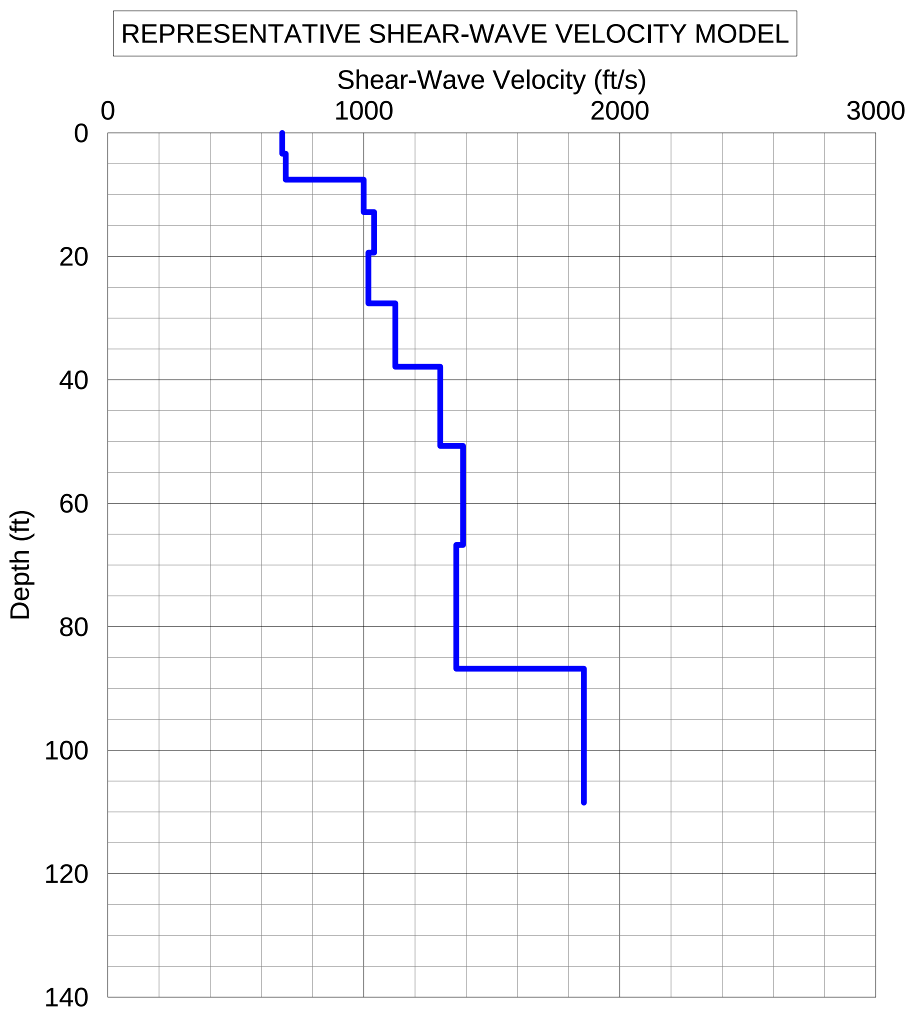
Ryan D. Hort

Ryan D. Hort, Ph.D
Senior Staff Scientist

SITE MAP

GeoPentech	Project No.: **19094A**	Project: **12850 Crenshaw Blvd (Surface Wave)**	Date: **DEC 2019**	Figure: **1**

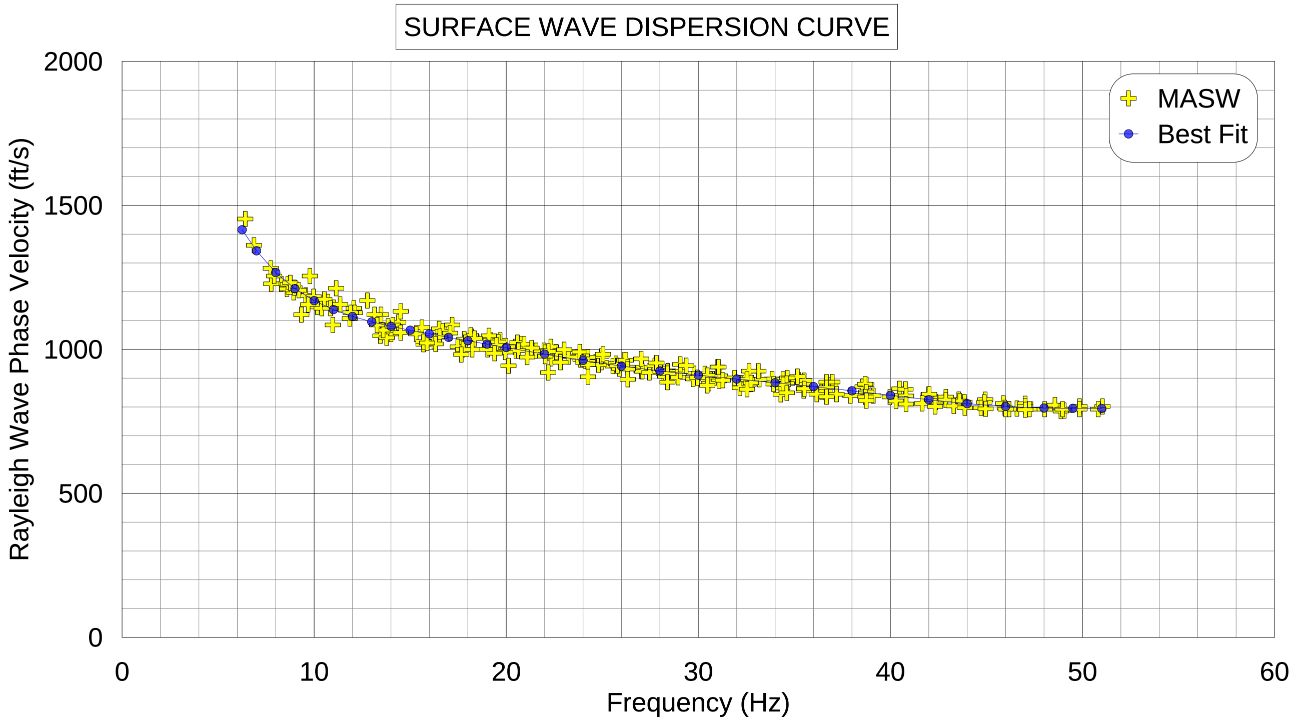


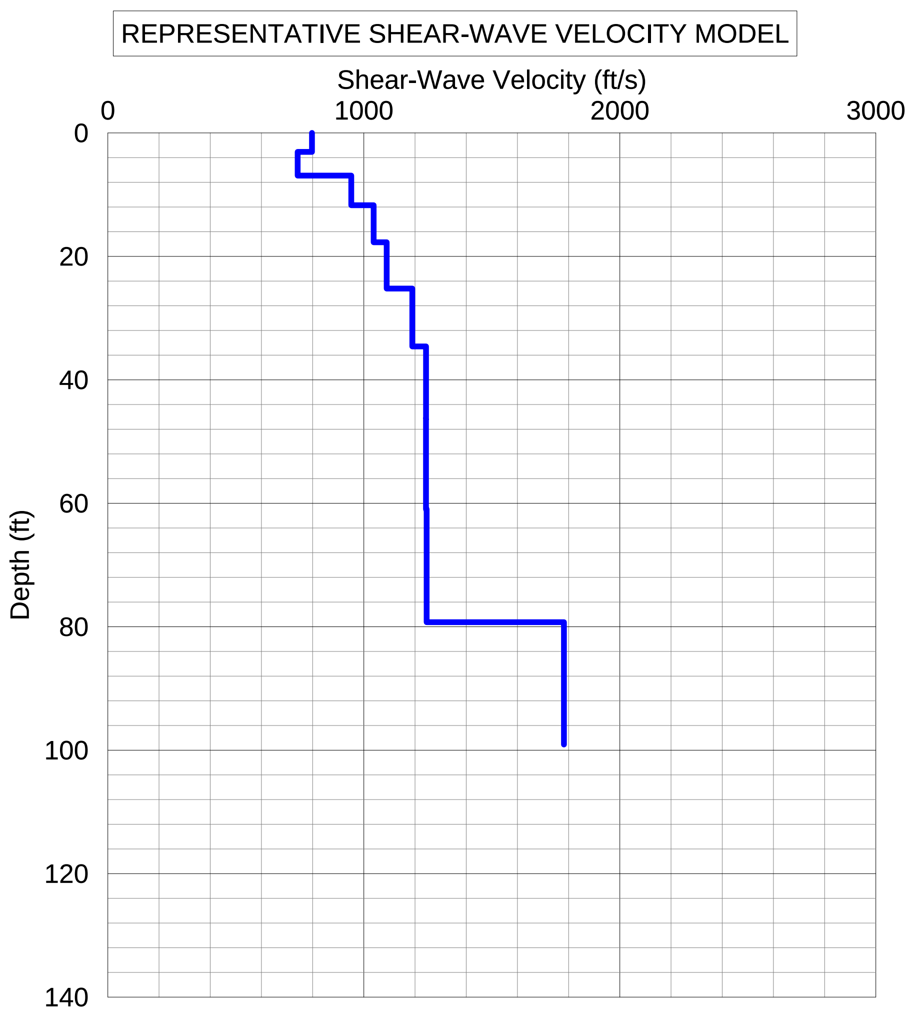


SW19-1: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 19094A	Project: 12850 Crenshaw Blvd. (Surface Wave)	Date: JAN 2019	Figure 2

GeoPentech

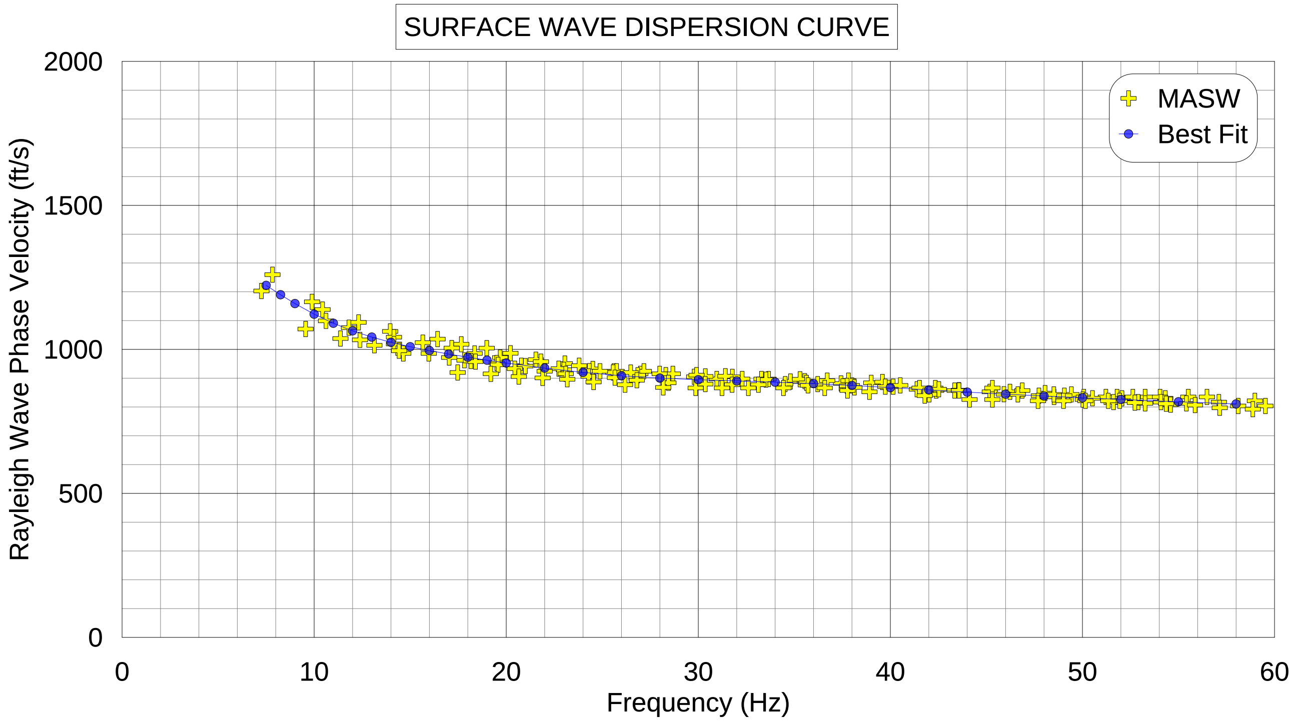


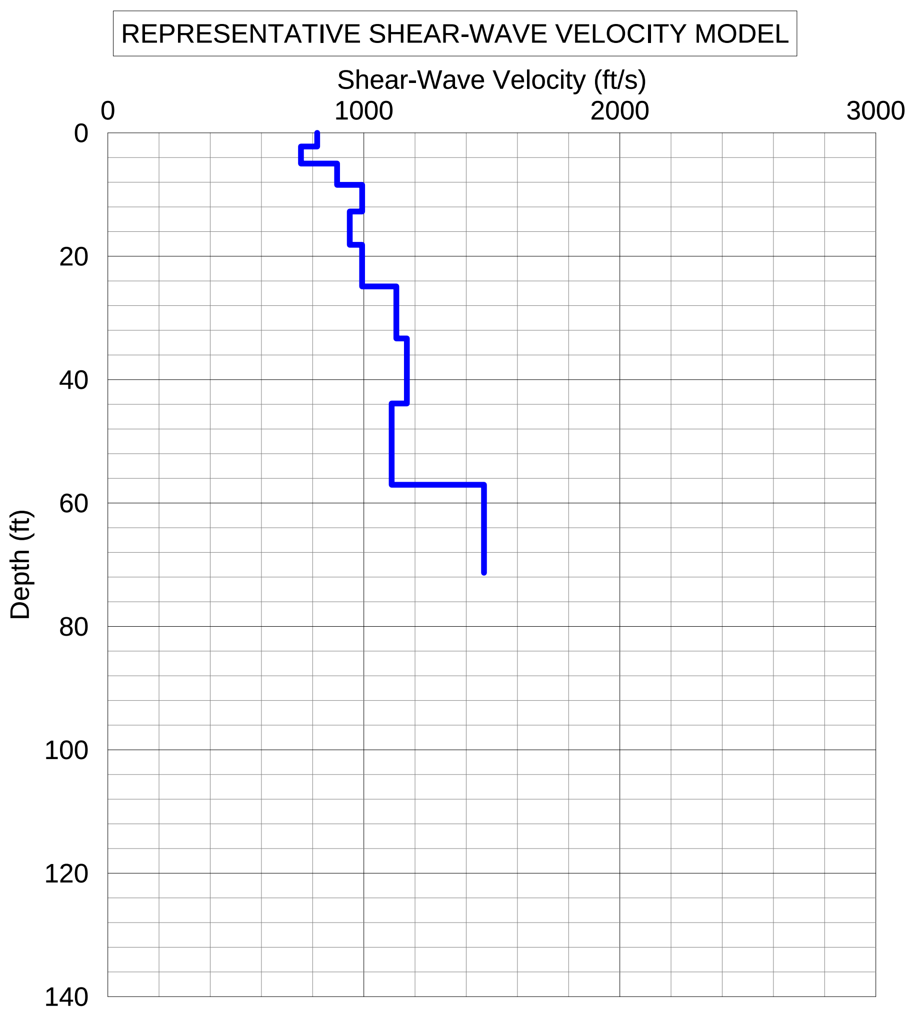


SW19-2: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 19094A	Project: 12850 Crenshaw Blvd. (Surface Wave)	Date: JAN 2019	Figure 3

GeoPentech

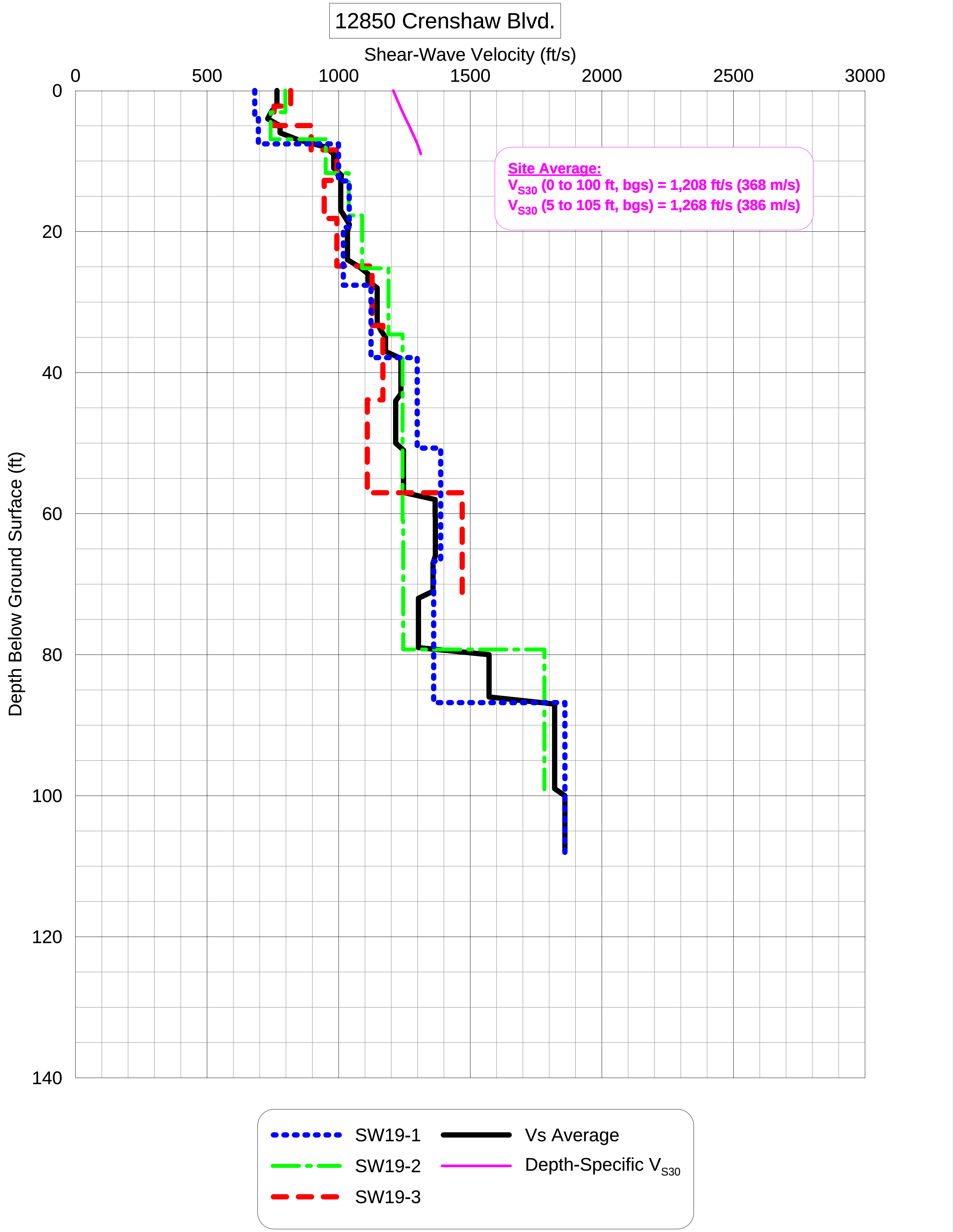




SW19-3: S-WAVE VELOCITY COMBINED SOURCE MODEL

Project No.: 19094A	Project: 12850 Crenshaw Blvd. (Surface Wave)	Date: JAN 2019	Figure 4

GeoPentech



SHEAR WAVE VELOCITY PROFILE SUMMARY

Project No.: 19094A	Project: 12850 Crenshaw Blvd. (Surface Wave)	Date: JAN 2019	Figure 5

GeoPentech

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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

August 20, 2020
File No. 21911

DIN/CAL 4, Inc.
3411 Richmond Avenue, Fifth Floor
Houston, Texas 77046

Attention: Curtis Burnett

Subject: Addendum I – Response to Geotechnical Engineering Investigation Peer Review
Proposed Residential Complex
12850 Crenshaw Boulevard, Gardena, California

References: *Reports by Geotechnologies, Inc.:*
Geotechnical Engineering Investigation, revised May 22, 2020

Technical Memorandum by Kimley Horn:
Geotechnical Engineering Investigation Peer Review for Gardena Transit Oriented Development Specific Plan, dated July 27, 2020

Dear Mr. Burnett:

This letter has been prepared to provide a response to the referenced geotechnical peer review prepared by Kimley Horn. A copy of the review letter is included at the end of this report for reference.

Item 1: Page 2, Second Paragraph and Page 10, Second Paragraph: These comments recommend that Geotechnologies, Inc. expand on the description of the Dominguez Channel retaining wall and provide on opinion on whether the planned development and the existing retaining wall have a potential to adversely impact each other. The Dominguez Channel appears to be comprised of vertical, approximately 15-foot-tall retaining walls and a bottom slab. The westernmost retaining wall is aligned parallel to the planned structure. The vertical and lateral stability of the planned residential structure should remain independent of the behavior and/or performance of the adjacent retaining wall. This can be achieved by establishing a setback distance away from the existing retaining wall. New building and/or fill loads should not be applied within the setback zone unless geotechnical analysis is performed to show that it is safe. To address this potential concern, Geotechnologies should perform an analysis of this condition to determine whether new building and fill loads and planned grading activities will affect or be affected by the existing retaining wall. In other words, if the Dominguez Channel retaining wall fails, will wall and soil movement impact the foundation support of the planned adjacent structure? If so, what actions are required?

Response: Based on review of the ALTA Survey prepared by Fuscoe Engineering, the channel retaining wall immediately east of the project site varies between 14.3 feet to 14.85 feet in height. An approximately 20-foot wide easement (access road) exists between the channel and the east property line. In addition, the edge of the proposed structure will be setback an additional 5.75 feet from the east property line. In total, the edge of building will be setback approximately 25.75 feet from the edge of the existing channel retaining wall. No grading activities or construction will occur beyond the edge of the property or within a 1:1 (h:v) surcharge zone of the existing channel retaining wall. Therefore, it is not anticipated that the proposed grading activities or construction will have any adverse effect on the existing channel.

Item 2: Page 31, Stormwater Disposal: This second issue involves the use of dry wells or infiltration systems. The addition of water at concentrated locations can result in a temporary rise in the groundwater level, which can exceed the historic water levels in the immediate area. Geotechnologies should evaluate this condition and provide an opinion on whether this condition will have an adverse impact on the existing Dominguez Channel retaining wall and the planned building and site improvements.

Response: A copy of the preliminary conceptual LID plan, prepared by Fuscoe Engineering is provided at the end of this report. Based on correspondences with Fuscoe, the intent is to capture the stormwater runoff in two gravel galleries which will be located at the north and south ends of the project site. The gravel galleries will start at an approximate depth of 4 feet and extend to a depth of 9 feet below the existing site grade. The gravel galleries will have an impermeable liner on the sides and bottoms so no water infiltration will occur above the bottom of the galleries. A series of shallow dry wells or long linear gravel trenches will be installed below the gravel galleries to allow the captured stormwater to infiltrate into the underlying soils. The nearest dry wells will be located approximately 40 feet away from the channel. Given the channel is only about 14 to 15 feet deep, and the infiltration system will be designed to infiltrate below a depth of 9 feet at a distance of 40 feet away, it is not anticipated that the proposed infiltration system will have an adverse impact on the existing channel.

The proposed development will be constructed at or near the current site grade. As recommended in the referenced geotechnical report, the proposed infiltration system shall maintain a minimum horizontal setback distance of 15 feet away from any foundation system. Therefore, it is the opinion of this firm that the conceptual LID plans will not have any adverse impact on the proposed development provided the recommendations presented in the referenced geotechnical report are implemented. The conceptual LID plans are acceptable from a geotechnical standpoint.

Should you have any questions please contact this office.
Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
STANLEY S. TAI TANG
No 56178
Exp 12/31/20
CIVIL
STATE OF CALIFORNIA

STANLEY S. TANG
R.C.E. 56178

SST:dy

Enclosures: ALTA Survey
Conceptual LID Plan
Technical Memorandum by Kimley Horn (2 pages)

Distribution: (3) Addressee

Email to: [Curtis.Burnett@tdc-properties.com]

ALTA/NSPS LAND TITLE SURVEY

TITLE INFORMATION:

THE TITLE INFORMATION SHOWN HEREON IS PER PRELIMINARY REPORT NO. 00116945-993-SD2-CFU DATED AUGUST 18, 2019, AS PREPARED BY THE CHICAGO TITLE COMPANY. TITLE OFFICER: KEN CYR AND MARK FRANKLIN (619) 521-3673. NO RESPONSIBILITY OF CONTENT, COMPLETENESS OR ACCURACY OF SAID COMMITMENT IS ASSUMED BY THE SURVEYOR.

OWNERSHIP:

TITLE TO SAID ESTATE OR INTEREST AT THE DATE HEREOF IS VESTED IN:

MAURICE JABBARI MARIAN, A MARRIED MAN, AS HIS SOLE AND SEPARATE PROPERTY.

LEGAL DESCRIPTION:

THE LAND REFERRED TO HEREIN BELOW IS SITUATED IN THE CITY OF GARDENA, IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AND IS DESCRIBED AS FOLLOWS:

LOTS 14, 15, 16 AND 17 OF TRACT NO. 18493, IN THE CITY OF GARDENA, IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 556, PAGE(S) 14 TO 16 INCLUSIVE OF MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

APN: 4060-004-039

TITLE EXCEPTIONS

NO. INDICATES ITEM PLOTTED HEREON

A. AND B. TAXES

1. EASEMENT(S) FOR THE PURPOSE(S) SHOWN BELOW AND RIGHTS INCIDENTAL THERETO, AS GRANTED IN A DOCUMENT: GRANTED TO: PACIFIC TELEPHONE AND TELEGRAPH COMPANY
PURPOSE: UNDERGROUND COMMUNICATION STRUCTURES
RECORDING DATE: JUNE 30, 1955
RECORDING NO: 4526 IN BOOK 48225 PAGE 398, OFFICIAL RECORDS

   AFFECTS: THE EASTERLY 5 FEET OF SAID LAND

2. ANY RIGHTS, INTERESTS OR CLAIMS WHICH MAY EXIST OR ARISE BY REASON OF THE FOLLOWING MATTERS DISCLOSED BY AN INSPECTION OR SURVEY

   A UTILITY EASEMENT IS LOCATED AT THE NORTHWESTERLY PORTION OF SAID LAND.

3. A DEED OF TRUST TO SECURE AN INDEBTEDNESS IN THE AMOUNT SHOWN BELOW, AMOUNT: $900,000.00
DATED: JUNE 27, 2014
TRUSTOR/GRANTOR MAURICE JABBARI MARIAN, A MARRIED MAN AS HIS SOLE AND SEPARATE PROPERTY
TRUSTEE: WILSHIRE BANK
BENEFICIARY: WILSHIRE BANK
LOAN NO.: 637949
RECORDING DATE: JULY 09, 2014
RECORDING NO: 20140706802 OF OFFICIAL RECORDS

4. AN ASSIGNMENT OF ALL MONEYS DUE, OR TO BECOME DUE AS RENTAL OR OTHERWISE FROM SAID LAND, TO SECURE PAYMENT OF AN INDEBTEDNESS, SHOWN BELOW AND UPON THE TERMS AND CONDITIONS THEREIN

   AMOUNT: $900,000.00
   ASSIGNED TO: WILSHIRE BANK
   ASSIGNED BY: MAURICE JABBARI MARIAN, A MARRIED MAN AS HIS SOLE AND SEPARATE
   PROPERTY RECORDING DATE: JULY 09, 2014
   RECORDING NO: 201406803 OF OFFICIAL RECORDS

5. WATER RIGHTS, CLAIMS OR TITLE TO WATER, WHETHER OR NOT DISCLOSED BY THE PUBLIC RECORDS.

6. MATTERS WHICH MAY BE DISCLOSED BY AN INSPECTION AND/OR BY A CORRECT ALTA/NSPS LAND TITLE SURVEY OF SAID LAND THAT IS SATISFACTORY TO THE COMPANY, AND/OR BY INQUIRY OF THE PARTIES IN POSSESSION THEREOF.

7. ANY RIGHTS OF THE PARTIES IN POSSESSION OF A PORTION OF, OR ALL OF, SAID LAND, WHICH RIGHTS ARE NOT DISCLOSED BY THE PUBLIC RECORDS.

   THE COMPANY WILL REQUIRE, FOR REVIEW, A FULL AND COMPLETE COPY OF ANY UNRECORDED AGREEMENT, CONTRACT, LICENSE AND/OR LEASE, TOGETHER WITH ALL SUPPLEMENTS, ASSIGNMENTS AND AMENDMENTS THERETO, BEFORE ISSUING ANY POLICY OF TITLE INSURANCE WITHOUT EXCEPTING THIS ITEM FROM COVERAGE.

   THE COMPANY RESERVES THE RIGHT TO EXCEPT ADDITIONAL ITEMS AND/OR MAKE ADDITIONAL REQUIREMENTS AFTER REVIEWING SAID DOCUMENTS.

ALTA/NSPS TABLE A ITEMS:

ITEM 2	12850 CRENSHAW BLVD GARDENA, CA 90249
ITEM 3	THE LAND SHOWN ON THIS SURVEY LIES WITHIN ZONE "X" (UNSHADED) BEING DESCRIBED AS AREAS DETERMINED TO BE OUTSIDE THE 0.2% ANNUAL CHANCE FLOODPLAIN, PER FLOOD INSURANCE RATE MAP (FIRM) COMMUNITY PANEL NUMBER 06037C1790F, EFFECTIVE DATE: SEPTEMBER 26, 2008.
ITEM 4	THE GROSS LAND AREA IS: 58,033 S.F. ± / 1.33 ACRES ±
ITEM 8	SEE THE SURVEY PLAT FOR ANY SUBSTANTIAL FEATURES OBSERVED IN THE PROCESS OF CONDUCTING THE SURVEY.
ITEM 9	THE SITE DOES NOT HAVE CLEARLY IDENTIFIABLE PARKING SPACES.
ITEM 13	SEE THE SURVEY PLAT FOR THE NAMES OF ADJOINING OWNERS.
ITEM 14	SEE THE SURVEY PLAT FOR THE DISTANCE TO THE NEAREST INTERSECTION.
ITEM 15	THE DATE OF THE RECTIFIED ORTHOPHOTOGRAPH IS OCTOBER 18, 2019. SEE AERIAL PHOTO NOTE HEREON.
ITEM 16	THERE IS NO OBSERVABLE EVIDENCE OF EARTH MOVING WORK OR DEMOLITION ON THE SUBJECT PROPERTY.
ITEM 20	PROFESSIONAL LIABILITY INSURANCE IS IN EFFECT THROUGHOUT THE CONTRACT TERM.

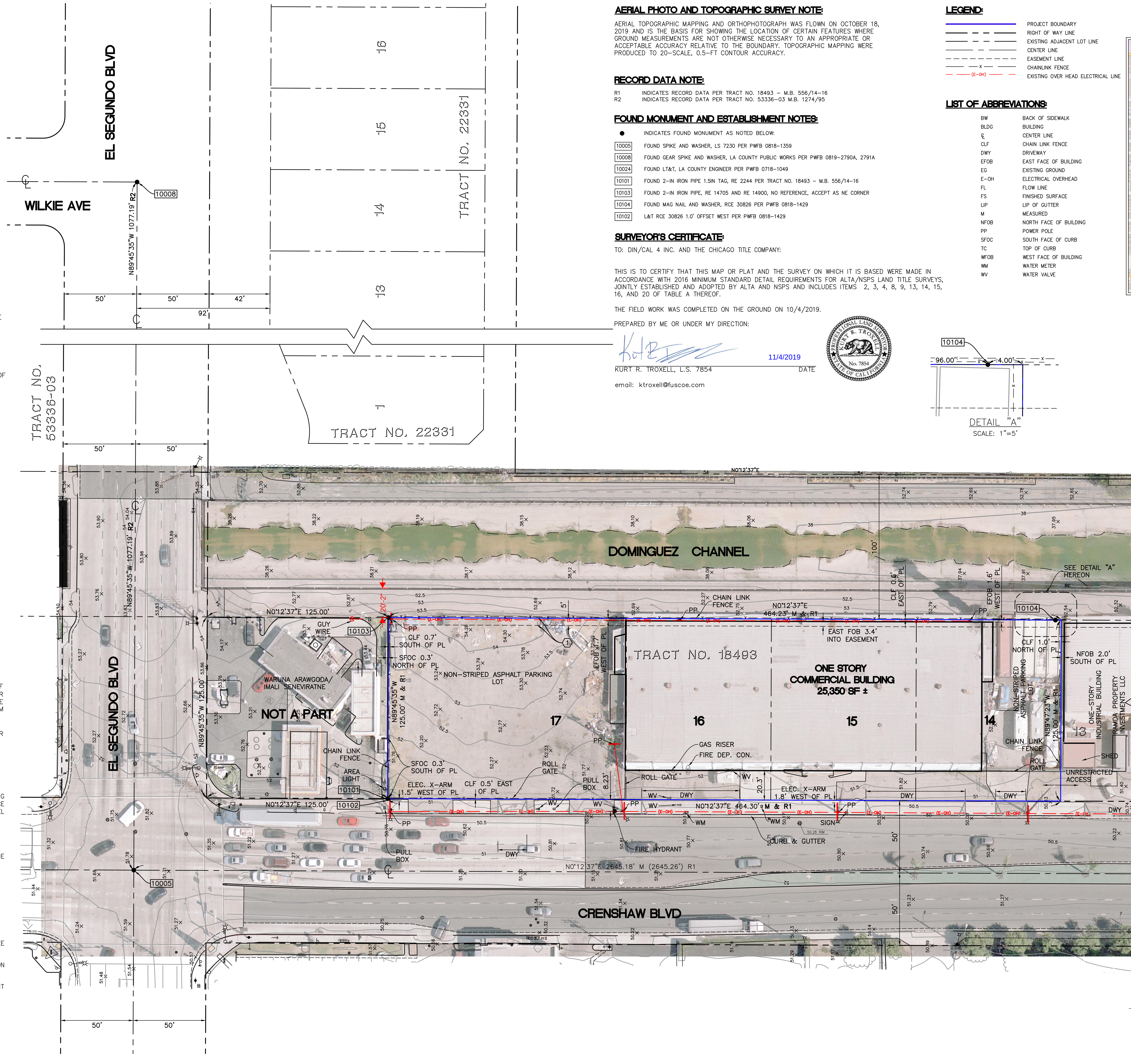
BASIS OF BEARING

THE BASIS OF BEARINGS HEREON ARE BASED ON THE NORTH AMERICAN DATUM OF 1983 (NAD-83), SHOWN IN TERMS OF THE CALIFORNIA COORDINATE SYSTEM OF 1983 (CCS-83), ZONE V (2007.00 EPOCH DATE). BASES LOCATED ON GPS CONTINUALLY OPERATING REFERENCE STATIONS CRHS, CSDH, AND TORP AS PUBLISED BY THE CALIFORNIA SPATIAL REFERENCE CENTER (CSRC).

BENCH MARK

ELEVATIONS HEREON ARE IN TERMS OF THENORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), BASED LOCALLY ON THE LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS PUBLISHED ELEVATION ON THE FOLLOWING BENCH MARK:

RY11749 PUBLISHED ELEVATION = 51.159 FEET (2005 ADJUSTMENT)



AERIAL PHOTO AND TOPOGRAPHIC SURVEY NOTE:

AERIAL TOPOGRAPHIC MAPPING AND ORTHOPHOTOGRAPH WAS FLOWN ON OCTOBER 18, 2019 AND IS THE BASIS FOR SHOWING THE LOCATION OF CERTAIN FEATURES WHERE GROUND MEASUREMENTS ARE NOT OTHERWISE NECESSARY TO AN APPROPRIATE OR ACCEPTABLE ACCURACY RELATIVE TO THE BOUNDARY. TOPOGRAPHIC MAPPING WERE PRODUCED TO 20-SCALE, 0.5-FT CONTOUR ACCURACY.

RECORD DATA NOTE:

R1	INDICATES RECORD DATA PER TRACT NO. 18493 – M.B. 556/14-16
R2	INDICATES RECORD DATA PER TRACT NO. 53336-03 M.B. 1274/95

FOUND MONUMENT AND ESTABLISHMENT NOTES:

●	INDICATES FOUND MONUMENT AS NOTED BELOW:
10005	FOUND SPIKE AND WASHER, LS 7230 PER PWFB 0818-1359
10008	FOUND GEAR SPIKE AND WASHER, LA COUNTY PUBLIC WORKS PER PWFB 0819-2790A, 2791A
10024	FOUND LT&T, LA COUNTY ENGINEER PER PWFB 0718-1049
10101	FOUND 2-IN IRON PIPE 1.5IN TAG, RE 2244 PER TRACT NO. 18493 – M.B. 556/14-16
10103	FOUND 2-IN IRON PIPE, RE 14705 AND RE 14900, NO REFERENCE, ACCEPT AS NE CORNER
10104	FOUND MAG NAIL AND WASHER, RCE 30826 PER PWFB 0818-1429
10102	L&T RCE 30826 1.0' OFFSET WEST PER PWFB 0818-1429

SURVEYOR'S CERTIFICATE:

TO: DIN/CAL 4 INC. AND THE CHICAGO TITLE COMPANY:

THIS IS TO CERTIFY THAT THIS MAP OR PLAT AND THE SURVEY ON WHICH IT IS BASED WERE MADE IN ACCORDANCE WITH 2016 MINIMUM STANDARD DETAIL REQUIREMENTS FOR ALTA/NSPS LAND TITLE SURVEYS, JOINTLY ESTABLISHED AND ADOPTED BY ALTA AND NSPS AND INCLUDES ITEMS 2, 3, 4, 8, 9, 13, 14, 15, 16, AND 20 OF TABLE A THEREOF.

THE FIELD WORK WAS COMPLETED ON THE GROUND ON 10/4/2019.

PREPARED BY ME OR UNDER MY DIRECTION:

11/4/2019

KURT R. TROXELL, L.S. 7854 DATE

email: ktroxell@fuscoe.com

PROFESSIONAL LAND SURVEYOR KURT R. TROXELL No. 7854 STATE OF CALIFORNIA

10104
96.00'
4.00'

DETAIL "A"
SCALE: 1"=5'

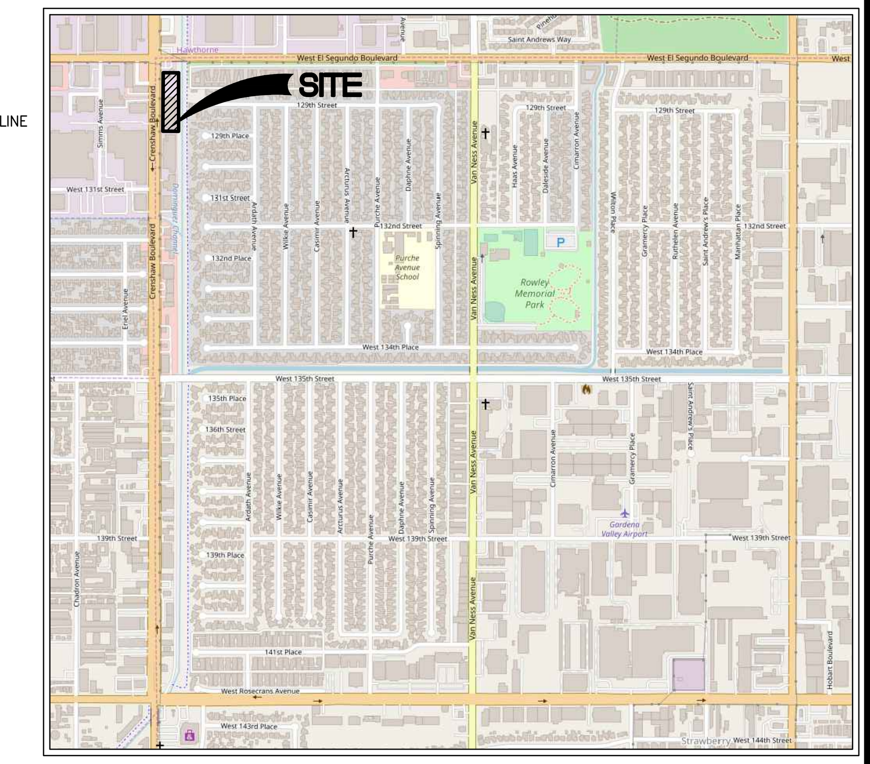
LEGEND:

	PROJECT BOUNDARY
	RIGHT OF WAY LINE
	EXISTING ADJACENT LOT LINE
	CENTER LINE
	EASEMENT LINE
—x—	CHAINLINK FENCE
(E-OH)	EXISTING OVER HEAD ELECTRICAL LINE

LIST OF ABBREVIATIONS:

BW	BACK OF SIDEWALK
BLDG	BUILDING
℄	CENTER LINE
CLF	CHAIN LINK FENCE
DWY	DRIVEWAY
EFOB	EAST FACE OF BUILDING
EG	EXISTING GROUND
E-OH	ELECTRICAL OVERHEAD
FL	FLOW LINE
FS	FINISHED SURFACE
LIP	LIP OF GUTTER
M	MEASURED
NFOB	NORTH FACE OF BUILDING
PP	POWER POLE
SFOC	SOUTH FACE OF CURB
TC	TOP OF CURB
WFOB	WEST FACE OF BUILDING
WM	WATER METER
WV	WATER VALVE

VICINITY MAP
NOT TO SCALE



NO.	DATE	REVISION

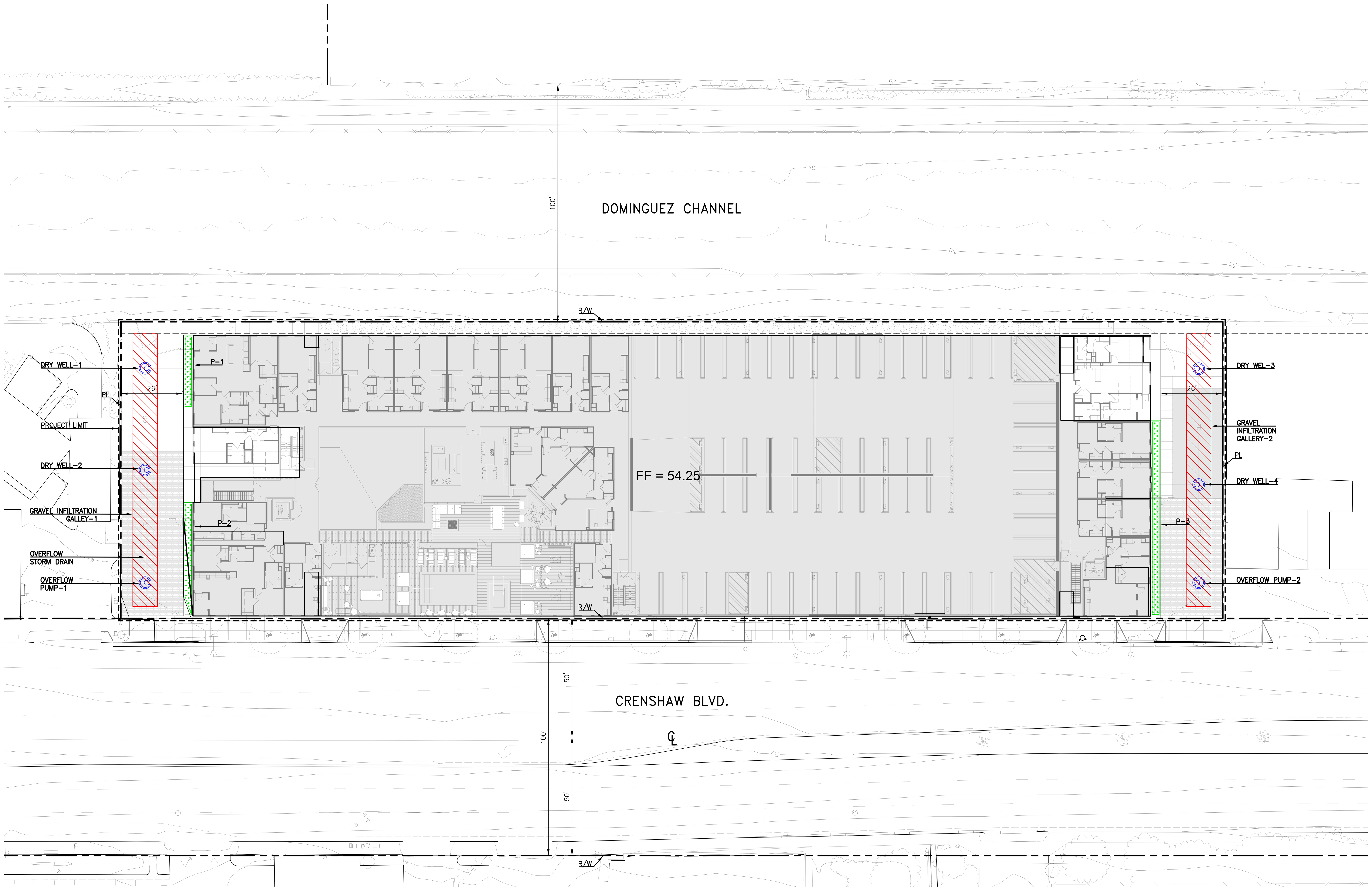
ALTA/NSPS LAND TITLE SURVEY

of: 12850 CRENSHAW BOULEVARD
GARDENA, CALIFORNIA

for: DIN/CAL 4, INC.
AND
THE CHICAGO TITLE COMPANY

FUSCOE ENGINEERING
16795 Von Karman, Suite 100
Irvine, California 92606
tel 949.474.1960 ◦ fax 949.474.5315
www.fuscoe.com

DATE: 11/4/19
SCALE: AS SHOWN
FN: 1724-002
JN: 1724-002
DRAWN BY: MC
CHECKED BY: KRT
SHEET 1 OF 1



Peak Flow Hydrologic Analysis

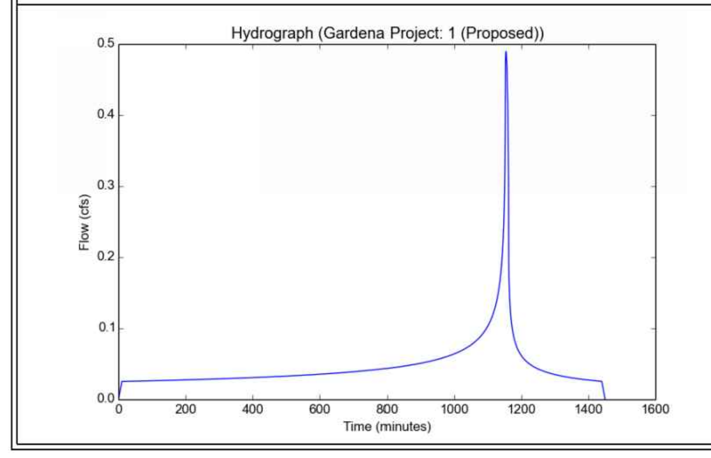
File location: F:/Projects/1724/002/_Support Files/Reports/Hydrology/Proposed Hydro - 85th.pdf
Version: HydroCalc 1.0.2

Input Parameters

Project Name	Gardena Project
Subarea ID	1 (Proposed)
Area (ac)	1.33
Flow Path Length (ft)	150.0
Flow Path Slope (vft/hft)	0.02
85th Percentile Rainfall Depth (in)	0.95
Percent Impervious	1.0
Soil Type	9
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.95
Peak Intensity (in/hr)	0.4092
Undeveloped Runoff Coefficient (Cu)	0.1918
Developed Runoff Coefficient (Cd)	0.9
Time of Concentration (min)	10.0
Clear Peak Flow Rate (cfs)	0.4898
Burned Peak Flow Rate (cfs)	0.4898
24-Hr Clear Runoff Volume (ac-ft)	0.094
24-Hr Clear Runoff Volume (cu-ft)	4093.7452



BMP TREATMENT VOLUMES (CF)		
	GROUND LEVEL/SITE	%
BIOFILTERS	425	10%
INFILTRATION	3,669	90%
TOTAL	4,094	

LEGEND

℄	CENTERLINE
EX	EXISTING
EG	EXISTING GRADE
FF	FINISHED FLOOR
FG	FINISHED GRADE
FL	FLOWLINE
FS	FINISHED SURFACE
PL	PROPERTY LINE
R/W	RIGHT OF WAY
SD	STORM DRAIN
SS	SANITARY SEWER
TC	TOP OF CURB
W	WATER
	PROJECT LIMIT
	RIGHT OF WAY / PL
	CENTERLINE
X	FENCE (PER LANDSCAPE PLANS)
	PROPOSED BUILDING
	BIOFILTER
	STORAGE TANK
	DRY WELL

NOTE: THE BMP TREATMENT PLAN INCORPORATES BOTH BIOFILTRATION AND INFILTRATION BMPs. ALTHOUGH THE PERCOLATION RATE IS BELOW THE DESIRED VALUE PER LA COUNTY (0.03 IN/HR VS 0.3 IN/HR), THERE ARE MINIMAL OPPORTUNITIES FOR BIOFILTRATION ON SITE. INFILTRATION WILL BE USED TO OFFSET THE REMAINING TREATMENT VOLUMES THAT CAN NOT BE ACCOMMODATED BY BIOFILTRATION PLANTERS IN ORDER FOR THE PROJECT TO MEET TOTAL STORMWATER VOLUME TREATMENT REQUIREMENTS

20' 0' 10' 20' 40'
SCALE: 1" = 20'

LID TREATMENT EXHIBIT

08/17/2020

FUSCOE
ENGINEERING
600 ▯ils▯ire ▯lvd., Suite 1470
▯os ▯n▯eles, California 90017
tel 213.9▯▯.▯▯02 ◦ fax 213.9▯▯.▯▯03
www.fuscoe.com

Kimley»Horn

TECHNICAL MEMORANDUM

To:	Ray Barragan and Lisa Kranitz, City of Gardena
From:	Rita Garcia, Kimley-Horn, and Dean Iwasa, Haley & Aldrich
Date:	July 27, 2020
Subject:	**Gardena Transit Oriented Development Specific Plan, 12850 and 12900 Crenshaw Boulevard, Geotechnical Engineering Investigation Peer Review**

On behalf of Kimley-Horn, Haley & Aldrich has conducted peer review of the *Geotechnical Engineering Investigation Proposed Residential Complex 12850 Crenshaw Boulevard Gardena, California (*Geotechnologies, Inc., January 2020). Specific comments are embedded in the attached copy of the report. Please note that the Project Description has been updated by the applicant and any appropriate changes should be made to the Geotechnical report.

Two of the more substantive comments are discussed in more detail, as follows.

1. Page 2, Second Paragraph and Page 10, Second Paragraph: These comments recommend that Geotechnologies, Inc. expand on the description of the Dominguez Channel retaining wall and provide on opinion on whether the planned development and the existing retaining wall have a potential to adversely impact each other. The Dominguez Channel appears to be comprised of vertical, approximately 15-foot-tall retaining walls and a bottom slab. The westernmost retaining wall is aligned parallel to the planned structure. The vertical and lateral stability of the planned residential structure should remain independent of the behavior and/or performance of the adjacent retaining wall. This can be achieved by establishing a setback distance away from the existing retaining wall. New building and/or fill loads should not be applied within the setback zone unless geotechnical analysis is performed to show that it is safe. To address this potential concern, Geotechnologies should perform an analysis of this condition to determine whether new building and fill loads and planned grading activities will affect or be affected by the existing retaining wall. In other words, if the Dominguez Channel retaining wall fails, will wall and soil movement impact the foundation support of the planned adjacent structure? If so, what actions are required?

2. Page 31, Stormwater Disposal: This second issue involves the use of dry wells or infiltration systems. The addition of water at concentrated locations can result in a temporary rise in the groundwater level, which can exceed the historic water levels in

the immediate area. Geotechnologies should evaluate this condition and provide an opinion on whether this condition will have an adverse impact on the existing Dominguez Channel retaining wall and the planned building and site improvements.

Please do not hesitate to contact Dean Iwasa at 925.949.1021 of diwasa@haleyaldrich.com with any questions.

Kimley»Horn

TECHNICAL MEMORANDUM

To: Ray Barragan and Lisa Kranitz, City of Gardena

From: Rita Garcia, Kimley-Horn, and Dean Iwasa, Haley & Aldrich

Date: January 14, 2021

Subject: **Gardena Transit Oriented Development Specific Plan, 12850 and 12900 Crenshaw Boulevard, Geotechnical Engineering Investigation Peer Review**

Kimley-Horn has conducted a follow-up third-party peer review of the Project's Geotechnical Engineering Investigation Addendum (Geotechnologies, Inc., August 2020) on behalf of the City of Gardena to verify that Kimley-Horn's July 27, 2020 third-party peer review Technical Memo (TM) recommendations have been incorporated. The August 2020 addendum addressed the third-party peer review comments and thus is in compliance with the TM recommendations. The analysis, as revised, meets the applicable provisions of CEQA and the State CEQA Guidelines and is adequate for inclusion in the Project EIR.

Please do not hesitate to contact Dean Iwasa at 925.949.1021 or diwasa@haleyaldrich.com with any questions.