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March 5, 2025
File Number 22600

Torrance Center South Complex, LLC
8540 S. Sepulveda Boulevard, #706
Los Angeles, California 90045

Attention: Ms. Min Gong

Subject: Preliminary Geotechnical Engineering Investigation
Proposed Residential Complex
21151, 21171, 21221, 21241, and 21281 South Western Avenue
Torrance, California

Dear Ms. Gong:

This letter transmits the Preliminary Geotechnical Engineering Investigation for the Project Site prepared by Geotechnologies, Inc. This report provides preliminary geotechnical recommendations for the development of the Project Site, including earthwork, seismic design, retaining walls, temporary excavations, shoring, and foundation design.

This report is preliminary in nature due to the conceptual nature of the Project and the limited explorations performed as part of the current scope. This report is therefore not intended for submission for building permit purposes. A comprehensive geotechnical report for design development should be prepared when the proposed development plan achieves more definition and additional explorations are performed for a thorough investigation of the Project Site.

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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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL COMPLEX
21151, 21171, 21221, 21241, AND 21281 SOUTH WESTERN AVENUE
TORRANCE, CALIFORNIA

INTRODUCTION

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

This report is preliminary in nature due to the conceptual nature of the Project and the limited explorations performed as part of the current scope. This report is therefore not intended for submission for building permit purposes. A comprehensive geotechnical report for design development should be prepared when the proposed development plan achieves more definition and additional explorations are performed for a thorough investigation of the Project Site.

This limited investigation included excavation of five exploratory borings, 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 boring locations are shown on the enclosed Site Plan and Plot Plan. The results of the exploration and the laboratory testing are presented in the Enclosures of this report.

PROPOSED DEVELOPMENT

Preliminary design information concerning the proposed development was furnished by the client. A conceptual design package by MVE+Partners, dated November 20, 2024, was reviewed for preparation of this report. The proposed project will be constructed in three phases. Phase 1 will consist of construction of a 7-story condominium structure over one subterranean parking level at

the southeastern portion of the Project Site, and 4-story at-grade townhouses at the northern portion of the site. Phases 2 and 3 will consist of the construction of 4-story at-grade townhouses at the southern and southwestern portions of the Project Site. The locations of proposed structures relative to nearby topographic features are shown in the enclosed Plot Plan.

Preliminary, column loads are estimated to be between 300 and 800 kips. Wall loads are estimated to be between 3 and 8 kips per lineal foot. Grading for the proposed townhouses will consist of removal and recompaction of existing unsuitable soils. It is anticipated that the construction of the proposed condominium structure will require excavations up to 15 feet below the existing site grade for the proposed subterranean level and foundation elements.

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 Project Site is located at 21151, 21171, 21221, 21241, and 21281 South Western Avenue, in the City of Torrance, California. The site is bounded by Torrance Boulevard to the north, S. Western Avenue to the east, 213th street to the south, and Bow Avenue to the west. The Project Site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The Project Site consists of an irregularly shaped lot, which is currently occupied by a business center developed with five, 2-story, at-grade office buildings and associated parking lots. Based on review of the Architectural Survey Plan, prepared by Mollenhauer Group (dated August 21, 2024), multiple utility and stormdrain easements traverse across the Project Site. The locations of existing structures and easements are shown in the enclosed Site Plan.

The site slopes downward very gently towards the north. The available Architectural Survey Plan, prepared by Mollenhauer Group, provides topographic details only for approximately ¾ of the Project Site. The elevations in the southeast portion of the site are not shown on the Survey Plan. The elevations within the remaining portion of site range from 58½ feet above Mean Sea Level (AMSL) along the southern site boundary to 38½ feet along the northern site boundary, corresponding to approximate site elevation difference of 20 feet.

Drainage across the Project Site is by the sheetflow to local area drains and the city streets. The vegetation of the site consists primarily of trees, shrubs, and grass areas located in isolated planters. The neighboring development consists primarily of low-rise to mid-rise residential and commercial structures.

RESEARCH

As a part of this preliminary geotechnical investigation, this firm requested any available records of past geotechnical reports performed for the Project site from the City of Torrance Department of Building and Safety. Based on research performed by the City staff, a geotechnical report prepared for a relocation and replacement of a cooling tower, prepared by MACTEC, was the only record available. Their findings and recommendations were summarized in the following report:

- Report of Geotechnical Investigation Proposed Relocate/Replace Cooling Tower, Existing SBC Facility, 21281 Wester Avenue, Torrance, California, Project 4953-05-2021, dated August 4, 2005.

The MACTEC report included one exploratory boring, which was drilled to the depth of 30 feet using a 16-inch-diameter bucket-type drilling equipment. According to the report, the proposed cooling tower was to be constructed in the parking lot north of the 21281 Western Avenue building. However, the Plot Plan showing the colling tower location and the boring was missing from the record report.

Based on the MACTEC report, two feet of existing fill consisting of clayey sands was encountered in their boring. Native soils underlying the fill consisted primarily of silty clay and silty sand, which were light grayish brown in color, slightly moist to moist, stiff, dense, fine grained. No groundwater or caving were encountered in the MACTEC boring which was excavated to a maximum depth of 30 feet. A sample between 2 and 3 feet from the MACTEC boring was found to be in the high expansion range with an Expansion Index of 119. A copy of the MACTEC report is enclosed at the end of this report for reference.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The Project Site was explored on January 2, 2025, and January 3, 2025, by drilling five exploratory borings. The exploratory borings varied in depth from 20 to 60 feet below the existing grade and were excavated with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. Samples were taken at regular intervals using an automatic hammer, consisting of a 140-pound weight dropped from a height of 30 inches. The exploratory boring locations are shown on the enclosed Site Plan and Plot Plan, and the geologic materials encountered are logged on Plates A-1 through A-5.

The location of exploratory borings was determined by information furnished by the client. The location of the exploratory borings should be considered accurate only to the degree implied by the method used. Elevations of the exploratory borings were determined from the Architectural Survey Plan by Mollenhauer Group dated August 21, 2024.

Geologic Materials

Fill materials were encountered in all exploratory borings to depths ranging from 3 to 14 feet below the existing site grade. Fill materials consist of silty clay, clayey silt, silty sand and sand, which are dark and grayish brown in color, moist, stiff to very stiff, medium dense to very dense, fine grained, with cobbles and minor asphalt fragments.

Native soil was encountered below the existing fill materials. Native soil consists of clayey and sandy silt, silty clay, and silty sand, which are dark to yellowish and grayish brown in color, moist to wet, stiff to very stiff, medium dense to very dense, fine grained. More detailed descriptions of the earth materials encountered may be obtained from individual boring logs.

Groundwater

Groundwater was encountered in exploratory boring B1 at a depth of 55 feet below existing site grade during exploration. Based on review of the Historically Highest Groundwater Contours Map, presented in the California Geological Survey Seismic Hazard Zone Report of the Torrance Quadrangle (SHZR 035), the historically highest groundwater level surrounding the Project Site is not well defined. The nearest groundwater contour line (approximately 2½ miles to the east of the Project Site) is shown to be 20 feet below the existing site grade.

The Historically Highest Groundwater Contour Map was established based on prior borehole data collected by CGS. These borehole data are available through the CGS Borehole Database website (https://maps.conservation.ca.gov/cgs/informationwarehouse/bhdb/). The boreholes surrounding the Project Site were excavated by various institutions and agencies, between 1954 and 1995. Based on research, these surrounding boreholes were drilled to depths between 40½ and 91 feet below the ground surface. Majority of these boreholes did not encounter any groundwater, except for two borings located east of the 110 Freeway. Groundwater was encountered at a depth of 69 feet below the ground surface in Well Name I6264MW1 (which is approximately 1.4 miles to the southeast of the Project Site. Groundwater was also encountered at a depth of 72½ feet below the ground surface in Well Name I2985B15MW7 (which is approximately 2 miles to the southeast of the Project Site. The borehole data from the nearby borings are presented in the enclosed CGS Borehole Database Map.

Based on the above findings, it is the opinion of this firm that the historically highest groundwater level is on the order of 50 feet below the existing ground surface. Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

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

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The Project Site 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.

REGIONAL FAULTING

Based on criteria established by the California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

These Holocene-active faults represent most of the earthquake hazard. Faults that have ruptured to the ground surface during the Holocene epoch are considered "active" and are included in Alquist-Priolo Earthquake Fault Zones. There are several other definitions of activity, used for other purposes, such as the siting and design of large dams, power plants and other facilities. Another commonly used designation is for faults that have offset geologic units from the Quaternary period (the last 1,800,000 years). Faults that offset Quaternary units are sometimes called "potentially active".

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. As revised in 2018, The Act defines "Holocene-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 11,700 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 Holocene-Active 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, no known Holocene-active faults underlie the Project Site. In addition, the Project 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 Earthquake Zones of Required Investigation Map (CGS, 2016) 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.

As a part of this geotechnical engineering investigation, a site-specific liquefaction analysis was performed following Guidelines for Evaluating and Mitigating Seismic Hazard in California Special Publication 117A (SP 117A, 2008), published by CGS. In addition, recommendations provided in EERI Monograph (MNO-12), by Idriss and Boulanger (2008) were also incorporated into the analysis.

Groundwater was encountered in exploratory boring B1 at a depth of 55 feet below existing site grade during site exploration. As discussed in the Groundwater section of this report, it is the opinion of this firm that the historically highest groundwater level for the Project Site is deeper than 50 feet. However, for the sake of conservatism, the enclosed liquefaction analysis is performed using a historic high groundwater level of 20 feet below the existing site grade, as this is the closest groundwater contour line (which is approximately 2½ miles to the east of the Project Site), based on the California Geological Survey Seismic Hazard Zone Report of the Torrance 7½-Minute Quadrangle (CDMG, revised 2006).

The ASCE Hazard Tool (https://ascehazardtool.online/) yields a peak ground acceleration (PGA_M) of 0.84g for the Project Site. The USGS Probabilistic Seismic Hazard Analysis Deaggregation program (USGS, 2014) was utilized and resulted a modal magnitude (M_W) of the 7.3 for the Project Site. These ground motion parameters were utilized for the enclosed liquefaction analysis.

The enclosed Liquefaction Potential Analysis is based on the Standard Penetration Test (SPT) blowcounts obtained from exploratory boring B1. The SPT blowcounts were recorded at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percentage passing a Number 200 sieve, Atterberg Limits, and the Plasticity Index (PI) of representative samples of the soils are presented on the enclosed Plates E and F.

Based on SP 117A, 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 for the analysis of the liquefaction potential of the Project 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 SP117A, 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 the collected field SPT data and laboratory testing of soils, the enclosed site-specific liquefaction analysis indicates that the potential for liquefaction at the site is considered to be remote.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches, and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately up gradient from the project site. Therefore, the risk of flooding from a seismically-induced seiche is considered to be remote.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within mapped inundation boundaries due to a seiche or a breached upgradient reservoir.

Landsliding

The probability of seismically-induced landslides occurring on the Project Site is considered to be low due to the general lack of elevation difference slope geometry across or adjacent to the Project Site. The County of Los Angeles Landslide Inventory Map (Leighton, 1990) indicates that the Project Site is not located in an area highly susceptible to landslides. Additionally, the CGS Earthquake Zones of Required Investigation of the Torrance Quadrangle (CGS, 2016) indicates the Project Site is not located within a Seismically Induced Landslide Zone. Based on these considerations, the potential of landslides negatively affecting the proposed development is considered to be low.

CONCLUSIONS AND RECOMMENDATIONS

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

This report is preliminary in nature due to the conceptual nature of the Project and the limited explorations performed as part of the current scope. This report is therefore not intended for submission for building permit purposes. A comprehensive geotechnical report for design development should be prepared when the proposed development plan achieves more definition and additional explorations are performed for a thorough investigation of the Project Site.

Approximately 3 to 14 feet of fill materials were encountered in the exploratory borings. The existing fill materials are not suitable for support of the proposed foundations. All existing fill materials and any fill generated during demolition should be removed and recompacted as controlled fill prior to foundation excavation or be wasted from the site.

The proposed 7-story condominium residential structure over 1 subterranean parking garage will be constructed in the southeastern portion of the Project Site. Preliminarily, it is anticipated that the construction of the condominium structure will require excavations up to 15 feet below the existing site grade for the proposed subterranean level and foundation elements. The excavation is anticipated to remove the existing fill materials and expose the underlying native soils. The proposed structure may be supported on conventional foundations bearing in the underlying native soils anticipated at the bottom of the excavation.

The proposed 4-story, at-grade, townhouses may be supported on conventional foundations bearing in engineered fill pads. All existing fill materials in the area of proposed townhouses shall be properly removed and recompacted for support of proposed foundations and slabs-on-grade. The compacted fill pads for the proposed townhouses shall extend to a minimum depth of 5 feet below the existing site grade, or 3 feet below proposed foundations, whichever is greater. 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 foundations, whichever is greater.

Localized areas of moderate to high expansive soils may be encountered during grading. It is recommended that the onsite soils shall be well blended to achieve low expansion potential (with Expansion Index of less than 50) prior to placement as compacted fill.

Foundations of small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which will not be tied-in to the proposed residential structures may be supported on conventional foundations bearing in the properly compacted fill and/or the underlying native soils.

It should be noted that multiple utility and stormdrain easements traverse across the Project Site. The locations of existing structures and easements are shown in the enclosed Site Plan. It is recommended the depths and the locations of all utility lines be ascertained prior to grading and construction. Extreme care should be taken during the grading process to avoid damage existing underground utilities. It is recommended that the existing utility and stormdrains be relocated to areas outside of the proposed structures. If the existing utilities and stormdrains are to remain sm be protected in place, then it will be necessary to structurally bridge over the existing easements. Additional evaluations and recommendations will be performed as the Project achieves more definition.

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 borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings, or which may result from changes in subsurface conditions. Once the project design achieves more definition, the proposed development plans shall be provided to this firm for review. Any changes in the design, as outlined in this report, shall be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

2022 California Building Code Seismic Parameters

Preliminary, the Project Site may be classified as Site Class D ("Stiff Soil" Profile), in accordance with Table 20.3-1 of ASCE 7-16 Standard. This information and the site coordinates were input into the ASCE Hazard Tool (https://ascehazardtool.org/) in order to calculate ground motion parameters for the site for preliminary design purposes.

2022 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2022
ASCE Design Standard	7-16
Risk Category	II
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.754g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.754g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.169g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.631g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.073g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.715g*

** These values are determined based on the tabulated values shown on ASCE 7-16, Table 11.4-2 for the Long-Period Site Coefficient, F_v. It should be noted that the exception to performing a site-specific hazard analysis for structures on Site Class D for values of S_1 greater than or equal to 0.2 presented by ASCE 7-16 may be followed, provided the conditions outlined in the ASCE 7-16 Section 11.4.8 (including Supplement 3 of the ASCE 7 Standards) are implemented in the structural design and analyses. This exception recommends that, where a site-specific ground motion hazard analysis is not required, the S_{M1} determined by eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this standard. This would require that the S_{M1} and S_{D1} values provided in the above table are increased by 50%.*

FILL SOILS

The maximum depth of fill encountered in the field explorations was 14 feet. All existing fill materials and any fill generated during demolition should be removed and recompacted as controlled fill for support of the proposed at-grade structures, or be removed during excavation of the subterranean structure.

EXPANSIVE SOILS

The onsite soils were found to be in a very low to moderate expansion range with expansion index ranging from 3 to 58 for bulk samples remolded to 90 percent of the laboratory maximum density. It should be noted that, based on record documents from City research, a sample between 2 and 3 feet from the MACTEC boring was found to be in the high expansion range with an Expansion Index of 119. Recommended reinforcing is noted in the "Foundation Design" and "Slabs-On-Grade" sections of this report.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomena in which the underlying soils collapse when wetted. Hydroconsolidation could potentially result in significant foundation movements, over a long period of time of wetting.

The underlying native soils are very dense and stiff. Soil samples collected from the underlying native soils are subject to a very minor degree of hydroconsolidation strains, between 0 and approximately 0.2 percent. The property owner shall maintain proper drainage of the subject site throughout the life of the structure. All utility and irrigation lines, and drainage devices should be checked periodically and maintained. In addition, landscape irrigation should be properly controlled, in order to reduce the amount of water infiltration into the underlying soils, which provide support to the proposed structure. The Site Drainage section below should be followed and implemented into the final construction documents.

SOIL CORROSION POTENTIAL

The results of soil corrosion potential testing performed by HDR, Inc. indicate that the electrical resistivities of the soils were in the mildly corrosive and moderately corrosive categories with as-received moisture. When saturated, the resistivities were in the moderately corrosive to corrosive categories. The soil pH values of the sample were found to be between 7.7 and 8.5, which corresponds to mildly to strongly alkaline conditions. The soluble salt content ranged from low to moderate. Ammonium was not detected. Nitrate was detected in low concentrations.

In summary, the soils are classified as corrosive to ferrous metals, aggressive to aluminum, and classified as negligible (S0) for sulfate attack on concrete. Detailed results, discussion of results and recommended mitigating measures are provided within the enclosed Soil Corrosivity Study prepared by HDR, Inc., dated February 3, 2025. Geotechnologies, Inc. does not practice in the field of corrosion engineering and mitigation. Any questions regarding the results of the soil corrosion report should be addressed to the corrosion engineer.

GRADING GUIDELINES

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. It is recommended that any existing or abandoned utilities or structures located within the footprint of the proposed grading area be removed or relocated where possible.

- All vegetation, associated root system, 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.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to moisture content within 3 percent of optimum moisture content, and recompacted to at least 90 percent of the maximum dry density.

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

Recommended Overexcavation

All existing fill materials and any fill generated during demolition should be removed and recompacted as controlled fill for support of the proposed at-grade structures. Localized areas of moderate to high expansive soils may be encountered during grading. It is recommended that the onsite soils shall be well blended to achieve low expansion potential (with Expansion Index of less than 50) prior to placement as compacted fill.

The compacted fill pads shall extend to a minimum depth of 5 feet below the existing site grade, or 3 feet below the proposed foundations, whichever is greater. In addition, the compacted fill pad shall extend horizontally a minimum of 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below foundations, whichever is greater.

Subsequent to the recommended removal, the exposed soils should be scarified to the depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required relative compaction of 90 percent of the laboratory maximum dry density in accordance with the most recent revision of ASTM D 1557. It is very important that the position of the proposed structures is accurately located so that the limits of the graded area are accurate, and the grading operation proceeds efficiently.

Compaction

All fill materials should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditioned to within 3 percent of the optimum moisture content of the particular material placed. All fill materials shall be compacted to at least 90 percent of the maximum laboratory dry 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 should not be used for 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 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 optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of the excavated plane may require significant drying and aeration prior to recompaction.

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

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

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and

recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

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

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

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within 3 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.

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 engineering purposes. Please advise this office at least twenty-four hours prior to any required site visit.

FOUNDATION DESIGN

Conventional Foundations (7-story Condominium with Subterranean Level)

The proposed 7-story residential structure will be constructed over 1 subterranean parking level, extending on the order of 15 feet below existing site grade. It is anticipated that excavation of the proposed subterranean level will remove the existing fill materials and expose the underlying native soils. The proposed structure may be supported on conventional foundations bearing in the underlying native soils anticipated at the bottom of the excavation.

Continuous foundations may be designed for a bearing capacity of 3,000 pounds per square foot and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the native soils.

Column foundations may be designed for a bearing capacity of 3,500 pounds per square foot and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the native soils.

The bearing capacity increase for each additional foot of width is 300 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 7,000 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.

Since the recommended bearing value 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.

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 (7-story Condominium with subterranean level)

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 0.35 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 250 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Conventional Foundations (At-Grade Townhouses)

Subsequent to the recommended grading, the proposed 4-story, at-grade, townhouses may be supported on conventional foundations bearing in engineered fill pads.

Continuous foundations may be designed for a bearing capacity of 2,000 pounds per square foot and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended compacted fill.

Column foundations may be designed for a bearing capacity of 2,500 pounds per square foot and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended compacted fill.

The bearing capacity increase for each additional foot of width is 150 pounds per square foot. The bearing capacity increase for each additional foot of depth is 400 pounds per square foot. The maximum recommended bearing capacity is 4,000 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.

Since the recommended bearing value 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.

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 (At-Grade Townhouses)

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 250 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components

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

Conventional Foundation Settlement

Settlement of the foundation systems is expected to occur on initial application of loading. The maximum settlement is expected to be on the order of 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ½ inch.

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed parking structure may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Continuous footings may be designed for a bearing capacity of 2,000 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 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

It is anticipated that subterranean walls up to 15 feet in height will be required as part of proposed development. Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure (pcf)
15 feet	42 pcf	62½ pcf

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. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

The upper ten feet of the retaining wall adjacent to streets, 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. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 24 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of basement walls under seismic loading conditions. For retaining walls less than 6 feet in height, 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 the 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 4-inch diameter perforated pipes, placed with perforations facing down. The pipe shall be encased in at least one 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 dry 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 shoring system, there is usually not enough space for placement of a standard perforated pipe and gravel drainage system. Under these circumstances, 2-inch diameter weepholes may be placed at the 8 feet on center along the base of the wall. The gravel may consist of ¾-inch to 1-inch crushed rocks. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard pipe and gravel system constructed under the slab.

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 external hydrostatic pressure due to water in addition to the lateral earth pressure. In any event, it is recommended that the 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 dry density obtainable by the latest revision of ASTM D 1557. Flooding shall 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.

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

TEMPORARY EXCAVATIONS

It is anticipated that temporary excavations up to 15 feet in vertical height will be required for proposed residential complex with one subterranean level. The excavations are expected to expose fill, and native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height should be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 15 feet. A uniform sloped excavation does not have a vertical component.

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

Excavation Observations

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

SHORING DESIGN

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials.

For design purposes, an allowable passive resistance for the earth materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.35 based on uniform contact between the lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

Water is not anticipated during excavation of soldier piles. Casing may be required should caving be experienced. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

Lagging

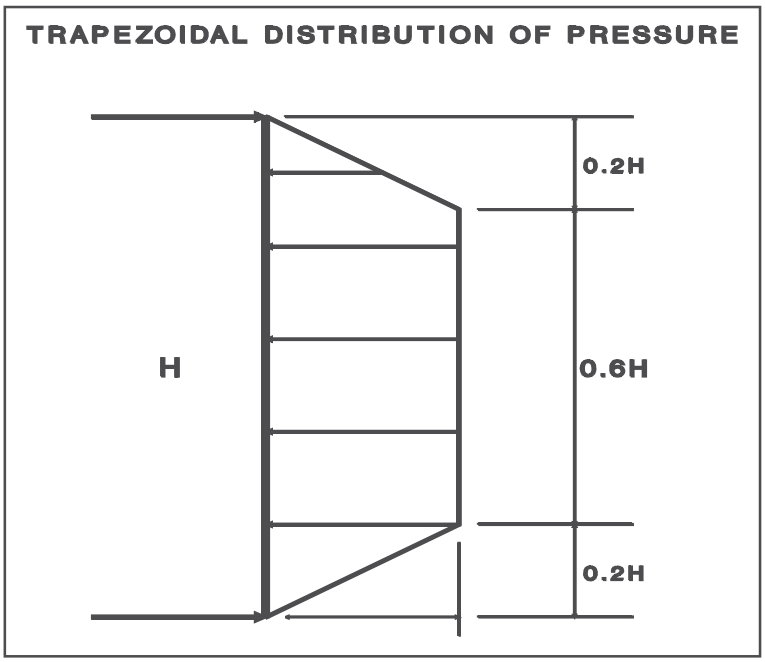
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The design of trapezoidal distribution of pressure is shown in the diagram below. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 15 ft	35 pcf	23H psf

*Where H is the height of the shoring in feet.



Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

Tie-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 30 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge.

Drilled friction anchors may be designed for a skin friction of 300 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. This skin friction is based on 15-foot-high shoring, a tied back anchor elevation 5 feet below grade and a minimum twenty-foot embedment beyond the potentially active wedge yielding an overburden of 10 feet below ground surface.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,000 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchors should be placed at least 6 feet on center to be considered isolated. It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity.

All anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading. After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Shoring Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

Shoring Observations

Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations ensure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

SLABS ON GRADE

Concrete Slabs on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness and should be reinforced with a minimum of #3 steel bars on 18-inch centers each way. Slabs-on-grade 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 laboratory maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness and should be reinforced with a minimum of #3 steel bars on 18-inch centers each way. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the laboratory maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, where necessary, it is recommended that a qualified consultant should be

engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor on various components of the structure.

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

All concrete slabs-on-grade should be supported on vapor retarder/barrier. The design of the slab and the installation of the vapor retarder/barrier should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder/barrier should comply with ASTM E 1745 Class A requirements. The necessity of a vapor retarder/barrier is not a geotechnical issue and should be confirmed by qualified members of the design team.

Where a vapor retarder/barrier is used, it should be placed on a level and compact subgrade. Precautions should be taken to protect the vapor retarder/barrier from damage during installation of reinforcing, utilities and concrete. The use of stakes driven thought the vapor retarder/barrier should be avoided. Repair any damaged areas of the vapor retarder/barrier prior to concrete placement.

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 where re-entrant slab corners occur.

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

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

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. Preliminary pavement sections are provided in the table below:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars (TI = 4)	3	5
Moderate Truck (TI = 6)	4	7
Heavy Truck (TI = 8)	7	9

The 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.

A subgrade modulus of 100 pounds per cubic inch may be assumed for the design of concrete paving. Concrete paving for passenger cars and moderate truck traffic shall be a minimum of 6 inches in thickness and shall be underlain by 6 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum of 8 inches in thickness and shall be underlain by 6 inches of aggregate base. Concrete paving should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacing 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.

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 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

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.

A percolation test was performed in boring B1 at depths between 25 and 45 feet below the existing site grade to provide a preliminary assessment of the stormwater infiltration feasibility. The percolation test was performed following the procedure for a deep borehole provided in the Guidelines for Design, Investigation and Reporting Low Impact Development Stormwater Infiltration (GS200.2), dated June 30, 2021, presented in the Administrative Manual for the County of Los Angeles, Department of Public Works, Geotechnical and Material Engineering Division. After applying the reduction factors, a design infiltration rate of 0.04 inches per hour was obtained. This design infiltration rate does not meet the minimum required infiltration rate specified in the GS200.2 guideline specified for stormwater infiltration systems.

Due to cohesive nature and the relatively low permeability of the underlying site soils, it is the preliminary assessment of this firm that stormwater infiltration will not be feasible for the Project Site.

Where infiltration of stormwater into the subgrade is not advisable, Building Officials have typically allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to

prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminants discharged into the planters. It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

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 aids 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 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 engineering 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, shored or slot cut. 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. 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 project described. The 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.

Geotechnical observations and testing during construction is considered to be a continuation of 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.

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 borings 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 soil samples were delivered to the laboratory for testing and examination. Samples noted on the excavation logs as SPT samples are obtained in 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 by 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’s 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 the load are made on the basis of the consolidation tests using 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 in 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 hours or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height and multiplied by 1,000.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of the most recent revision of ASTM D 1557. 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.

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 revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. The grain size distributions are plotted on the E-Plates presented in the Enclosures of this report.

REFERENCES

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Bray, J. D., Sancio R.B., 2004, Assessment of Liquefaction Susceptibility of Fine-Grained Soils.

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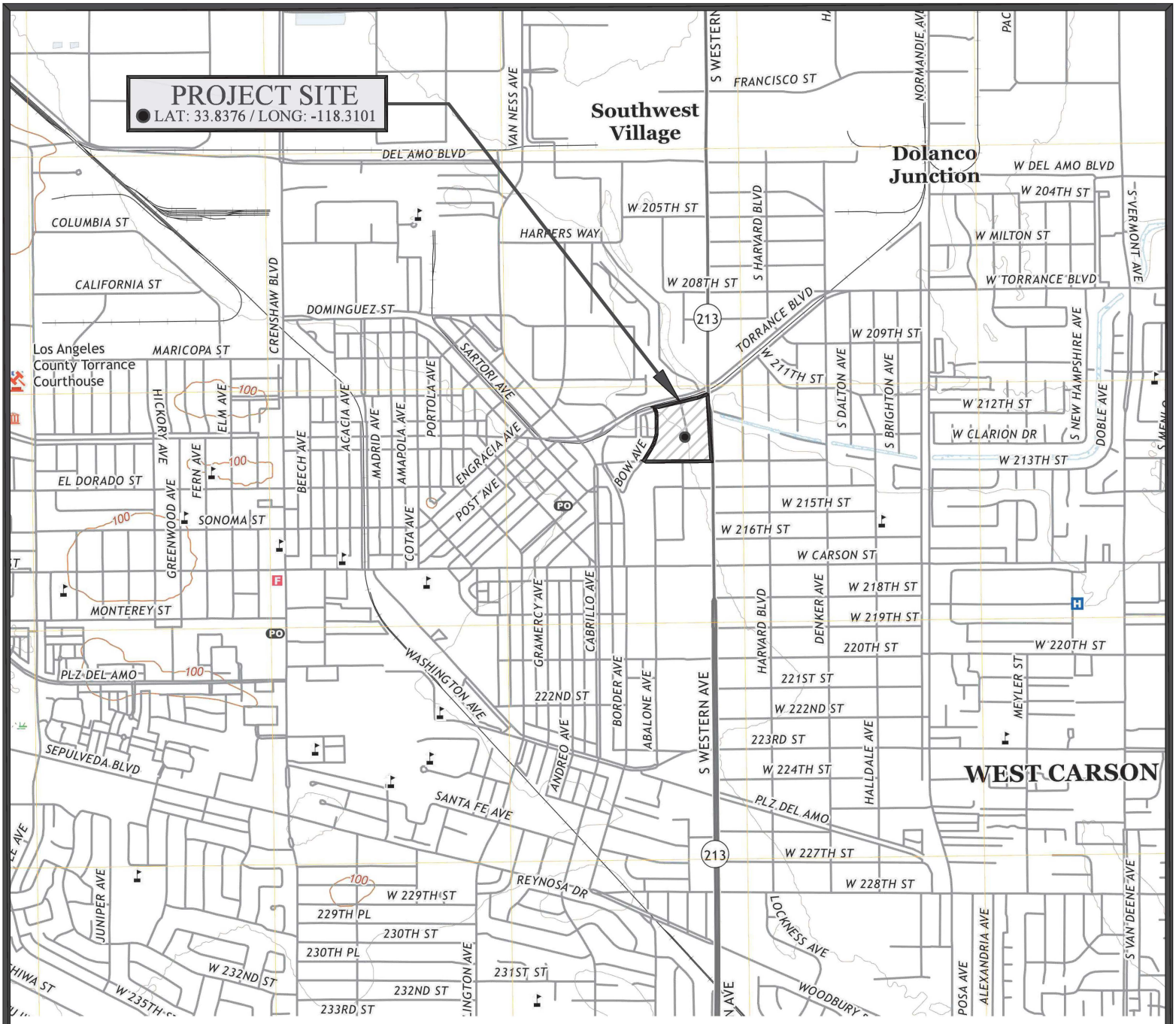
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Hart, E.W. and Bryant, W.A., 1999 (updated 2005), Fault Rupture Zones in California, Division of Mines and Geology, Special Publication 42, 25pp.

Idriss I.M. and Boulanger R W., 2008, Monograph (MNO-12), California Earthquake Engineering Research Institute.

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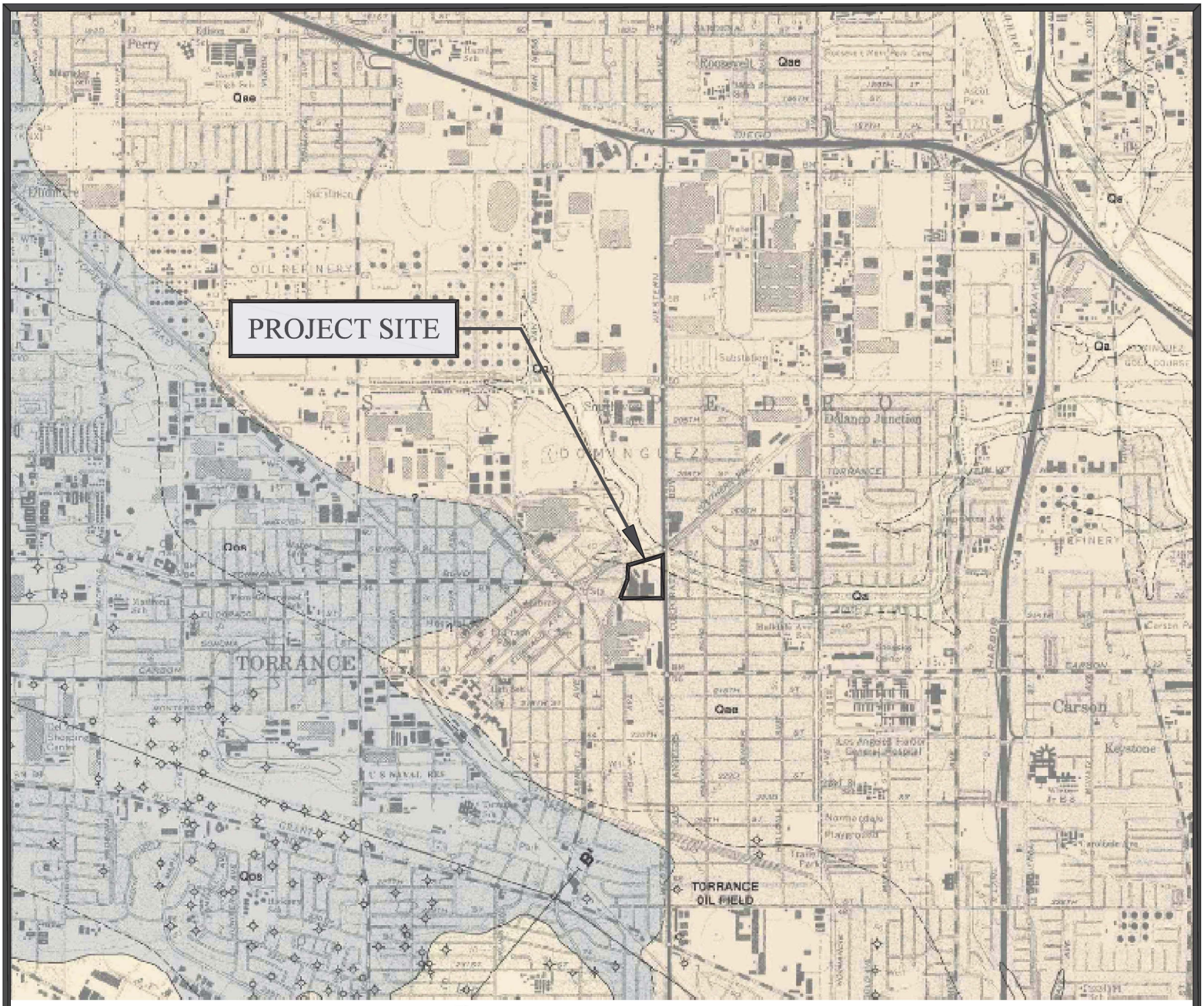
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 - MINUTE SERIES, TORRANCE, CA QUADRANGLE 2021

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

TORRANCE CENTER SOUTH COMPLEX, LLC
21151, 21171, 21221, 21241, 21281 S WESTERN AVE., TORRANCE

FILE NO: 22600



LEGEND

Qa: Surficial Sediments - Alluvium, mostly loamy clay of valley and flood plains
Qae: Alluvium, similar to Qa but slightly elevated and locally dissected
Qos: Older Dissected Surficial Sediments - Older, stabilized dune and drift sand - mostly unconsolidated fine-grained sand

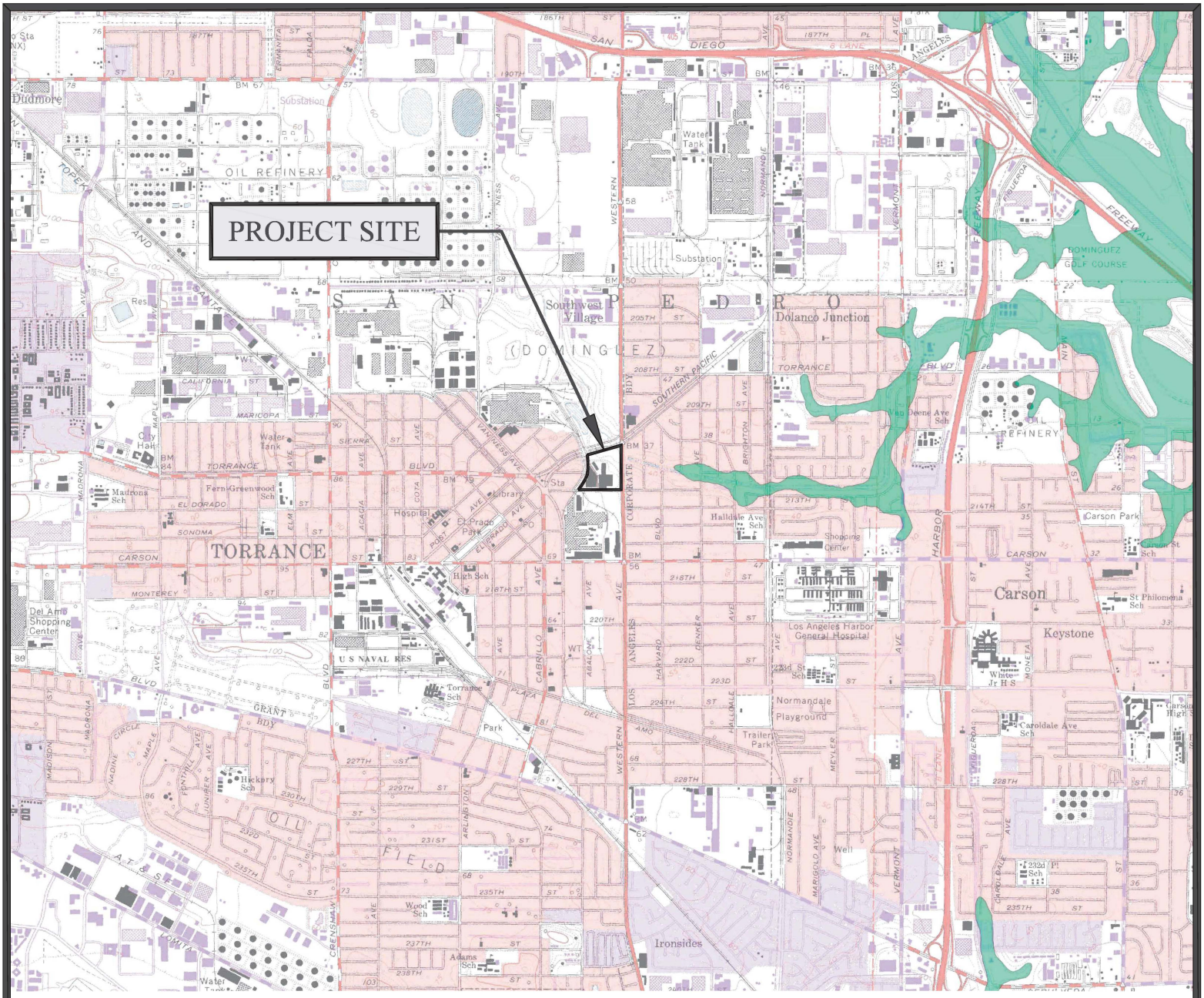
REFERENCE: T.W. DIBBLEE (2011) GEOLOGIC MAP OF REDONDO BEACH, TORRANCE, AND SAN PEDRO QUADRANGLES (#DF-70)

GEOLOGIC MAP (DIBBLEE)

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FILE NO: 22600



LEGEND

LIQUEFACTION ZONES

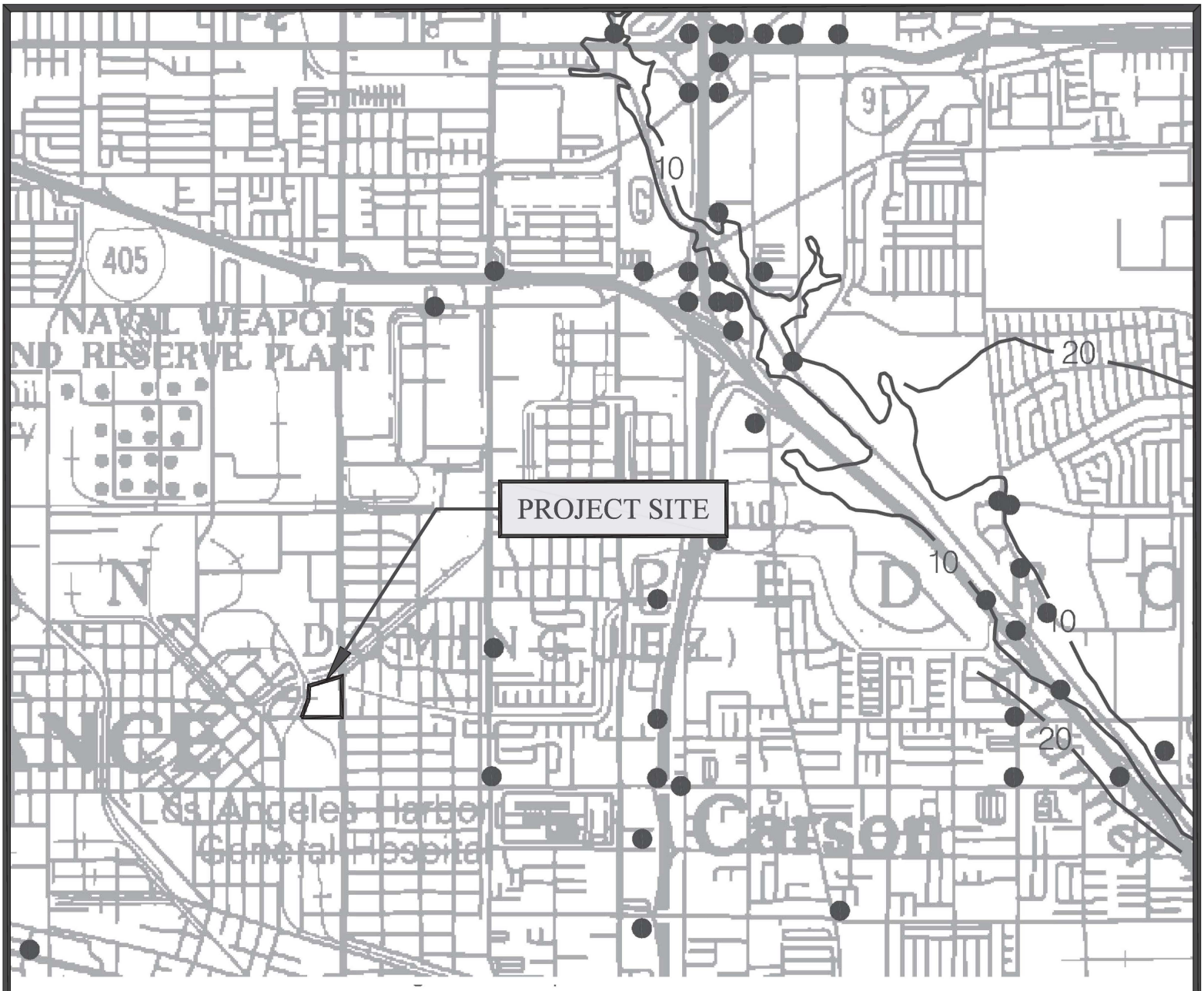
REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, TORRANCE QUADRANGLE (CGS, 2016)

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

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FILE NO: 22600



LEGEND

- Borehole Site
- 30 — Depth to ground water in feet

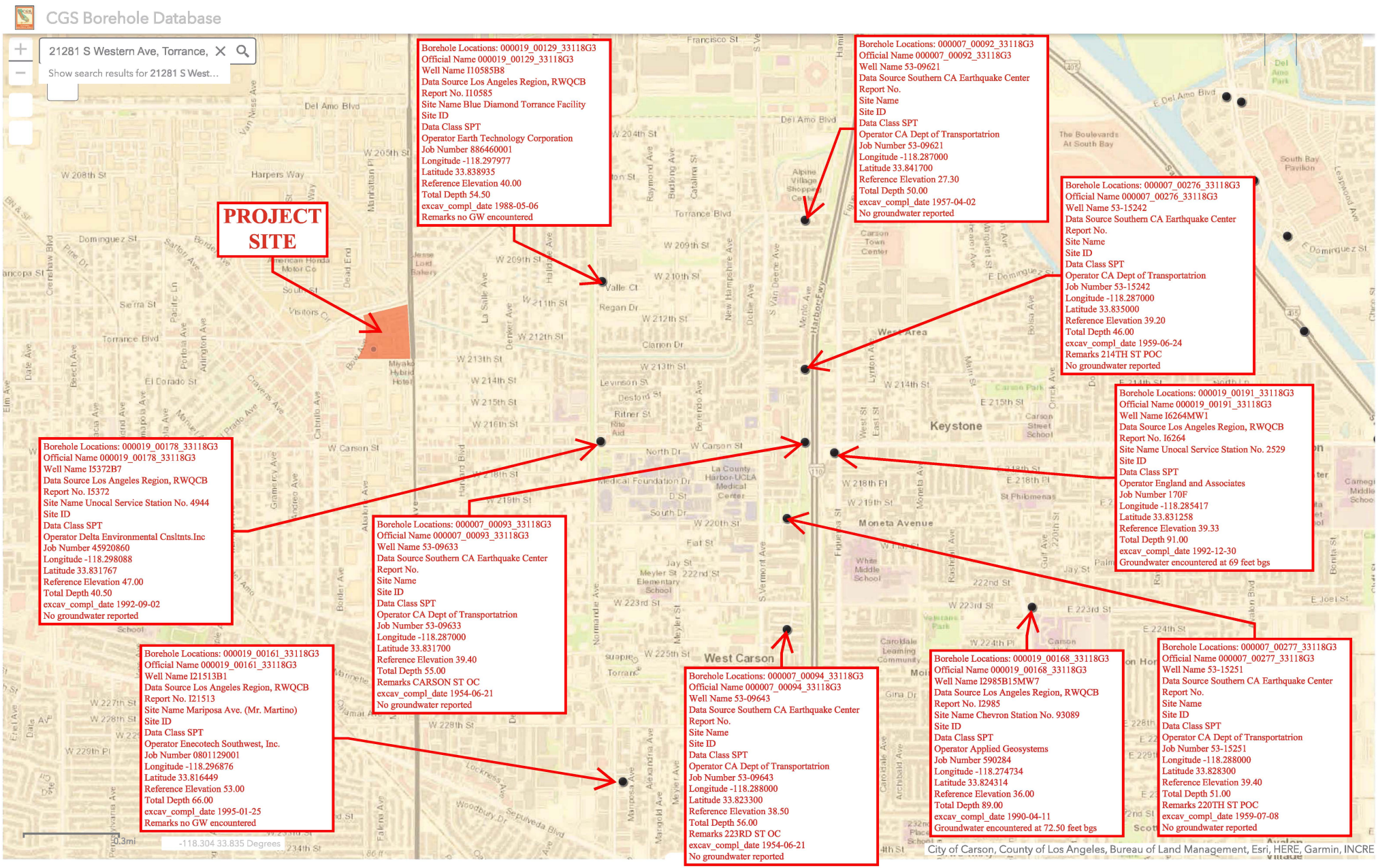
REFERENCE: SEISMIC HAZARD ZONE REPORT, 035 TORRANCE, 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA 1998 (REV 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS MAP

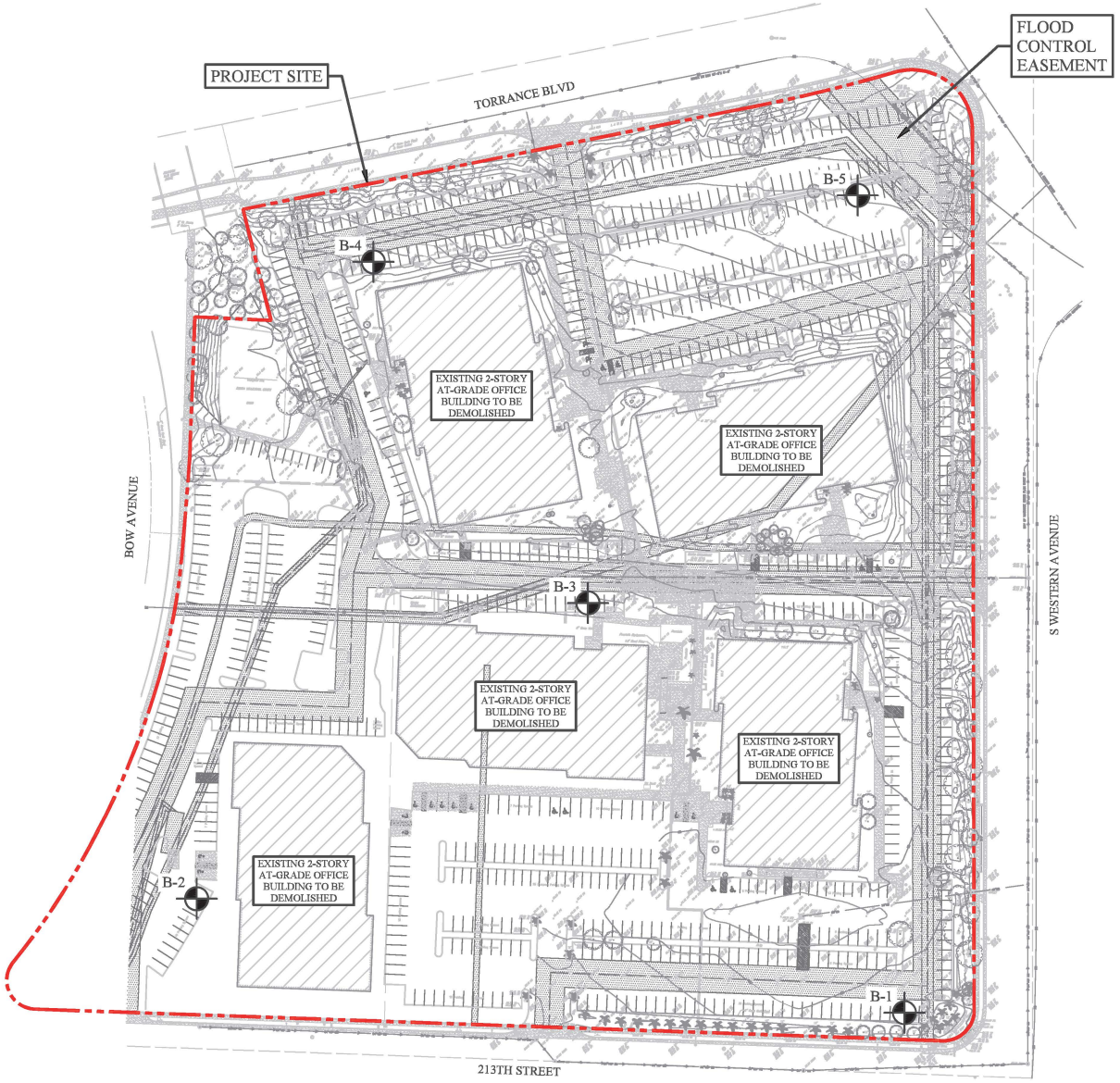
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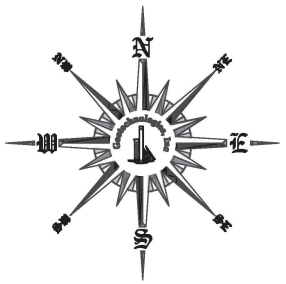
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FILE NO: 22600


CGS Borehole Database
21281 S Western Ave, Torrance,
Show search results for 21281 S West...
PROJECT SITE
Borehole Locations: 000019_00129_33118G3
Official Name 000019_00129_33118G3
Well Name I10585B8
Data Source Los Angeles Region, RWQCB
Report No. I10585
Site Name Blue Diamond Torrance Facility
Site ID
Data Class SPT
Operator Earth Technology Corporation
Job Number 886460001
Longitude -118.297977
Latitude 33.838935
Reference Elevation 40.00
Total Depth 54.50
excav_compl_date 1988-05-06
Remarks no GW encountered
Borehole Locations: 000007_00092_33118G3
Official Name 000007_00092_33118G3
Well Name 53-09621
Data Source Southern CA Earthquake Center
Report No.
Site Name
Site ID
Data Class SPT
Operator CA Dept of Transportatrion
Job Number 53-09621
Longitude -118.287000
Latitude 33.841700
Reference Elevation 27.30
Total Depth 50.00
excav_compl_date 1957-04-02
No groundwater reported
Borehole Locations: 000007_00276_33118G3
Official Name 000007_00276_33118G3
Well Name 53-15242
Data Source Southern CA Earthquake Center
Report No.
Site Name
Site ID
Data Class SPT
Operator CA Dept of Transportatrion
Job Number 53-15242
Longitude -118.287000
Latitude 33.835000
Reference Elevation 39.20
Total Depth 46.00
excav_compl_date 1959-06-24
Remarks 214TH ST POC
No groundwater reported
Borehole Locations: 000019_00191_33118G3
Official Name 000019_00191_33118G3
Well Name I6264MW1
Data Source Los Angeles Region, RWQCB
Report No. I6264
Site Name Unocal Service Station No. 2529
Site ID
Data Class SPT
Operator England and Associates
Job Number 170F
Longitude -118.285417
Latitude 33.831258
Reference Elevation 39.33
Total Depth 91.00
excav_compl_date 1992-12-30
Groundwater encountered at 69 feet bgs
Borehole Locations: 000019_00178_33118G3
Official Name 000019_00178_33118G3
Well Name I5372B7
Data Source Los Angeles Region, RWQCB
Report No. I5372
Site Name Unocal Service Station No. 4944
Site ID
Data Class SPT
Operator Delta Environmental Cnsltnts.Inc
Job Number 45920860
Longitude -118.298088
Latitude 33.831767
Reference Elevation 47.00
Total Depth 40.50
excav_compl_date 1992-09-02
No groundwater reported
Borehole Locations: 000007_00093_33118G3
Official Name 000007_00093_33118G3
Well Name 53-09633
Data Source Southern CA Earthquake Center
Report No.
Site Name
Site ID
Data Class SPT
Operator CA Dept of Transportatrion
Job Number 53-09633
Longitude -118.287000
Latitude 33.831700
Reference Elevation 39.40
Total Depth 55.00
Remarks CARSON ST OC
excav_compl_date 1954-06-21
No groundwater reported
Borehole Locations: 000019_00161_33118G3
Official Name 000019_00161_33118G3
Well Name I21513B1
Data Source Los Angeles Region, RWQCB
Report No. I21513
Site Name Mariposa Ave. (Mr. Martino)
Site ID
Data Class SPT
Operator Enecotech Southwest, Inc.
Job Number 0801129001
Longitude -118.296876
Latitude 33.816449
Reference Elevation 53.00
Total Depth 66.00
excav_compl_date 1995-01-25
Remarks no GW encountered
Borehole Locations: 000007_00094_33118G3
Official Name 000007_00094_33118G3
Well Name 53-09643
Data Source Southern CA Earthquake Center
Report No.
Site Name
Site ID
Data Class SPT
Operator CA Dept of Transportatrion
Job Number 53-09643
Longitude -118.288000
Latitude 33.823300
Reference Elevation 38.50
Total Depth 56.00
Remarks 223RD ST OC
excav_compl_date 1954-06-21
No groundwater reported
Borehole Locations: 000019_00168_33118G3
Official Name 000019_00168_33118G3
Well Name I2985B15MW7
Data Source Los Angeles Region, RWQCB
Report No. I2985
Site Name Chevron Station No. 93089
Site ID
Data Class SPT
Operator Applied Geosystems
Job Number 590284
Longitude -118.274734
Latitude 33.824314
Reference Elevation 36.00
Total Depth 89.00
excav_compl_date 1990-04-11
Groundwater encountered at 72.50 feet bgs
Borehole Locations: 000007_00277_33118G3
Official Name 000007_00277_33118G3
Well Name 53-15251
Data Source Southern CA Earthquake Center
Report No.
Site Name
Site ID
Data Class SPT
Operator CA Dept of Transportatrion
Job Number 53-15251
Longitude -118.288000
Latitude 33.828300
Reference Elevation 39.40
Total Depth 51.00
Remarks 220TH ST POC
excav_compl_date 1959-07-08
No groundwater reported
Keystone
Moneta Avenue
West Carson
Del Amo Blvd
Torrance Blvd
Harpers Way
Dominguez St
El Dorado St
W Carson St
Sepulveda Blvd
City of Carson, County of Los Angeles, Bureau of Land Management, Esri, HERE, Garmin, INCRE

Plot Date: Feb. 5, 2025
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LEGEND

B-5 — LOCATION AND NUMBER OF BORING (THIS INVESTIGATION)

DEMOLITION OF EXISTING BUILDINGS

EASEMENT LINES

REFERENCE: SURVEY PLAN BY MOLLENHAUER GROUP DATED 08/21/2024

SITE PLAN

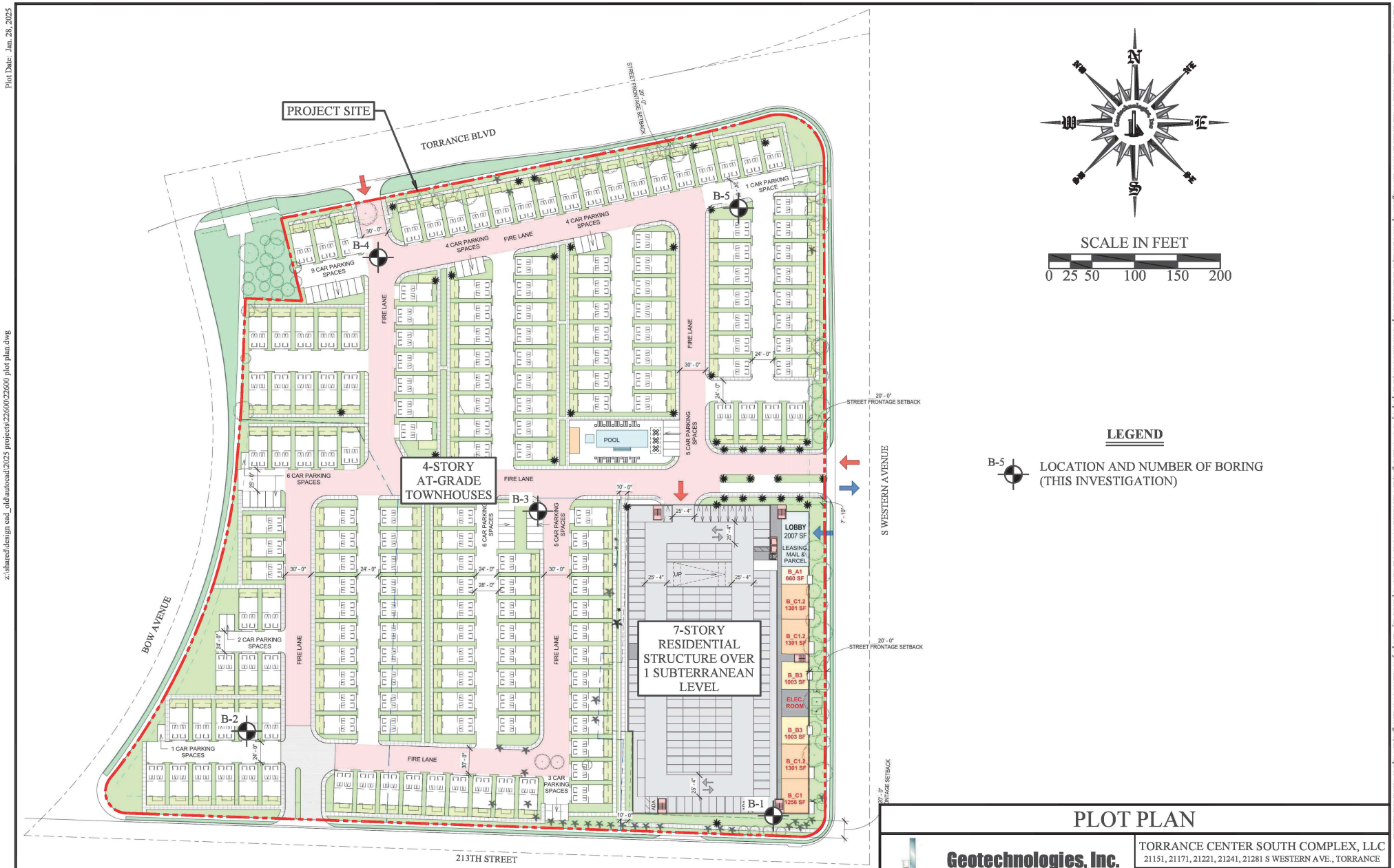
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TORRANCE CENTER SOUTH COMPLEX, LLC
21151, 21171, 21221, 21241, 21281 S WESTERN AVE., TORRANCE

Drawn by: JD	File No.: 22600
Date: February 2025	

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Plot Date: Jan. 28, 2025
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REFERENCE: SITE PLAN BY MVE + PARTNERS DATED 11/20/2024

PLOT PLAN

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TORRANCE CENTER SOUTH COMPLEX, LLC
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Drawn by: JD	File No.: 22600
Date: January 2025	

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BORING LOG NUMBER 1

Torrence Center South Complex **Date: 01/02/25** **Elevation: 55.9'***

File No. 22600 **Method: 8-inch diameter Hollow Stem Auger**

kk/km

*Reference: Based on Survey Plan by Mollenhauer Group dated August 21, 2024

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 4-inch Base
				1		FILL: Clayey Silt to Sandy Clay, dark and grayish brown, moist, stiff
2.5	26	23.8	102.4	2		
				3		
				4		
5	15	26.3	SPT	5	ML/CL	NATIVE SOIL: Clayey Silt to Sandy Clay, yellowish brown, moist, stiff
				6		
				7		
7.5	50	26.6	99.8	8		very stiff
				9		
10	26	11.2	SPT	10	SM/ML	Silty Sand, yellowish brown, moist, dense, fine grained
				11		
				12		
12.5	43	15.9	111.4	13		dark and yellowish brown, dense
				14		
15	22	14.7	SPT	15	SC	Clayey Sand, grayish brown, moist, dense, fine grained
				16		
				17		
17.5	41	18.0	108.3	18	ML	Sandy Silt, dark to yellowish brown, stiff, fine grained
				19		
20	20	22.5	SPT	20		Sandy Silt, grayish brown, moist, stiff
				21		
				22		
22.5	30	18.7	107.9	23		Sandy Silt, yellowish brown, moist, dense to stiff, fine grained
				24		
25	54	19.4	SPT	25		Sandy Silt, yellowish and grayish brown, moist, very stiff

BORING LOG NUMBER 1

Torrence Center South Complex

File No. 22600

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 --		Sandy Silt, yellowish and grayish brown, moist, very stiff
27.5	39	24.1	98.1	- 28 -- - 29 --		dark and grayish brown
30	24	20.9	SPT	- 30 -- - 31 -- - 32 --		stiff
32.5	62	15.5	109.1	- 33 -- - 34 --	SM/ML	Silty Sand to Sandy Silt, dark and grayish brown, moist, dense to very stiff, fine grained
35	21	27.4	SPT	- 35 -- - 36 -- - 37 --	CL	Sandy Clay, dark and grayish brown, moist, stiff to very stiff
37.5	50	35.4	90.8	- 38 -- - 39 --		
40	24	19.7	SPT	- 40 -- - 41 -- - 42 --	SM	Silty Sand, dark and grayish brown, moist, dense to very dense, fine grained
42.5	70	13.6	114.5	- 43 -- - 44 --		
45	28	17.1	SPT	- 45 -- - 46 -- - 47 --		
47.5	50	17.9	111.4	- 48 -- - 49 --		
50	26	29.7	SPT	- 50 -- -	CL	Silty Clay, dark and yellowish brown, moist, stiff

BORING LOG NUMBER 1

Torrence Center South Complex

File No. 22600

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 51 -- - 52 --	CL	Silty Clay, dark and yellowish brown, moist, stiff
52.5	80	30.9	95.7	- 53 -- - 54 -- -		very stiff
55	25	23.7	SPT	55 -- - 56 -- - 57 --	ML	Sandy Silt, grayish brown, wet, stiff, fine grained
57.5	70	20.2	105.3	- 58 -- - 59 -- -		
60	34	25.3	SPT	60 -- - 61 -- - 62 -- - 63 -- - 64 -- - 65 -- - 66 -- - 67 -- - 68 -- - 69 -- - 70 -- - 71 -- - 72 -- - 73 -- - 74 -- - 75 -- -		Total Depth 60 feet Water at 55 feet Fill to 4 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 SPT=Standard Penetration Test

BORING LOG NUMBER 2

Torrence Center South Complex **Date: 01/02/25** **Elevation: Not Known**

File No. 22600 **Method: 8-inch diameter Hollow Stem Auger**

kk/km

*Reference: Based on Survey Plan by Mollenhauer Group dated August 21, 2024

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 --		7-inch Asphalt over 4-inch Base
				-		
				1 --		
				-		FILL: Clayey Silt to Sandy Clay, dark brown, moist, stiff
				2 --		
2.5	39	23.8	106.9	-		
				3 --		
				-	ML/CL	NATIVE SOIL: Clayey Silt to Sandy Clay, dark to yellowish
				4 --		brown, moist, very stiff
				-		
5	39	24.1	102.1	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	44	22.2	106.4	10 --		
				-		grayish brown, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	100/10"	17.7	114.6	15 --		
				-	SM/SC	Silty Sand to Clayey Sand, dark to yellowish brown, moist,
				16 --		very dense, very stiff, fine grained
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/8"	22.1	106.1	20 --		
				-		Total Depth 20 feet
				21 --		No Water
				-		Fill to 3 feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 3

Torrence Center South Complex **Date: 01/03/25** **Elevation: 53.5'***

File No. 22600 **Method: 8-inch diameter Hollow Stem Auger**

kk/km

*Reference: Based on Survey Plan by Mollenhauer Group dated August 21, 2024

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 --		3-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Clay, dark brown, moist, very stiff
				-		
				2 --		
2.5	72	23.0	100.4	-		
				3 --		
				-	ML/CL	NATIVE SOIL: Sandy Silt to Sandy Clay, dark to yellowish brown, moist, very stiff
				4 --		
				-		
5	27 50/5"	17.4	111.2	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	34 50/5"	19.0	111.3	10 --	SM	Silty Sand, dark to yellowish brown, moist, very dense, fine grained
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	48 50/5"	28.7	94.8	15 --	ML	Sandy Silt, dark to yellowish brown, moist, very stiff
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/8"	23.3	102.3	20 --	SM	Silty Sand, dark to yellowish brown, moist, very dense, fine grained
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	100/9"	25.1	96.1	25 --	ML	Sandy Silt, dark to yellowish brown, moist, very stiff
				-		

BORING LOG NUMBER 3

Torrence Center South Complex

File No. 22600

kk/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, dark to yellowish brown, moist, very stiff
30	100/12"	30.0	95.2	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 No Water 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 4

Torrence Center South Complex **Date: 01/03/25** **Elevation: 49.0'***

File No. 22600 **Method: 8-inch diameter Hollow Stem Auger**

kk/km

*Reference: Based on Survey Plan by Mollenhauer Group dated August 21, 2024

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 4-inch Base
				1 --		FILL: Silty Sand, dark and grayish brown, moist, medium dense, fine grained
				2 --		
2.5	30 50/5"	11.9	110.5	3 --		very dense, asphalt fragments
				4 --		
5	100/12"	3.4	116.1	5 --		dark brown, with cobbles and concrete fragments
				6 --		
				7 --		
7.5	100/10"	9.7	112.0	8 --		
				9 --		
10	98	10.3	107.5	10 --	SM	NATIVE SOILS: Silty Sand, dark brown, moist, very dense, fine grained
				11 --		
				12 --		
				13 --		
				14 --		
15	100/11"	7.9	111.8	15 --		dark brown
				16 --		
				17 --		
				18 --		
				19 --		
20	100/9"	11.6	118.3	20 --		Total Depth 20 feet No Water Fill to 10 feet
				21 --		
				22 --		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24 --		
				25 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 5

Torrence Center South Complex **Date: 01/03/25** **Elevation: 41.8'***

File No. 22600 **Method: 8-inch diameter Hollow Stem Auger**

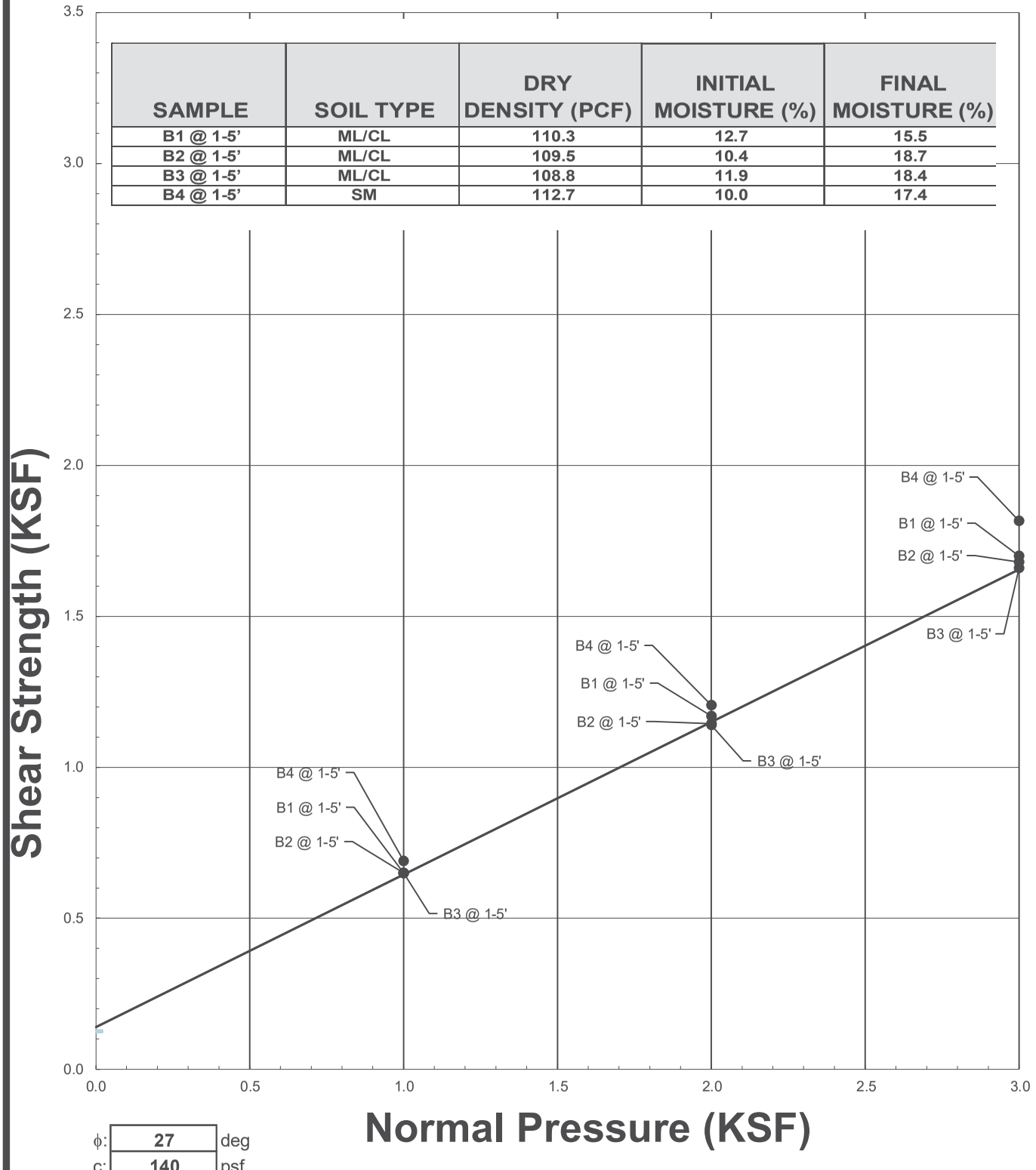
kk/km

*Reference: Based on Survey Plan by Mollenhauer Group dated August 21, 2024

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				-		
				2 --		
2.5	36	12.2	110.7	-		
				3 --		Silty Sand, dark and yellowish brown, moist, medium dense, fine grained, minor fragments of asphalt
				-		
				4 --		
				-		
5	47	No Recovery		5 --		
				-		
				6 --		
				-		
				7 --		
7.5	44	15.2	110.8	-		Silty Sand to Sandy Silt, dark to yellowish brown, moist, dense to stiff, fine grained
				8 --		
				-		
				9 --		
				-		
				10 --		
10	35	No Recovery		-		
				11 --		
				-		
				12 --		
12.5	80	11.8	106.8	-		
				13 --		Silty Sand, dark brown, moist, very dense, fine grained
				-		
				14 --		
				-	CL	NATIVE SOILS: Sandy Clay, dark and grayish brown, moist, very stiff
15	100/8"	17.4	109.7	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-	SM	Silty Sand, yellowish brown, moist, very dense, fine grained
20	100/12"	12.5	116.3	20 --		
				-		Total Depth 20 feet
				21 --		No Water
				-		Fill to 14 feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BULK SAMPLES REMOLDED TO 90% OF THE MAXIMUM DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 1-5'	ML/CL	110.3	12.7	15.5
B2 @ 1-5'	ML/CL	109.5	10.4	18.7
B3 @ 1-5'	ML/CL	108.8	11.9	18.4
B4 @ 1-5'	SM	112.7	10.0	17.4



ϕ:	27	deg
c:	140	psf

SHEAR TEST DIAGRAM

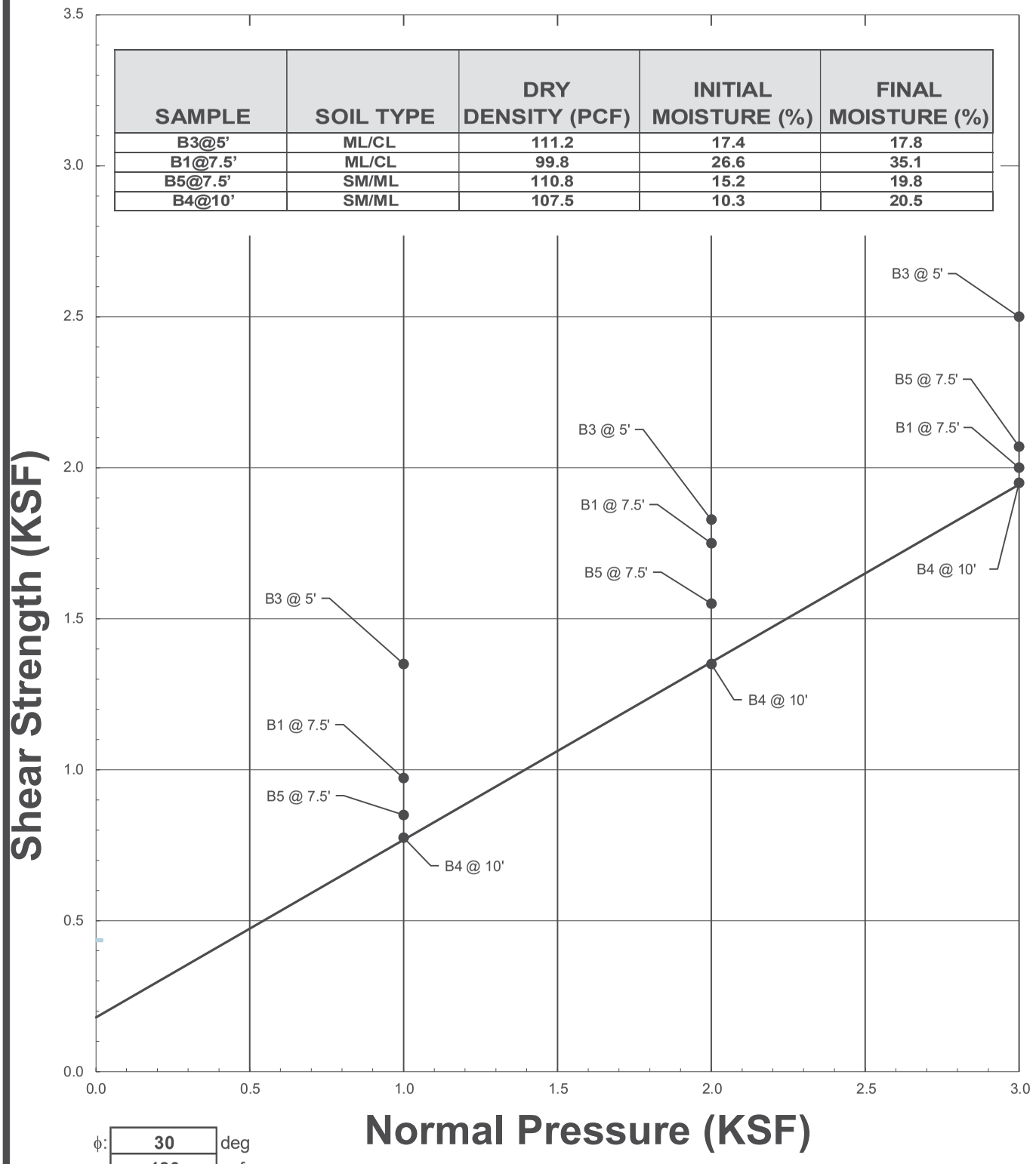
Geotechnologies, Inc.
Consulting Geotechnical Engineers

TORRANCE CENTER SOUTH COMPLEX
21151, 21171, 21221, 21241, 21281 S. WESTERN AVE., TORRANCE

FILE NO.: 22600 | **PLATE: B-1**

NATIVE SOIL

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@5'	ML/CL	111.2	17.4	17.8
B1@7.5'	ML/CL	99.8	26.6	35.1
B5@7.5'	SM/ML	110.8	15.2	19.8
B4@10'	SM/ML	107.5	10.3	20.5



ϕ:	30	deg
c:	180	psf

SHEAR TEST DIAGRAM

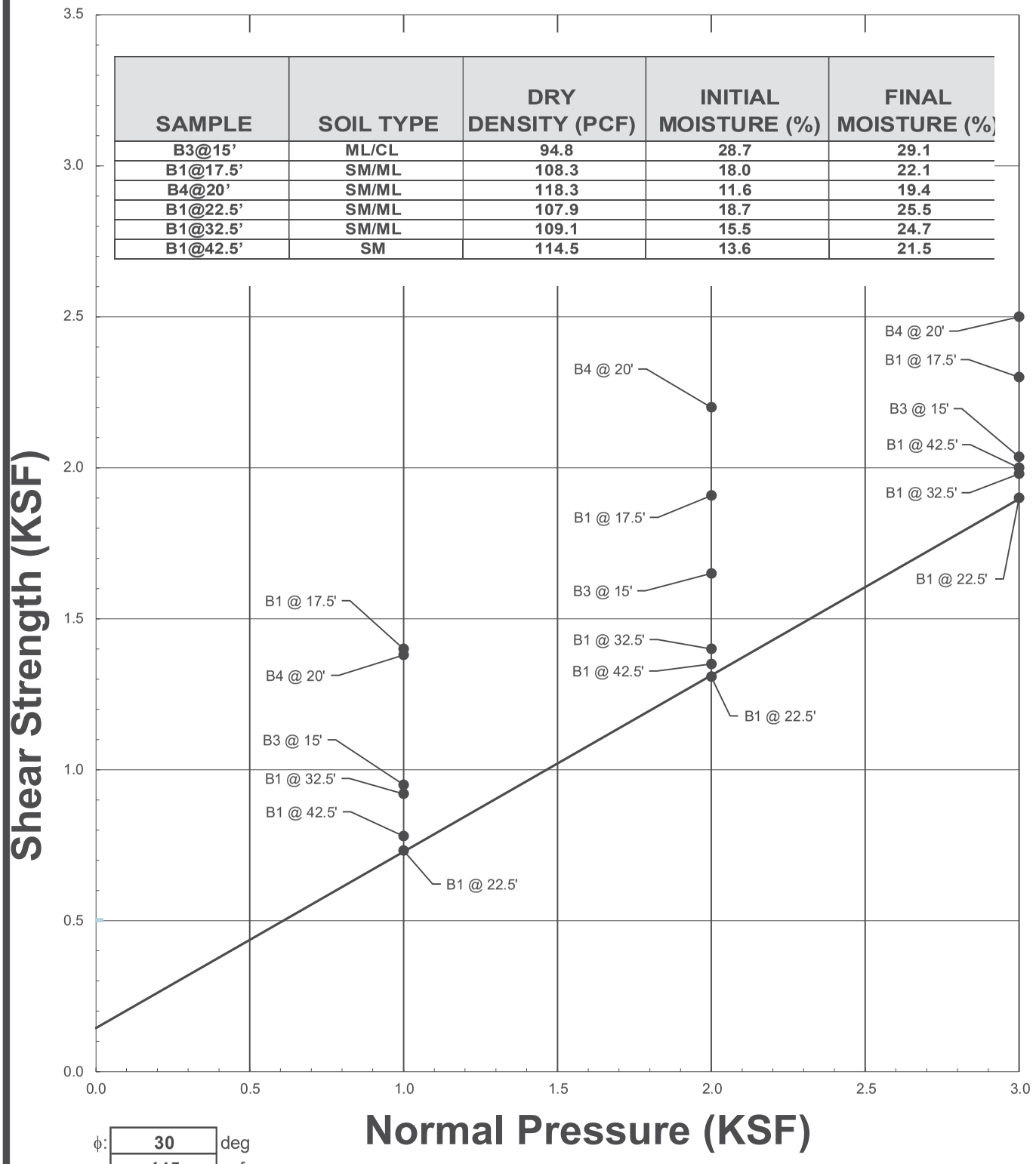
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TORRANCE CENTER SOUTH COMPLEX
21151, 21171, 21221, 21241, 21281 S. WESTERN AVE., TORRANCE

FILE NO.: 22600 | PLATE: B-2

NATIVE SOIL

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@15'	ML/CL	94.8	28.7	29.1
B1@17.5'	SM/ML	108.3	18.0	22.1
B4@20'	SM/ML	118.3	11.6	19.4
B1@22.5'	SM/ML	107.9	18.7	25.5
B1@32.5'	SM/ML	109.1	15.5	24.7
B1@42.5'	SM	114.5	13.6	21.5



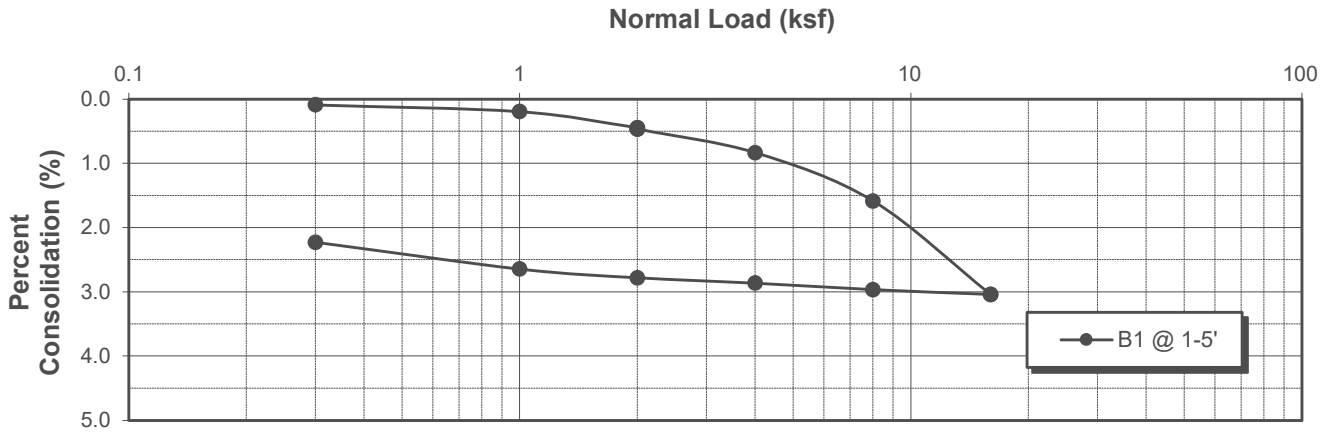
ϕ:	30	deg
c:	145	psf

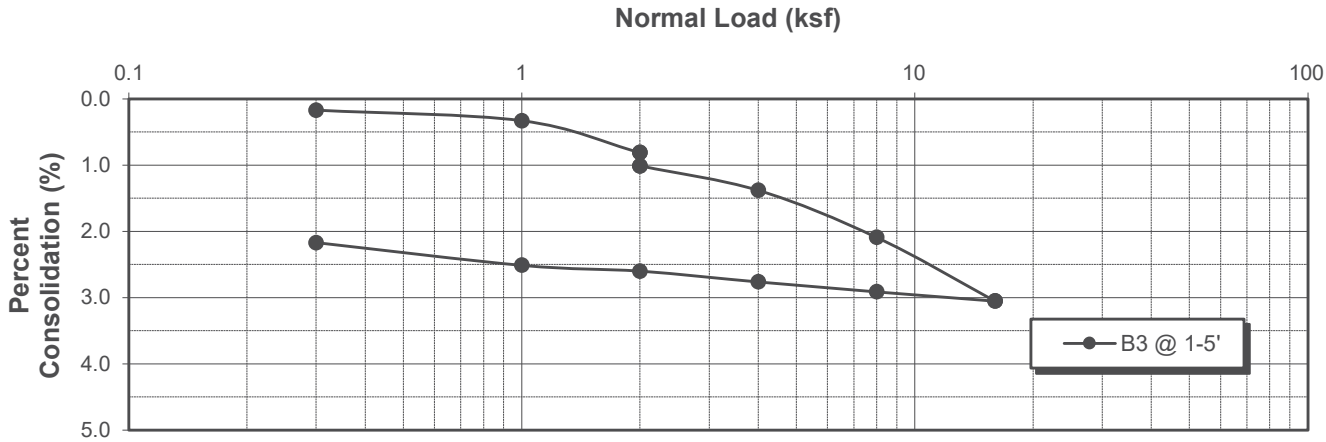
SHEAR TEST DIAGRAM

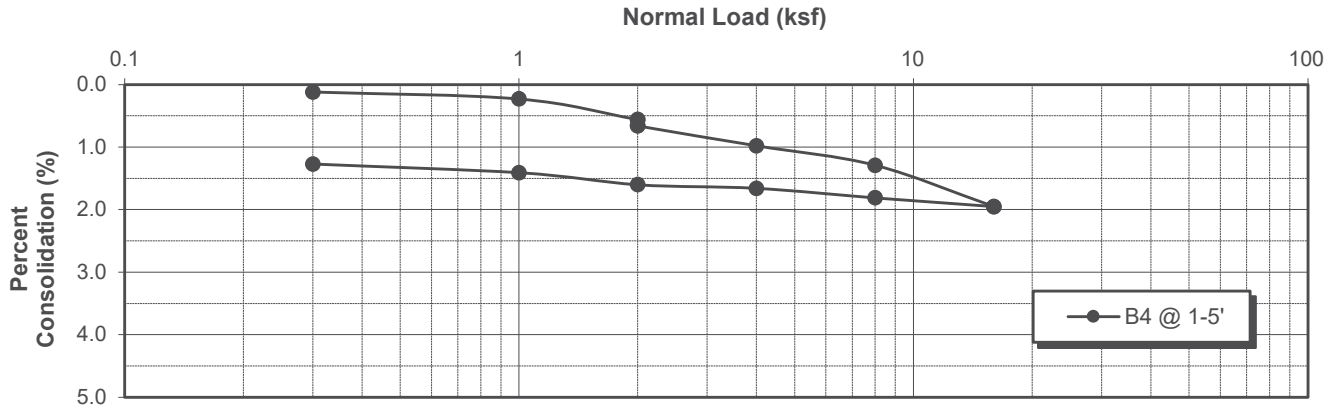
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TORRANCE CENTER SOUTH COMPLEX
21151, 21171, 21221, 21241, 21281 S. WESTERN AVE., TORRANCE

FILE NO.: 22600 | PLATE: B-3







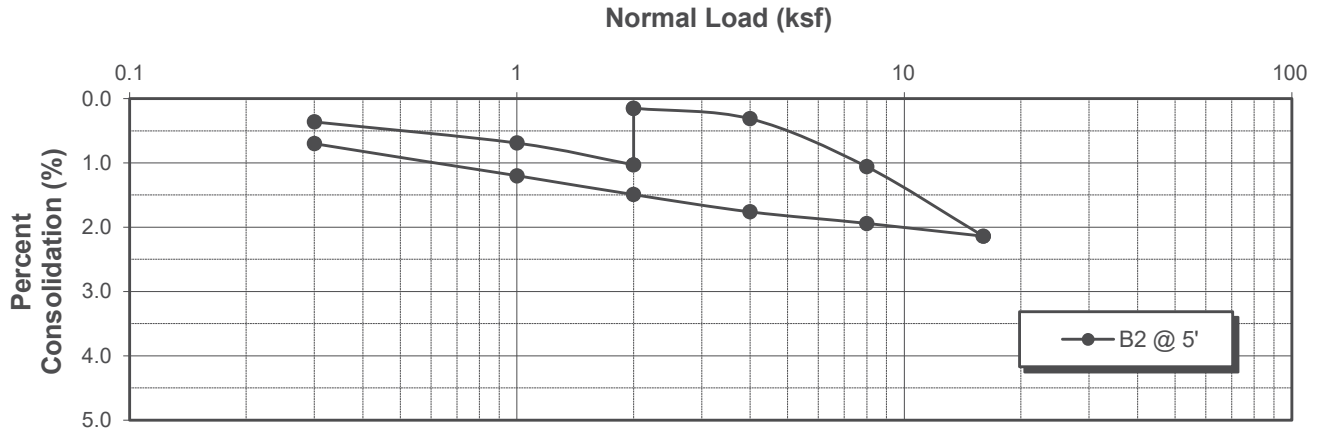
Water added at 2 KSF

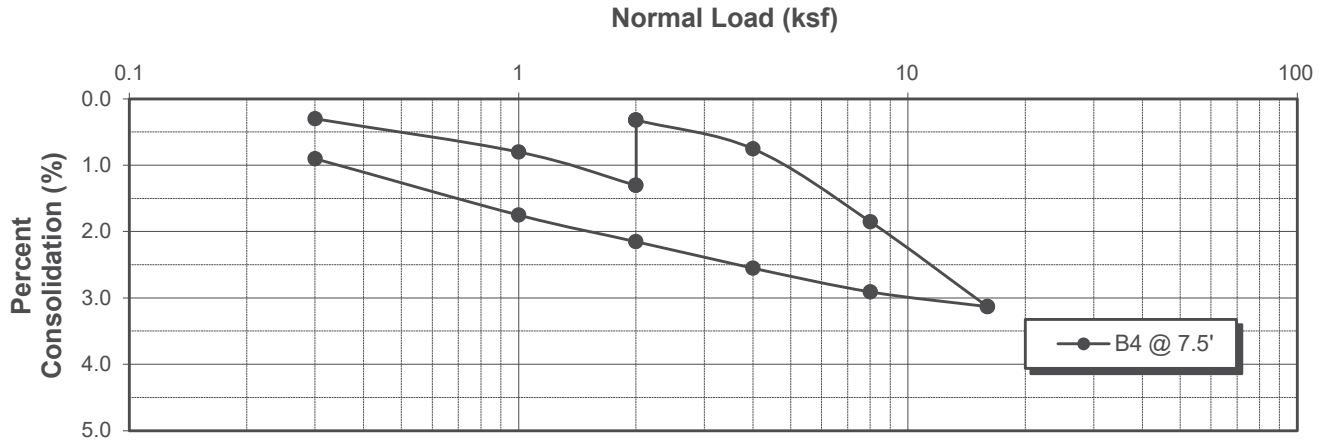
CONSOLIDATION

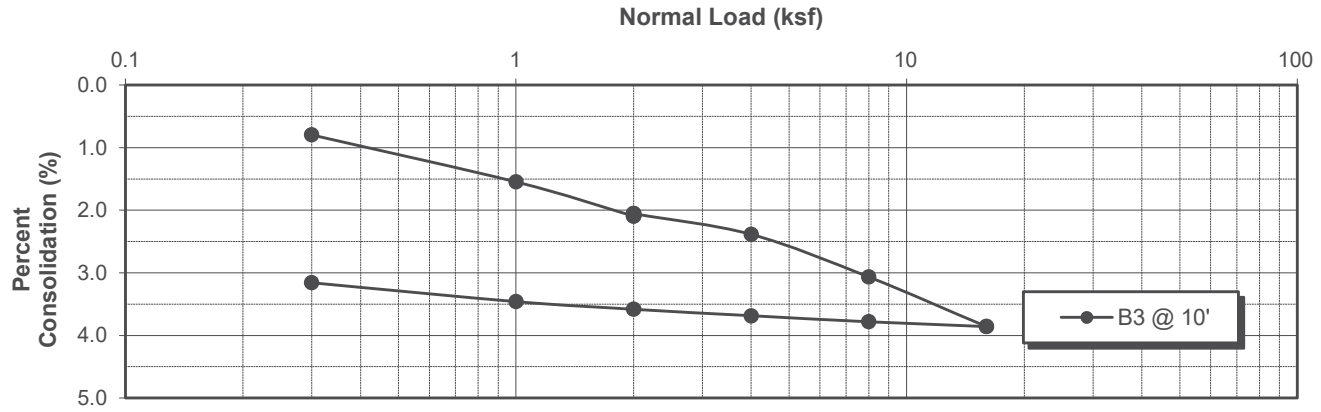
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: Torrance Center South Complex, LLC

FILE NO.: 22600 | PLATE: C-1





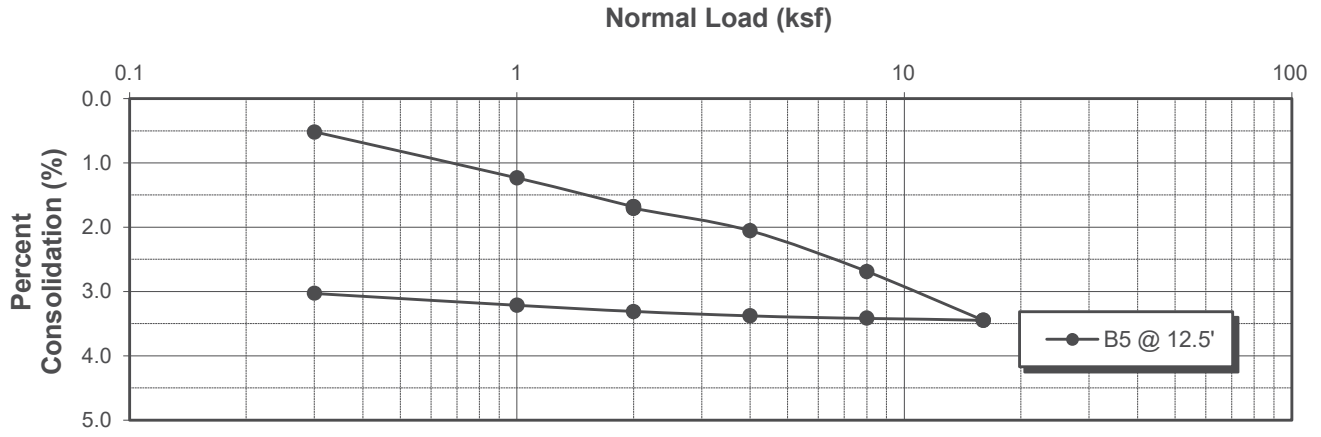


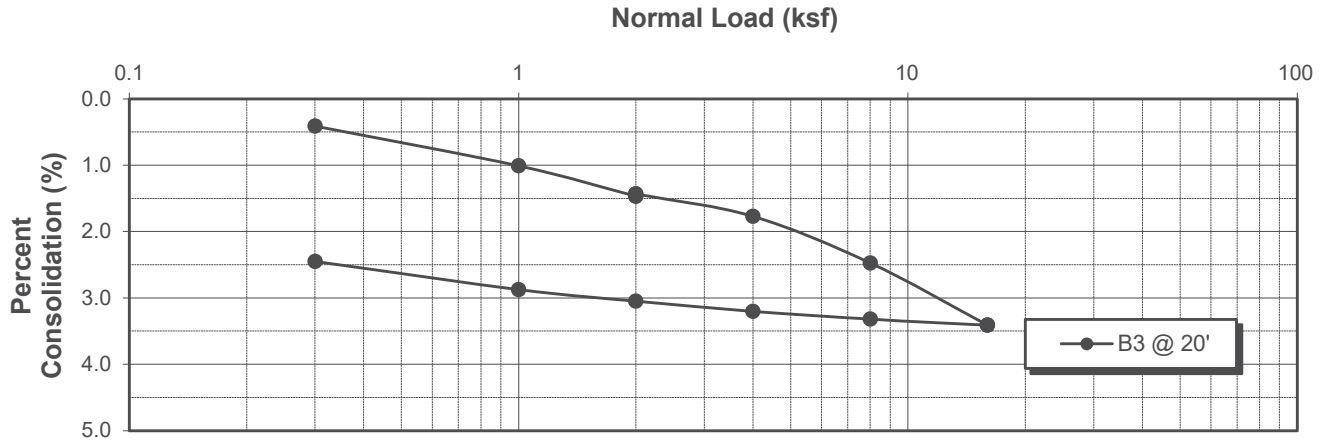
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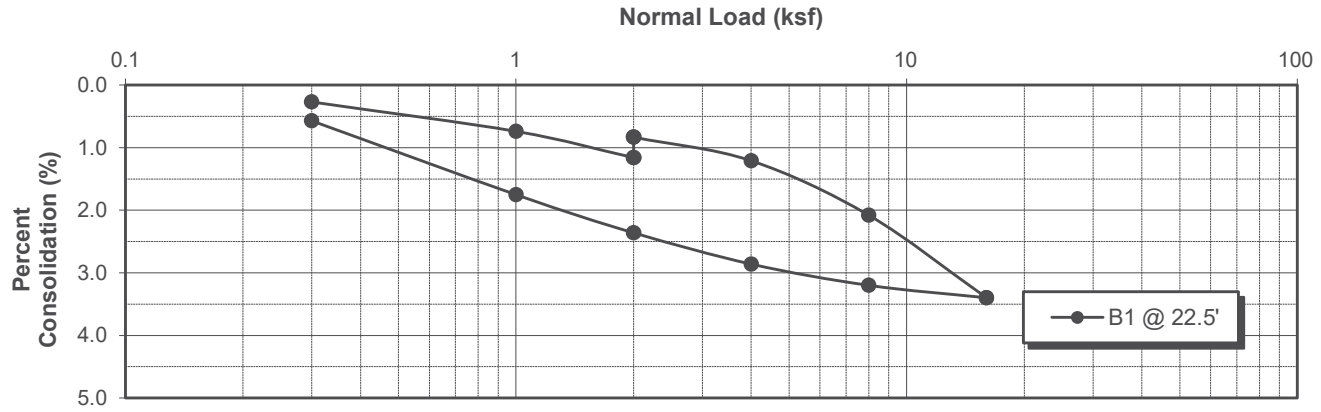
CONSOLIDATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: Torrance Center South Complex, LLC	
FILE NO.: 22600	PLATE: C-2





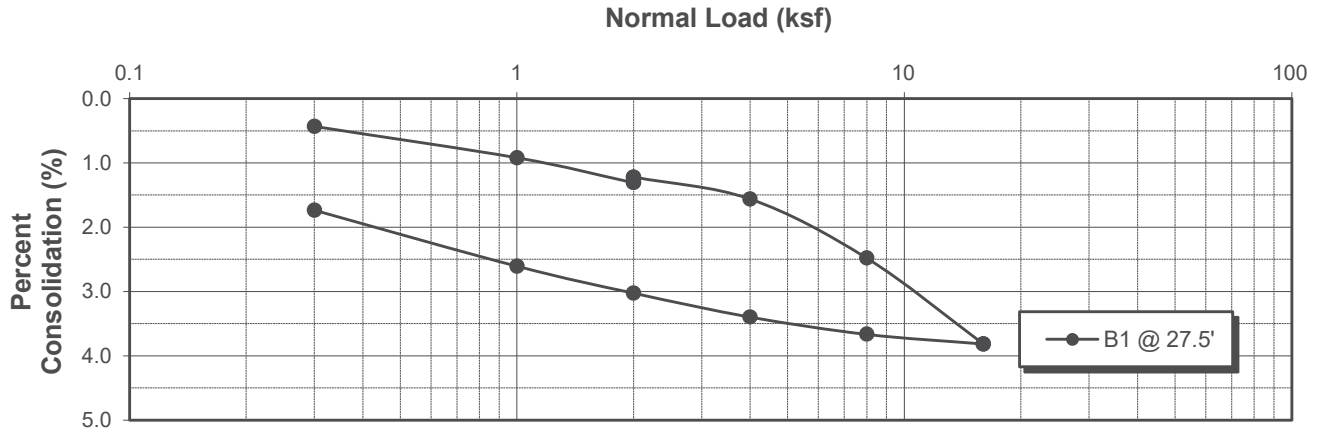


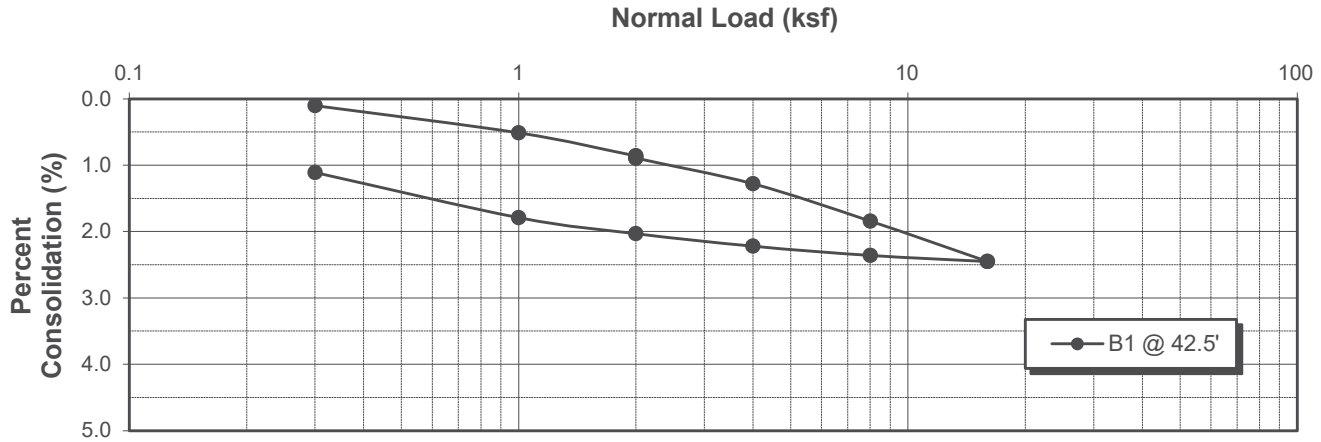
Water added at 2 KSF

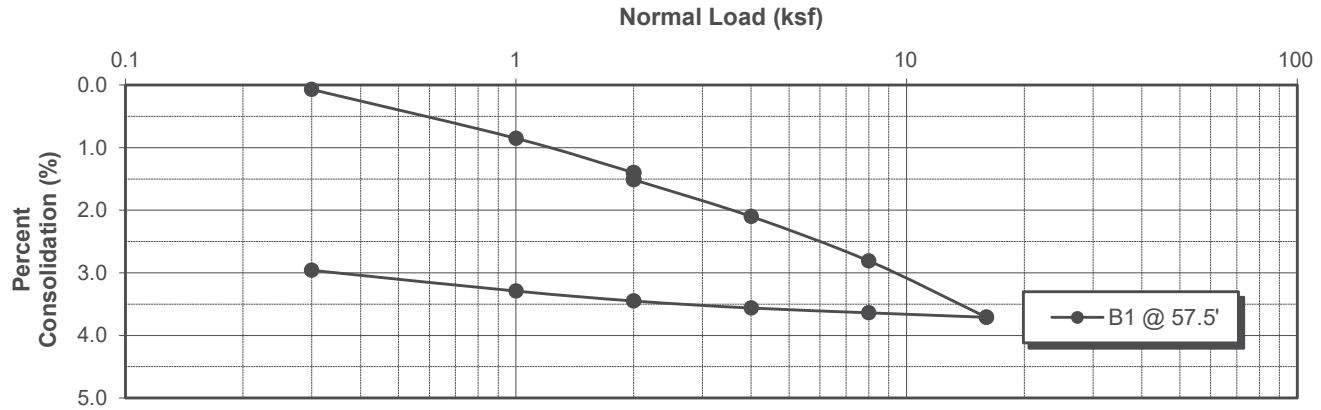
CONSOLIDATION

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PROJECT: Torrance Center South Complex, LLC	
FILE NO.: 22600	PLATE: C-3







Water added at 2 KSF

CONSOLIDATION

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Consulting Geotechnical Engineers

PROJECT: Torrance Center South Complex, LLC	
FILE NO.: 22600	PLATE: C-4

LABORATORY COMPACTION CHARACTERISTICS
(ASTM D1557)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'	B3 @ 1'-5'	B4 @ 1'-5'
SOIL TYPE	ML/CL	ML/CL	ML/CL	SM
MAXIMUM DRY DENSITY (PCF)	122.6	121.7	120.9	125.2
OPTIMUM MOISTURE %	12.7	10.4	11.9	10.0

EXPANSION INDEX
(ASTM D4829)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'	B3 @ 1'-5'	B4 @ 1'-5'
SOIL TYPE	ML/CL	ML/CL	ML/CL	SM
EXPANSION INDEX UBC STANDARD 18-2	58	10	13	3
EXPANSION CHARACTER	MODERATE	VERY LOW	VERY LOW	VERY LOW

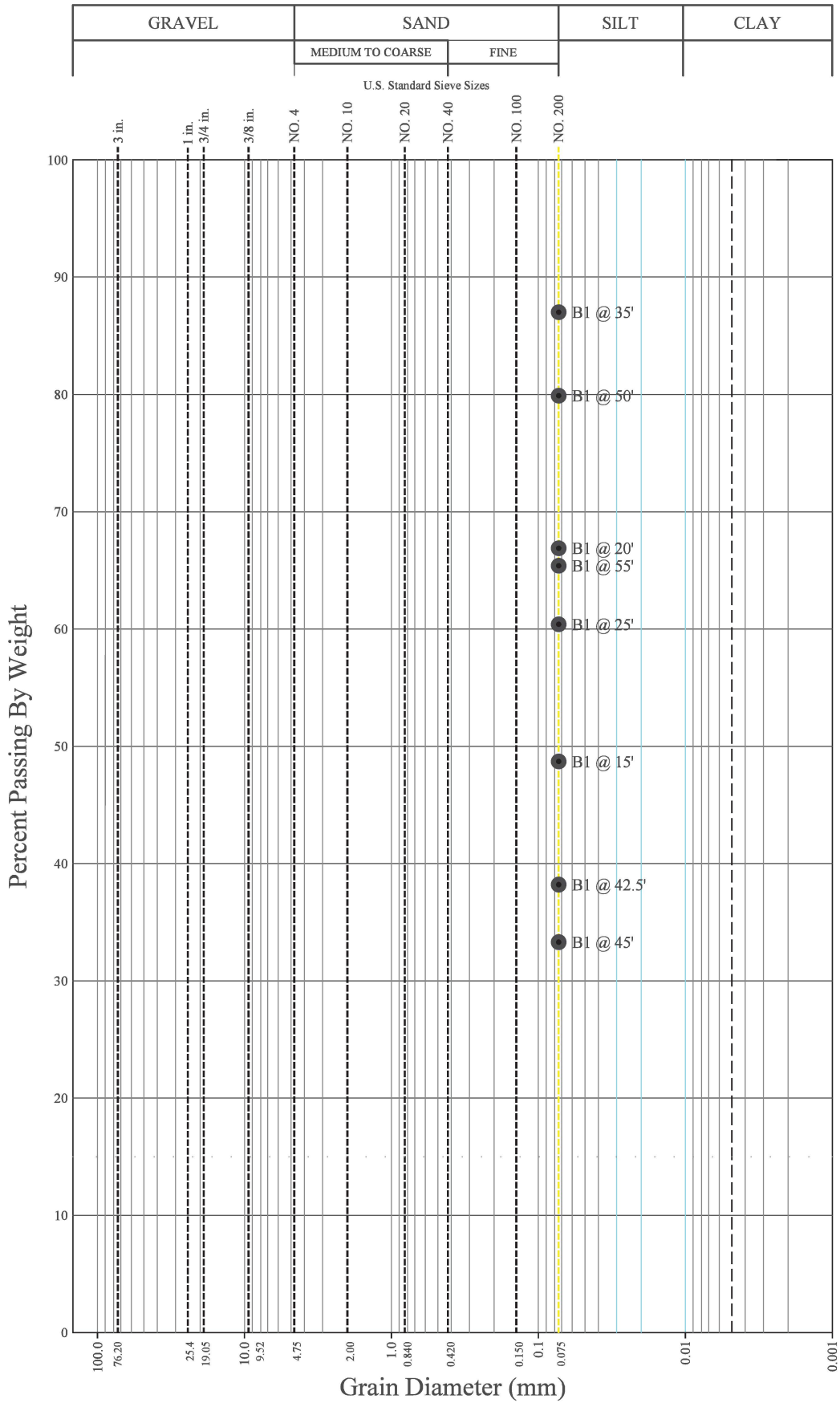


COMPACTION/EXPANSION DATA SHEET

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Consulting Geotechnical Engineers

TORRANCE CENTER SOUTH COMPLEX, LLC
21151, 21171, 21221, 21241, 21281 S WESTERN AVE., TORRANCE

FILE NO: 22600 | PLATE: D



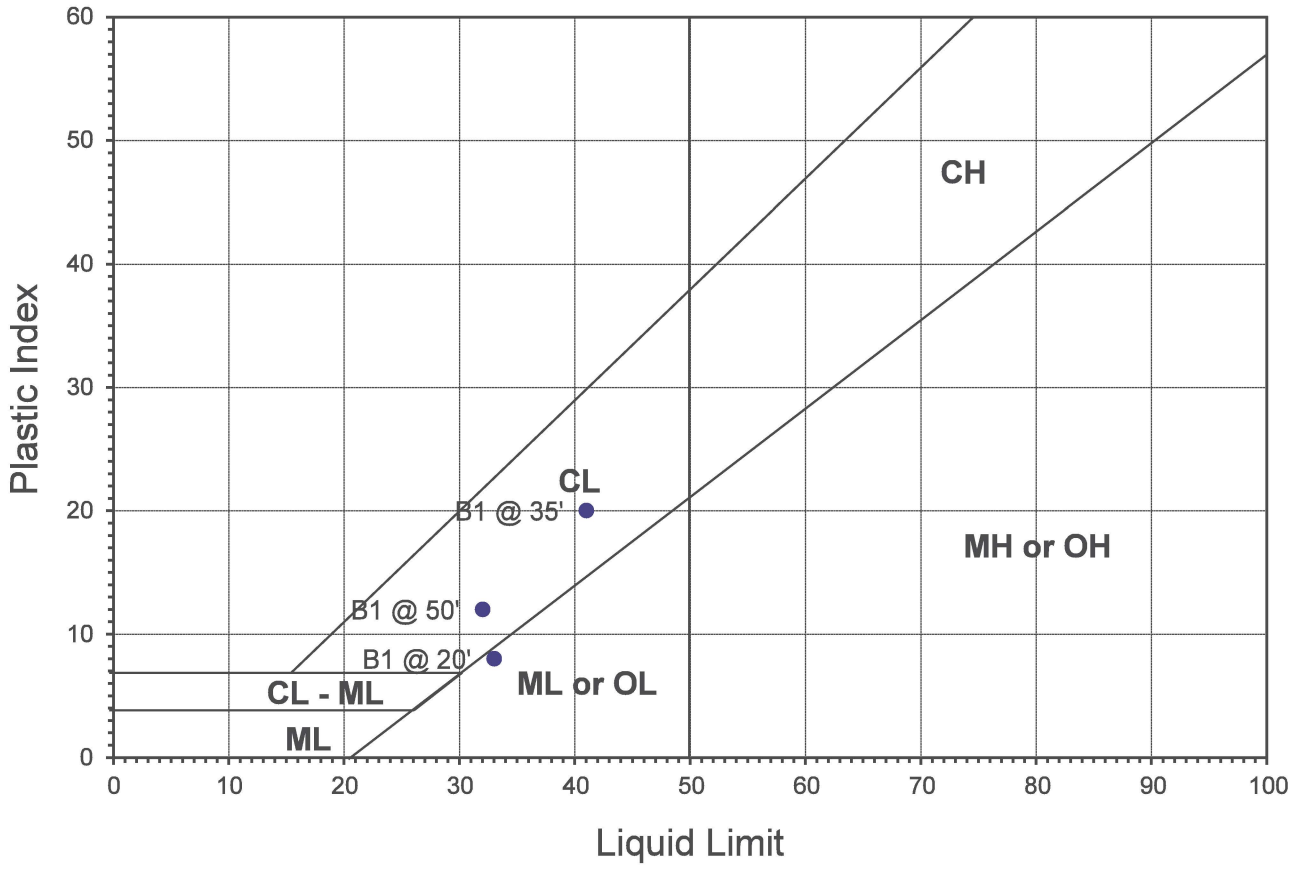
GRAIN SIZE DISTRIBUTION (ASTM D1140)

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TORRANCE CENTER SOUTH COMPLEX, LLC
21151, 21171, 21221, 21241, 21281 S WESTERN AVE., TORRANCE

FILE NO: 22600

PLATE: E



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B1 @ 20'	ML	66.9	33.0	25.0	8.0
B1 @ 35'	CL	87.0	41.0	21.0	20.0
B1 @ 50'	CL	79.9	32.0	20.0	12.0

ATTERBERG LIMITS (ASTM D4318)

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: TORRANCE CENTER SOUTH COMPLEX, LLC

FILE NO.: 22600

PLATE: F

Geotechnologies, Inc.

Project: Torrance Center South Complex
File No.: 22600
Description: Liquefaction Analysis
Boring Numb(1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	7.3
Peak Ground Horizontal Acceleration, PGA (g):	0.84
Calculated Mag.Wtg.Factor:	1.054

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	55.0
Historically Highest Groundwater Level* (ft):	20.0
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 $(N_1)_{60\text{-}cs}$	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	126.8	Unsaturated	Unsaturated	15	5	0.0	0	126.8	126.8	35.1	1.00	0.551	1.299	Non-Liq.	0.00
2	126.8	Unsaturated	Unsaturated	15	5	0.0	0	253.6	253.6	35.1	1.00	0.550	1.299	Non-Liq.	0.00
3	126.8	Unsaturated	Unsaturated	15	5	0.0	0	380.4	380.4	35.1	1.00	0.548	1.299	Non-Liq.	0.00
4	126.8	Unsaturated	Unsaturated	15	5	0.0	0	507.2	507.2	35.1	1.00	0.547	1.299	Non-Liq.	0.00
5	126.8	Unsaturated	Unsaturated	15	5	0.0	0	634.0	634.0	37.0	0.99	0.545	2.000	Non-Liq.	0.00
6	126.8	Unsaturated	Unsaturated	15	5	0.0	0	760.8	760.8	34.1	0.99	0.544	1.067	Non-Liq.	0.00
7	126.8	Unsaturated	Unsaturated	15	5	0.0	0	887.6	887.6	31.5	0.99	0.542	0.691	Non-Liq.	0.00
8	126.3	Unsaturated	Unsaturated	15	5	0.0	0	1013.9	1013.9	29.4	0.98	0.540	0.523	Non-Liq.	0.00
9	126.3	Unsaturated	Unsaturated	15	5	0.0	0	1140.2	1140.2	29.5	0.98	0.539	0.531	Non-Liq.	0.00
10	126.3	Unsaturated	Unsaturated	15	5	0.0	0	1266.5	1266.5	28.0	0.98	0.537	0.444	Non-Liq.	0.00
11	129.1	Unsaturated	Unsaturated	26	10	0.0	0	1395.6	1395.6	46.6	0.97	0.535	2.000	Non-Liq.	0.00
12	129.1	Unsaturated	Unsaturated	26	10	0.0	0	1524.7	1524.7	45.4	0.97	0.533	2.000	Non-Liq.	0.00
13	129.1	Unsaturated	Unsaturated	26	10	0.0	0	1653.8	1653.8	44.3	0.97	0.531	2.000	Non-Liq.	0.00
14	129.1	Unsaturated	Unsaturated	26	10	0.0	0	1782.9	1782.9	43.4	0.96	0.529	2.000	Non-Liq.	0.00
15	129.1	Unsaturated	Unsaturated	26	10	0.0	0	1912.0	1912.0	47.4	0.96	0.527	2.000	Non-Liq.	0.00
16	127.8	Unsaturated	Unsaturated	22	15	48.7	0	2039.8	2039.8	45.1	0.96	0.525	2.000	Non-Liq.	0.00
17	127.8	Unsaturated	Unsaturated	22	15	48.7	0	2167.6	2167.6	44.3	0.95	0.523	2.000	Non-Liq.	0.00
18	127.8	Unsaturated	Unsaturated	22	15	48.7	0	2295.4	2295.4	43.4	0.95	0.520	2.000	Non-Liq.	0.00
19	127.8	Unsaturated	Unsaturated	22	15	48.7	0	2423.2	2423.2	42.6	0.94	0.518	2.000	Non-Liq.	0.00
20	127.8	Unsaturated	Unsaturated	22	15	48.7	0	2551.0	2551.0	41.8	0.94	0.516	1.991	Non-Liq.	0.00
21	128.1	Unsaturated	Saturated	20	20	66.9	8	2679.1	2616.7	37.3	0.94	0.526	1.865	3.5	0.00
22	128.1	Unsaturated	Saturated	20	20	66.9	8	2807.2	2682.4	37.0	0.93	0.535	1.701	3.2	0.00
23	128.1	Unsaturated	Saturated	20	20	66.9	8	2935.3	2748.1	36.6	0.93	0.544	1.561	2.9	0.00
24	128.1	Unsaturated	Saturated	20	20	66.9	8	3063.4	2813.8	36.3	0.92	0.551	1.441	2.6	0.00
25	128.1	Unsaturated	Saturated	20	20	66.9	8	3191.5	2879.5	36.0	0.92	0.559	1.336	2.4	0.00
26	121.7	Unsaturated	Saturated	54	25	60.4	0	3313.2	2938.8	93.5	0.91	0.566	1.902	3.4	0.00
27	121.7	Unsaturated	Saturated	54	25	60.4	0	3434.9	2998.1	93.0	0.91	0.572	1.890	3.3	0.00
28	121.7	Unsaturated	Saturated	54	25	60.4	0	3556.6	3057.4	97.1	0.91	0.578	1.878	3.3	0.00
29	121.7	Unsaturated	Saturated	54	25	60.4	0	3678.3	3116.7	96.7	0.90	0.583	1.866	3.2	0.00
30	121.7	Unsaturated	Saturated	54	25	60.4	0	3800.0	3176.0	96.2	0.90	0.588	1.854	3.2	0.00
31	121.7	Unsaturated	Saturated	24	30	60.4	0	3921.7	3235.3	45.2	0.89	0.593	1.843	3.1	0.00
32	121.7	Unsaturated	Saturated	24	30	60.4	0	4043.4	3294.6	44.9	0.89	0.597	1.831	3.1	0.00
33	121.7	Unsaturated	Saturated	24	30	60.4	0	4165.1	3353.9	44.7	0.88	0.601	1.820	3.0	0.00
34	126.0	Unsaturated	Saturated	24	30	0.0	0	4291.1	3417.5	38.8	0.88	0.604	1.809	3.0	0.00
35	126.0	Unsaturated	Saturated	24	30	0.0	0	4417.1	3481.1	38.5	0.87	0.607	1.797	3.0	0.00
36	122.9	Unsaturated	Saturated	21	35	87.0	20	4540.0	3541.6	37.4	0.87	0.610	1.717	Non-Liq.	0.00
37	122.9	Unsaturated	Saturated	21	35	87.0	20	4662.9	3602.1	37.2	0.86	0.612	1.614	Non-Liq.	0.00
38	122.9	Unsaturated	Saturated	21	35	87.0	20	4785.8	3662.6	36.9	0.86	0.615	1.522	Non-Liq.	0.00
39	122.9	Unsaturated	Saturated	21	35	87.0	20	4908.7	3723.1	36.7	0.85	0.616	1.441	Non-Liq.	0.00
40	122.9	Unsaturated	Saturated	21	35	87.0	20	5031.6	3783.6	36.5	0.85	0.618	1.367	Non-Liq.	0.00
41	130.1	Unsaturated	Saturated	24	40	38.2	0	5161.7	3851.3	42.7	0.84	0.619	1.734	2.8	0.00
42	130.1	Unsaturated	Saturated	24	40	38.2	0	5291.8	3919.0	42.4	0.84	0.620	1.723	2.8	0.00
43	130.1	Unsaturated	Saturated	24	40	38.2	0	5421.9	3986.7	42.2	0.83	0.621	1.713	2.8	0.00
44	130.1	Unsaturated	Saturated	24	40	38.2	0	5552.0	4054.4	42.0	0.83	0.622	1.702	2.7	0.00
45	130.1	Unsaturated	Saturated	24	40	38.2	0	5682.1	4122.1	41.7	0.82	0.622	1.692	2.7	0.00
46	130.1	Unsaturated	Saturated	28	45	33.3	0	5812.2	4189.8	49.1	0.82	0.622	1.682	2.7	0.00
47	130.1	Unsaturated	Saturated	28	45	33.3	0	5942.3	4257.5	49.0	0.81	0.622	1.672	2.7	0.00
48	131.3	Unsaturated	Saturated	28	45	33.3	0	6073.6	4326.4	48.8	0.81	0.622	1.662	2.7	0.00
49	131.3	Unsaturated	Saturated	28	45	33.3	0	6204.9	4395.3	48.6	0.80	0.622	1.652	2.7	0.00
50	131.3	Unsaturated	Saturated	28	45	33.3	0	6336.2	4464.2	48.4	0.80	0.621	1.642	2.6	0.00
51	125.3	Unsaturated	Saturated	26	50	79.9	12	6461.5	4527.1	44.6	0.79	0.621	1.634	2.6	0.00
52	125.3	Unsaturated	Saturated	26	50	79.9	12	6586.8	4590.0	44.4	0.79	0.620	1.625	2.6	0.00
53	125.3	Unsaturated	Saturated	26	50	79.9	12	6712.1	4652.9	44.2	0.78	0.620	1.617	2.6	0.00
54	125.3	Unsaturated	Saturated	26	50	79.9	12	6837.4	4715.8	44.0	0.78	0.619	1.608	2.6	0.00
55	125.3	Unsaturated	Saturated	26	50	79.9	12	6962.7	4778.7	43.9	0.77	0.618	1.600	2.6	0.00
56	126.6	Saturated	Saturated	25	55	65.4	0	7089.3	4842.9	41.8	0.77	0.617	1.592	2.6	0.00
57	126.6	Saturated	Saturated	25	55	65.4	0	7215.9	4907.1	41.6	0.76	0.616	1.583	2.6	0.00
58	126.6	Saturated	Saturated	25	55	65.4	0	7342.5	4971.3	41.4	0.76	0.615	1.575	2.6	0.00
59	126.6	Saturated	Saturated	25	55	65.4	0	7469.1	5035.5	41.3	0.75	0.614	1.567	2.6	0.00
60	126.6	Saturated	Saturated	25	55	65.4	0	7595.7	5099.7	41.1	0.75	0.613	1.559	2.5	0.00
											Total Liquefaction Settlement, S =				**0.00 inches**

February 3, 2025 via email: vdunaev@geoteq.com

Geotechnologies, Inc.
439 Western Ave.
Glendale, CA, 91201

Attention: Vasily Dunaev

Re: Soil Corrosivity Study
Torrance Center South Complex, LLC
Torrance, CA
HDR #25-0025SCS, Geotechnologies #22600

Introduction

Laboratory tests have been completed on three soil samples provided to HDR for the Torrance Center South Complex, LLC project. The purpose of these tests was to determine whether the soils are likely to have deleterious effects on underground utility piping, hydraulic elevator cylinders, concrete structures, and post tensioning systems. HDR assumes that the provided samples are representative of the most corrosive soils at the site.

The proposed structure is a residential complex with up to seven stories and one subterranean level. The complex is located at 21151, 21171, 21221, 21241, and 21281 South Western Avenue in Torrance, California, and the water table is reportedly 57.5 feet deep.

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.

Soil Corrosivity Testing

Laboratory Testing

The electrical resistivity of each sample was measured in a soil box per *ASTM International (ASTM)* G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and *American Water Works Association (AWWA*) Standard Method 2320-B.

The laboratory analyses were performed under HDR laboratory number 25-0025SCS. The full set of test results are shown in the attached Table A1.

Discussion

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 shown in Table 1.[1]

Table 1: Soil Corrosivity Categories

Soil Resistivity (ohm-cm)	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 resistivities were in the mildly corrosive and moderately corrosive categories with as-received moisture. When saturated, the resistivities were in the moderately corrosive to corrosive categories.

Soil pH values varied from 7.7 to 8.5. This range is mildly to strongly alkaline.[2]

Soil with a pH greater than or equal to 8.5 is considered aggressive to aluminum.

The soluble salt content of the samples ranged from low to moderate.

Per ACI-318, the soil is classified as negligible (S0) with respect to sulfate concentration.[3]

Nitrate was detected in low concentrations. Ammonium was not detected.

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

In conclusion, this soil is classified as corrosive to ferrous metals, aggressive to aluminum, and negligible (S0) for sulfate attack on concrete.

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

[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*
[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*
[3] *American Concrete Institute (ACI) 318-19 Table 19.3.1.1.*

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 evaluation of soil corrosivity described above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

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.

3. To prevent differential aeration corrosion cells, provide at least 2 inches of pipe bedding or backfill material all around metallic piping, including the bottom. Do not lay pipe directly on undisturbed soil.

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 cathodic protection.

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

    a. At each end of the pipeline.

    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 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.

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

   OPTION 2

   a. As an alternative to the coating systems described in Option 1 and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or 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. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

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

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the 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 cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the 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. Apply cathodic protection to ductile iron piping as per NACE SP0169.

   OPTION 2

   a. As an alternative to the coating systems described in Option 1 and cathodic protection, 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. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

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. Use the following parameters for clean sand backfill:

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

   b. pH between 6.0 and 8.0.

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

Copper Tubing

1. Use Type K or Type L copper tubing as required by the applicable local plumbing code. Type M tubing should not be used for buried applications.[4]

2. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286. Sleeve copper pipe through footings and foundations to prevent pH concentration cells and prevent leaks caused by settlement.

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

4. Protect cold water pipe using all of the following measures:

    a. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves. Ensure that sleeves are intact and free of cuts, tears, punctures, or other damage.

    b. Remove any construction debris, rocks, wood, or organic matter from the trench prior to backfill.

    c. Bed and backfill with at least 2 inches of clean sand all around the tubing, including the bedding. Use the following parameters for clean sand backfill:

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

        ii. pH between 6.0 and 8.0.

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

    d. Copper tubing for cold water can also be treated the same as for hot water.

5. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing using one of the following measures:

    a. Prevent soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. Either seal the PVC pipe at both ends using ammonia- and methanol-free caulk, or terminate both ends above-grade in a manner that doesn't allow water to infiltrate; *or*

    b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing with a suitable dielectric coating that is compatible with cathodic protection, such as Polyken 930.

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[4] *2022 California Plumbing Code (CPC) Section 604.3.*

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 designed cathodic protection system per NACE SP0169.

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 (S0), from 0 to 0.10 percent. Use a minimum strength of 2,500 psi per applicable codes.[5,6,7]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations found on site.[8] Limit the water-soluble chloride ion content in the concrete mix design to less than 0.06 percent by weight of cement.[9]

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

Although chloride levels were relatively low, 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:[10,11,12]

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

2. Design all tendons 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 protective grease cap filled with corrosion protection material that provides a watertight seal for the strand end and wedge cavity. Ensure the cap fully seats against the face of the standard anchor at the live end.

5. Protect all components from moisture prior to installation and within one working day after installation.

6. Ensure the minimum concrete cover over the tendon tail is 1 inch, or greater if required

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[5] 2021 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[6] 2021 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[7] 2022 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[8] Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65
[9] American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[10] Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.
[11] PTI M10.2-17: Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2017.
[12] ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Concrete Institute (ACI), 2001

by the applicable building code.

7. Install caps within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.
8. Install pre-cast concrete plug over the grease cap to ensure the live end is sealed from further moisture intrusion.
9. Limit the access of direct runoff onto the anchorage area by designing proper drainage. Do not allow water to pond against anchors.
10. Provide at least 2 inches of space between finish grade and the anchorage area, or more if required by applicable building codes.

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; and

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

   OPTION 2

   a. 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, and

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

OPTION 2

a. 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.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. 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.



Teddie Algorri
Laboratory Manager

Bradley M. Stuart, PE
Corrosion Engineer

Enc: Table A1

25-0025SCS SCS TA_BS_TA.docx

Table A1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Torrance Center South Complex, LLC
Your #22600, HDR Lab #25-0025SCS
20-Jan-25

Sample ID			B1 @ 1'-5'	B3 @ 1'-5'	B4 @ 1'-5'
Resistivity		**Units**			
as-received		ohm-cm	24,000	19,200	7,600
saturated		ohm-cm	1,280	1,880	2,480
pH			7.7	8.5	7.7
Electrical Conductivity		mS/cm	0.25	0.10	0.12
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	44	14	69
magnesium	Mg^{2+}	mg/kg	12	3.0	8.0
sodium	Na^{1+}	mg/kg	264	107	38
potassium	K^{1+}	mg/kg	6.0	1.0	7.0
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	515	156	153
fluoride	F^{1-}	mg/kg	29	6.0	8.0
chloride	Cl^{1-}	mg/kg	24	14	5.0
sulfate	SO_4^{2-}	mg/kg	113	67	132
nitrate	NO_3^{1-}	mg/kg	1.0	ND	1.0
phosphate	PO_4^{3-}	mg/kg	4.0	7.0	4.0
Other Tests					
% moisture	H_2O	%	na	na	na
total acidity	H^{1+}	mmol/kg	na	na	na
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Resistivity per ASTM G187, pH per ASTM G51, 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

CITY OF TORRANCE
BUILDING & SAFETY DEPARTMENT
3031 Torrance Blvd.
Torrance, CA 90503 (310) 618-5910
For inspections call (310) 618-5915

GRADING PERMIT
Permit #..: **GRD05-00033**
Issued.....: 02/08/2006
Issued by: **IBL**
Date sub.: 06/16/2005

Job Address **21281 WESTERN AVE**
LOT BLOCK TRACT
4 PM 20505

APN.....: 7355-032-012
Zone.....: **IRP**

Owner: ---------- : ----------

TTC OFFICE PARK LLC **PH#1:**
23532 PASEO DE VALENCIA STI **PH#2:**
LAGUNA HILLS CA 92653 **PH#3:**

Project Description: SBC: GRADING FOR NEW EQUIP ENCLOSURE IN PKG LOT, 9' HI (850 SQ FT) W/ CHAIN LINK ON TOP.

OWNER / BUILDER DECLARATION
I hereby affirm under penalty of perjury that I am exempt from the Contractors License Law for the following reason:
( ) I, as owner of the property, or my employees with wages as their sole compensation, will do the work, and the structure is intended or offered for sale.
( ) I, as owner of the property, am exclusively contracting with licensed contractors to construct the project (Section 704 of the Business and Professions Code).
Owner: _____ Date: _____

LICENSED CONTRACTOR'S DECLARATION
I hereby affirm under penalty of perjury that I am licensed under the provisions of Chapter 9 (commencing at Sec. 7000) of Division 3 of the Business and Professions Code, and my license is in full force and effect.
H C OLSEN CONSTRUCTION CO INC
710 E LOS ANGELES AVENUE
MONROVIA CA 91016
PHONE: 626-359-8900
LIC#: 177029 EXP: 5/31/2006 CLASS: B
Signature: _____ Date: 2/8/06

WORKER'S COMPENSATION DECLARATION
I hereby affirm under penalty of perjury one of the following per Sec. 3700 of the Labor Code:
( ) I have and will maintain workers' comp. ins., or a certificate of consent to self insure for workers' comp. for the performance of the work for which the permit is issued.
Policy#: WCS4376471-01 Exp Date: 4/1/2006
Company: AMERICAN GUARANTEE
( ) I certify that in the performance of work for which the permit is issued, I shall not employ any person in any manner so as to become subject to the workers' comp. laws of CA, and agree that if I should become subject to the workers' comp. provisions of Section 3700 of the Labor Code, I shall forthwith comply with those provisions. Sig: _____ Date: _____

CONSTRUCTION LENDING AGENCY
I hereby affirm that there is a construction Lending agency for the performance of this work for which this permit is issued (Section 3097 California Civil code).
Lender's name: _____
Address: _____

GRADING

MAX DEPTH OF FILL..:	1	ft	PAVING AREA..:	0	sf		
MAX DEPTH OF CUT...:	0	ft	PUMP SIZE.....:	0	gpm	ISSUED...:	02/08/2006
MATL HANDLED-FILL:	10	cy	SOIL TYPE.....:			EXPIRES..:	08/07/2006
MATL HANDED-CUT.:	0	cy	EXPAN INDEX (EI).:				

NO GRADING WORK SHALL BE DONE WITHOUT NOTIFYING THE GRADING INSPECTOR.

PRE-JOB MEETING: _____

APPROVALS	DATE	SIGNATURE
SUBGRADE		
BASE		
PAVING		
STORM DRAIN/SWALE		
H/C SPACES/ACCESS		
SUMP PUMP		
FINAL COMPACION REPORT		
ENGINEERS CERTIFICATION		
FINAL	2/10/06	

CONFIDENTIAL

RIGHT OF ENTRY: I certify that I have read this application and state that the above information is correct. I agree to comply with all City and applicable County ordinances, and State laws relating to building construction and hereby authorize representatives of the City to enter upon the above mentioned property for the purposes of inspections.

Signature: _____ Date: 2/8/06

ANY PERMIT ISSUED AS A RESULT OF THIS APPLICATION BECOMES NULL AND VOID IF WORK IS NOT COMMENCED WITHIN 180 DAYS FROM DATE OF ISSUANCE OF SUCH PERMIT OR FROM THE DATE WORK IS SUSPENDED OR ABANDONED. CLAIMS FOR REFUNDS OF ANY FEES MUST BE FILED WITHIN 100 CALENDAR DAYS FROM THE DATE COLLECTED BY THE CITY.

CIRCLE - 1) Inspector 2) File 3) Applicant

REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED RELOCATE/REPLACE COOLING TOWER

EXISTING SBC FACILITY
21281 WESTERN AVENUE
TORRANCE, CALIFORNIA

Prepared for:

SBC

Torrance, California

August 4, 2005

Project 4953-05-2021

MACTEC

MACTEC

engineering and constructing a better tomorrow

August 4, 2005

Mr. John T. Chan, President
JTC Architects, Inc.
65 North First Avenue, Suite 201
Arcadia, California 91006

Subject: **Report of Geotechnical Investigation**
Proposed Relocate/Replace Cooling Tower
Existing SBC Facility
21281 Western Avenue
Torrance, California
JTC Project No. 50302.0B
MACTEC Project 4953-05-2021

Dear Mr. Chan:

We are pleased to submit this report presenting the results of our geotechnical investigation for the proposed relocate/replace a cooling tower at the subject SBC facility in Torrance, California. Our investigation was conducted in general accordance with our proposal dated July 6, 2005, as authorized by you on July 10, 2005.

The scope of our services was planned with you. We were advised of the structural features of the proposed project by Mr. John Chan, structural engineer and Mr. Ferdinand Mata of your office.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.

MACTEC Engineering and Consulting, Inc.
200 Citadel Drive • Los Angeles, CA 90040 • Phone: 323.889.5300 • Fax: 323.721.6700
www.mactec.com

It has been a pleasure to be of professional service to you. Please call if you have any questions or if we can be of further assistance.

Sincerely,

MACTEC Engineering and Consulting, Inc.



Mike Shahabi
Senior Engineer
Project Manager

Jake Kharraz
Principal Engineer

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(6 copies submitted)

REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED RELOCATED/REPLACE COOLING TOWER

EXISTING SBC FACILITY
21281 WESTERN AVENUE
TORRANCE, CALIFORNIA

Prepared for:

SBC

Torrance, California

MACTEC, Engineering and Consulting, Inc.

Los Angeles, California

August 4, 2005

Project 4953-05-2021

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1.0 SCOPE

This report presents the results of our geotechnical investigation performed for construction of a new cooling tower to service Building A only within the SBC facility in Torrance, California. An existing cooling tower currently services Building A and adjacent Building B. The location of the proposed cooling tower relative to the existing structures and streets, and the location of our exploration boring are shown on Figure 1, Plot Plan.

This investigation was authorized to determine the static physical characteristics of the soils at a selected location, and to provide foundation design recommendations for the proposed cooling tower structure. More specifically, the scope of this investigation included the following objectives:

- Results of the subsurface exploration and laboratory tests, with a description of the soil and groundwater conditions encountered within the area of proposed construction.
- Evaluation of the liquefaction potential of the soils underlying the site.
- Recommendations for design of foundations to be used for support of the proposed cooling tower, including allowable increases for wind or seismic loads.
- Estimated settlements for the anticipated loadings.
- Recommendations regarding frictional and passive values for the resistance of lateral forces.
- Determination of the applicable site coefficient based on the Uniform Building Code.
- Recommendations for floor slab support.
- Recommendations for earthwork, including site preparation, excavation, and the placing of any required compacted fill.

The scope of this investigation did not include geologic or seismic studies for the site. Accordingly, our conclusions and recommendations are for static loading conditions only; however, this does not imply that there is a geologic or seismic hazard affecting the site. Also, the

assessment of general site environmental conditions for the presence of contaminants in the soils and groundwater of the site was beyond the scope of this investigation.

Our recommendations are based on the results of our field exploration, laboratory tests, and appropriate engineering analyses. The results of the field exploration and laboratory tests, which form the basis of our recommendations, are presented in this report.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for SBC and their design consultants to be used solely in the design of the proposed new cooling tower. The report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses.

2.0 STRUCTURAL CONSIDERATIONS

It planned to construct a new cooling to service Building A at the existing SBC facilities in Torrance, California. As shown in Plan on Figure 1, the cooling tower will be constructed at the parking lot to the north of Building A. The cooling tower will be enclosed by a perimeter wall.

Based on the plans and information provided to us, the cooling tower unit will weight about 8,000 pounds and will be supported on a reinforced pad about 16 feet long, 6½ feet wide, and about 8 inches thick. The reinforced pad and will act as a mat type foundation. The cooling tower structure will be bolted to the mat. A slab, 4 inches thick will be constructed around the mat. The mat and slab will be about 15 by 30 feet in plan dimensions. The mat will impose a dead-plus-live pressure of less than 500 pounds per square foot.

The cooling tower slab will be enclosed by a 9-foot high and about 8½-inch thick concrete block wall. The enclosure wall will be supported on a continuous spread footing extending about 18 inches below the top of the slab surrounding the mat or below the adjacent asphaltic paving. The wall loads will be about 1,000 pounds per lineal foot.

3.0 FIELD EXPLORATION AND LABORATORY TEST RESULTS

3.1 FIELD EXPLORATION

The soil and ground-water conditions beneath the site of the proposed cooling tower was explored by drilling one boring at the location shown on Figure 1. The boring was drilled to a depth of 30 feet below the existing grade using 16-inch-diameter bucket-type drilling equipment. Caving and raveling of the boring walls did not occur during drilling; casing or drilling mud was not used to extend the boring to the depth drilled.

The soils encountered were logged by our field technician, and undisturbed and bulk samples were obtained for laboratory inspection and testing. The log of the boring is presented on Figure 2; the depths at which undisturbed samples were obtained are indicated to the left of the boring log. The number of blows required to drive the sampler 12 inches and the hammer weight and drop are indicated on the boring log. The soils are classified in accordance with the Unified Soil Classification System described on Figure 3.

3.2 LABORATORY TEST RESULTS

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left of the boring log.

To determine the strength of the soils encountered, direct shear tests were performed on selected undisturbed samples. The tests were performed at field moisture content and after soaking to near-saturated moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented on Figure 4, Direct Shear Test Data.

Confined consolidation tests were performed on two undisturbed samples to determine the compressibility of the soils. The samples were tested at field moisture content. The results of the tests are presented on Figure 5, Consolidation Test Data.

The optimum moisture content and maximum dry density of the upper soils were determined by performing a compaction test on a sample obtained from the boring. The test was performed in

accordance with the ASTM Designation D155 method of compaction. The results of the test are presented on Figure 6, Compaction Test Data.

The Expansion Index of the upper clay soils was determined in accordance with the ASTM Designation D4829 method. The results of the test are shown in Figure 7, Expansion Index Test Data.

To provide data for paving design, a stabilometer (R-value) test was performed on a sample of the upper soils. The results of the test, which indicated an R-value of less than 5, are presented on Figures 8.1 and 8.2.

4.0 SITE AND SOIL CONDITIONS

The site of proposed building is currently paved and used for parking. Underground utility lines may cross the site.

Fill soils, about 2 feet thick, were encountered in the exploration boring. The fill soils consist of clayey sand and silty clay and are not uniformly well compacted. Deeper and/or poorer quality fill could occur elsewhere at the site.

The natural soils beneath the site to the depth explored by our boring consist primarily of cohesive silty clay underlain by granular silty sand deposits. The cohesive clay soils are generally stiff and the granular silty sand deposits are dense. The upper silty clay soils have high expansion potential and would swell and shrink within changes in the moisture content. In addition, the upper clay soils underlying the site are at a moisture content that is higher than their optimum moisture content.

Ground water was not encountered within the 30-foot deep boring drilled.

5.0 LIQUEFACTION POTENTIAL

Liquefaction potential is greatest where the ground water level is shallow, and loose granular sandy soils occur within a depth of about 50 feet. Liquefaction potential decreases as grain size

and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

Based on the State of California Seismic Hazard Zones (Torrance Quadrangle Official Map, March 29, 1999), the subject site is not in a liquefiable zone. Therefore, the potential for liquefaction and the associated ground deformation beneath the site is considered to be very low.

6.0 RECOMMENDATIONS

6.1 FOUNDATIONS

General

The existing fill soils at the site are not suitable for support of the proposed cooling tower structure. The upper natural soils consist generally of cohesive stiff clay and the proposed cooling tower unit may be supported on a mat-type foundation and the enclosure wall may be supported on shallow spread footing, as planned. If the grading recommendations presented in the report are followed the footing and the mat may be established in properly compacted and/or natural soils.

The floor slab surrounding the mat may be supported on grade. However, as previously stated, the upper clay soils have relatively high expansion potential; therefore, we recommend a layer of relatively non-expansive soils be placed beneath the mat and the surrounding slab. The layer of non-expansive soils will not be necessary beneath the footing supporting the enclosure wall or beneath the areas to be paved with asphalt and concrete paving.

Recommendations for grading are presented in a following section. The reworking of the upper soils and the compaction of all required fill should be observed by a representative of our firm.

Bearing Values

The spread footing for the wall enclosure established in either properly compacted fill and/or the undisturbed natural soils, may be designed to impose a net dead-plus-live load pressure of up to 1,000 pounds per square foot. The mat-type foundation to support the cooling tower unit,

established in properly compacted fill and/or the undisturbed natural soils may be designed to impose a net dead-plus-live load pressure of up to 750 pounds per square foot. Spread footings should extend at least 1½ feet below the top of the slab or the adjacent asphaltic paving. The mat should also be structurally separated from the surrounding slab.

A one-third increase in the bearing values may be used for any wind or seismic loads. When determining the downward loads on the foundations, the weight of concrete in the foundations below grade may be taken as 50 pounds per cubic foot, and the weight of soil backfill may be neglected.

While the actual bearing value of the compacted fill will depend on the materials used and the compaction methods employed, the quoted bearing values will be applicable if acceptable soils are used and are compacted as recommended. The bearing value of the compacted fill should be confirmed during grading.

Settlement

We estimate the settlement of the wall enclosure supported on a spread footing in the manner recommended, will be about ½ inch or less. The settlement of the mat foundation supporting the cooling tower unit is estimated to be about ½ inch. Differential settlement between the mat and the adjacent footing is expected to be ¼ inch or less.

Lateral Loads

Lateral loads may be resisted by soil friction against the footing, the mat foundation, and the slab, and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used between the spread footing and the mat and the slab, and the supporting soils. The passive resistance of the undisturbed natural soils or properly compacted fill against foundations may be assumed to be 250 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads.

The passive resistance of the soils and the frictional resistance between the foundations and the slab, and the supporting soils may be combined without reduction in determining the total lateral resistance.

Foundation Observation

To verify the presence of satisfactory soils at design elevations, all footing excavations should be observed by personnel of our firm. Footings should be deepened as necessary to reach satisfactory supporting soils. The sides of foundation excavations deeper than 4 feet should be sloped back at 1:1 or shored for safety. Adjacent to any existing structure or underground utilities, the bottom of any unshored excavation should be restricted so as not to extend below a plane drawn downward at 1½:1 (horizontal to vertical) from the foundation bottoms of the existing structure or underground utilities.

Backfill around and over footings and utility trench backfill within the cooling tower area should be mechanically compacted; flooding should not be permitted.

Inspection of the foundation excavations may also be required by the appropriate reviewing governmental agencies. The contractor should be familiar with the inspection requirements of the reviewing agencies.

6.2 SITE COEFFICIENT AND SEISMIC ZONATION

The site coefficient, S, can be determined as established in the Earthquake Regulations under Section 1629 of the 2001 Edition of the California Building Code (CBC) for seismic design of the proposed cooling tower. Based on a review of the local soil and geologic conditions, the site may be classified as Soil Profile S_D, as specified in the 2001 CBC. The site is located within CBC Seismic Zone 4.

The site is near the Palos Verdes fault, which has been determined to be a Type B seismic source by the California Division of Mines and Geology. The proposed cooling tower is located at a distance of about 5 kilometers from the Palos Verdes fault according to Map M-33 (ICBO, 1998).

At this distance for a seismic source Type B, the near source factors, N_a and N_v, are to be taken as 1.05 and 1.27, respectively, according to Tables 16-S and 16-AR of the 2001 CBC.

6.3 MAT AND SLAB SUPPORTS

If the subgrade is prepared as recommended in the following section on grading, the mat supporting the cooling tower unit and the slab surrounding the mat may be supported on grade. As previously stated, the upper on-site clay soil has a high expansion potential. Therefore we recommend that the upper clay soils be removed such that the mat and the surrounding slab are underlain by at least 1½ feet of relatively no-expansive soils with an Expansion Index of less than 35. The non-expansive soils will not be required beneath the spread footing and beneath the asphaltic or concrete paving.

Since construction activities and exposure to the environment may cause deterioration of a prepared subgrade, we recommend that our field representative observe the condition of the final subgrade soils immediately prior to mat and slab construction and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

6.4 PAVING

General

To provide support for paving, the subgrade soils should be prepared as recommended in Section 6.5. Compaction of the subgrade to at least 90%, including trench backfills, will be important for paving support. The preparation of the parking area subgrade should be done immediately prior to the placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

To provide information for paving design, stabilometer (R-value) test was performed on a sample of the upper soils. The results of the test, which indicated an R-value of less than 5, are presented on Figures 8.1 and 8.2.

Asphaltic Paving

The required asphaltic paving and base course thickness will depend on the anticipated wheel loads and volume of traffic (Traffic Index). The recommended paving sections for a range of Traffic Indices and type of traffic are presented below. The paving sections were determined using the Caltrans design method.

Assumed Traffic Index	Asphaltic Paving (Inches)	Base Course (Inches)
4 (Automobile Parking)	3	6
5 (Driveways with Light Truck Traffic)	4	7
6 (Roadways with Heavy Truck Traffic)	5	8

Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved and that proper construction procedures are used. We could provide paving sections for other Traffic Index values if desired.

Portland Cement Concrete Paving

Portland cement concrete paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic Indices are presented in the following table. We have assumed that the Portland cement concrete will have a compressive strength of at least 3,000 pounds per square inch.

Assumed Traffic Index	Concrete Paving (Inches)	Base Course (Inches)
4 (Automobile Parking)	7½	4
5 (Driveways with Light Truck Traffic)	8	4
6 (Roadways with Heavy Truck Traffic)	8½	4

We recommend that the concrete paving be properly reinforced. In addition, dowels are recommended at joints in the paving to reduce possible offsets.

Base Course

The base course for both asphaltic and concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California Department

of Transportation, Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction. The base course should be compacted to at least 95%.

6.5 GRADING

General

To provide support for the spread footing, the mat foundation and the slab surrounding the mat, all the existing fill and disturbed and loose natural soils within the construction area and 3 feet beyond in plan, should be excavated and replaced as properly compacted fill. The upper clay soils are expansive and we recommend the placing of at least 1½ feet of non-expansive soils beneath the mat and the surrounding slab. The layer of non-expansive soils will not be necessary beneath the spread footing and asphaltic and concrete paving. Adjacent to any existing structure, the excavation should be restricted so as not to endanger the existing structure. Any required additional fill should be properly compacted. Based on the laboratory test data, the upper soils have relatively high moisture contents and drying of these soils may be required.

Site Preparation and Compaction

After clearing the site and removing the existing fill and loose and disturbed natural soils, the upper clay soils beneath the mat and surrounding slab should be excavated to allow the placement of at least 1½ feet of relatively non-expansive soils beneath the mat and slab. Next, the moisture content of the exposed soils should be determined and the soils slowly and uniformly moistened (or dried), as necessary, to bring the soils to a uniformly moist condition. The moisture content of the clayey soils should be brought to between 2% and 4% over optimum moisture content to a depth of at least 6 inches.

After moisture conditioning, the exposed soils should be rolled with heavy compaction equipment. The upper 6 inches of exposed natural soils should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557 method of compaction. If the exposed soils are too wet to compact, the exposed soils should be overexcavated and replaced with at least 1 foot of 1½-inch crushed rock to provide a base for the compaction of the required backfill.

After compacting the exposed natural soils or placing the layer of crushed rock, the required fill should be placed in loose lifts not more than 8 inches thick and compacted to at least 90%. It is recommended that the moisture content of the on site clayey and silty soils at the time of compaction be brought to between 2% and 4% over optimum moisture content. The relatively non-expansive soils should be compacted to a moisture content varying no more than 2% below or above optimum moisture content.

Material for Fill

The on-site soils, less any debris or organic matter, may be used in required fills. The clay soils should not be placed within 1½ feet of the subgrade beneath the mat and slab. Any required imported fill and at least the upper 1½ feet of fill should consist of relatively non-expansive soils with an Expansion Index of less than 35. Any import material should contain sufficient fines (binder) so as to be relatively impermeable and provide a stable subgrade when compacted.

Field Observation

The reworking of the upper soils and the compaction of all required fill should be observed and tested by a representative of our firm. The observation and testing should include:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade, observe proofrolling, and delineate areas requiring overexcavation.
- Perform visual observation to evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Perform field density and compaction testing to determine the percentage of compaction achieved during fill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits may be obtained and arrangements may be made for the required inspection(s).

7.0 BASIS FOR RECOMMENDATIONS

The recommendations provided in this report are based on our understanding of the described project information and on our interpretation of the data collected during the subsurface explorations. We have made our recommendations based on experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the proposed cooling tower configuration, loads, location, or the site grade should be provided to us so we may review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based on the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to determine that the actual soil conditions are as anticipated. This also provides for a procedure whereby the client can be advised of unanticipated or changed conditions that would require modifications of our original recommendations. In addition, the presence of our representative at the site provides the client with an independent professional opinion regarding the geotechnically-related construction procedures. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.



FIGURES

FIGURES

BORING 1

DATE DRILLED: July 21, 2005
EQUIPMENT USED: Bucket
HOLE DIAMETER (in.): 16
ELEVATION: 100**

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
95	5	20.6	104	1	
		25.2	97	3	
		28.8	94	3	
90	10	22.9	102	2	
85	15	25.3	99	3	
80	20	2.7	97	6	
75	25	8.6	96	16	
70	30	12.6	105	17	
65	35				
	40				

2" Asphaltic paving - 5" Base Course

SC FILL - CLAYEY SAND - fine to medium, greyish brown

CL SILTY CLAY - stiff, light greyish brown

Becomes light brown

SM SILTY SAND - fine, dense, lenses of sand, light greyish brown

Thin layers of Silty Clay below 21 feet

END OF BORING AT 30 FEET.

NOTES: Water not encountered. No caving.

* Number of blows required to drive the Crandall sampler 12 inches for depths of:

0 to 26 feet using a 1600 pound hammer falling 12 inches
Below 26 feet using an 800 pound hammer falling 12 inches.

**Elevation refer to an assumed datum; see Figure 1 for location of the bench mark.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: FK
Prepared By: MS/MH
Checked By: MS

B11SOIL CRANDALL(ELEVATION) 52021.GPJ LAW_CRAN.GDT 8/3/05

JTC - Replace/Relocate Cooling Tower
SBC Facility, Torrance, California

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LOG OF BORING
Project: 4953-05-2021 Figure: 2

MAJOR DIVISIONS			GROUP SYMBOLS		TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)		GW	Well graded gravels, gravel - sand mixtures, little or no fines.
				GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)		GM	Silty gravels, gravel - sand - silt mixtures.
				GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)		SW	Well graded sands, gravelly sands, little or no fines.
				SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)		SM	Silty sands, sand - silt mixtures
				SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)			ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
				CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
				OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS				PT	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

Undisturbed Sample		Auger Cuttings	
Standard Penetration Test		Bulk Sample	
Rock Core		Crandall Sampler	
Dilatometer		Pressure Meter	
Packer		No Recovery	
Water Table at time of drilling		Water Table after 24 hours	

Correlation of Penetration Resistance with Relative Density and Consistency

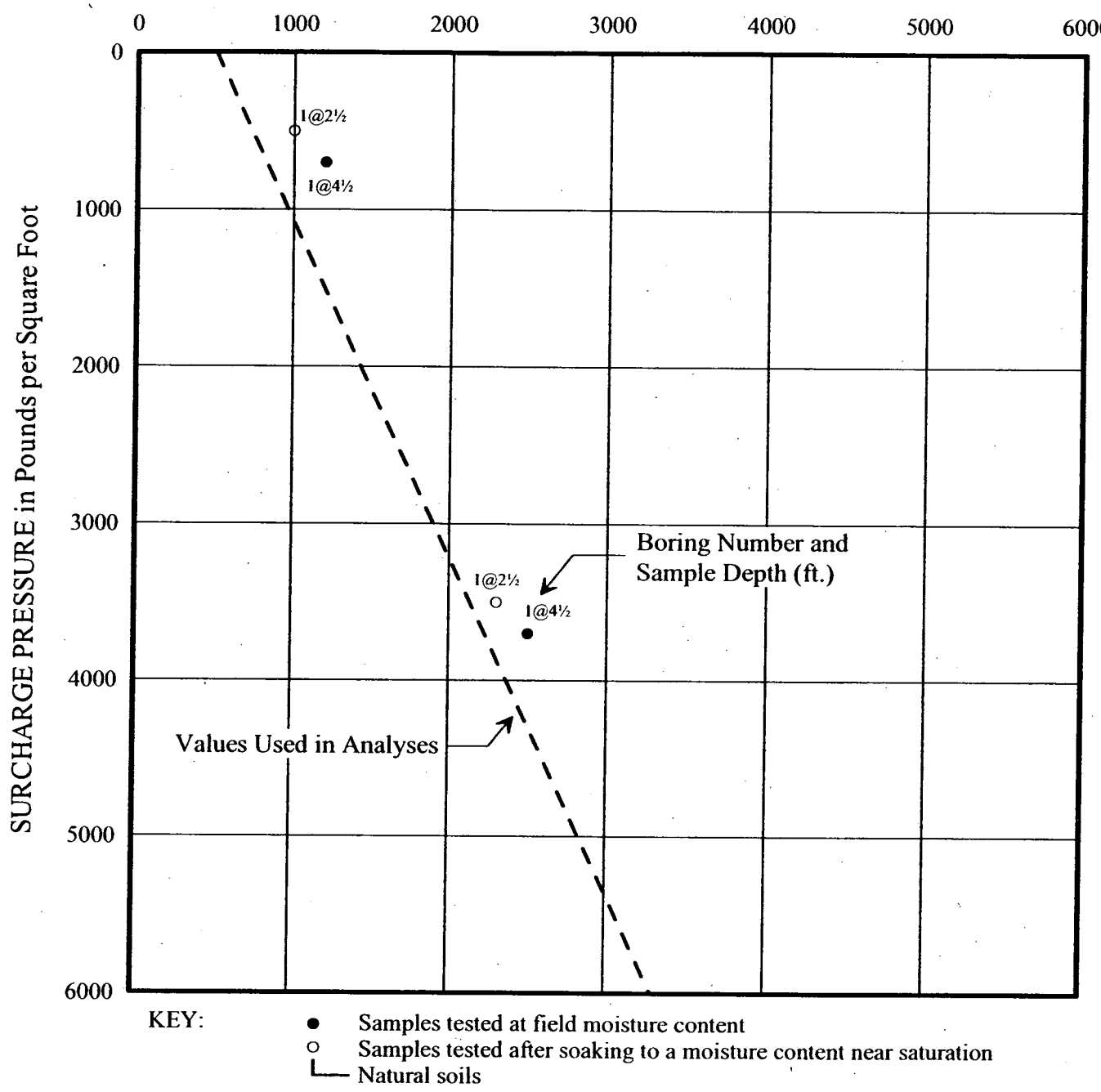
SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		Over 30	Hard

KEY TO SYMBOLS AND DESCRIPTIONS

MACTEC

FIGURE 3

JOB 4953-05-2021 DATE July 29, 2005 F.T. _____ DR. MH O.E. MS CHKD _____

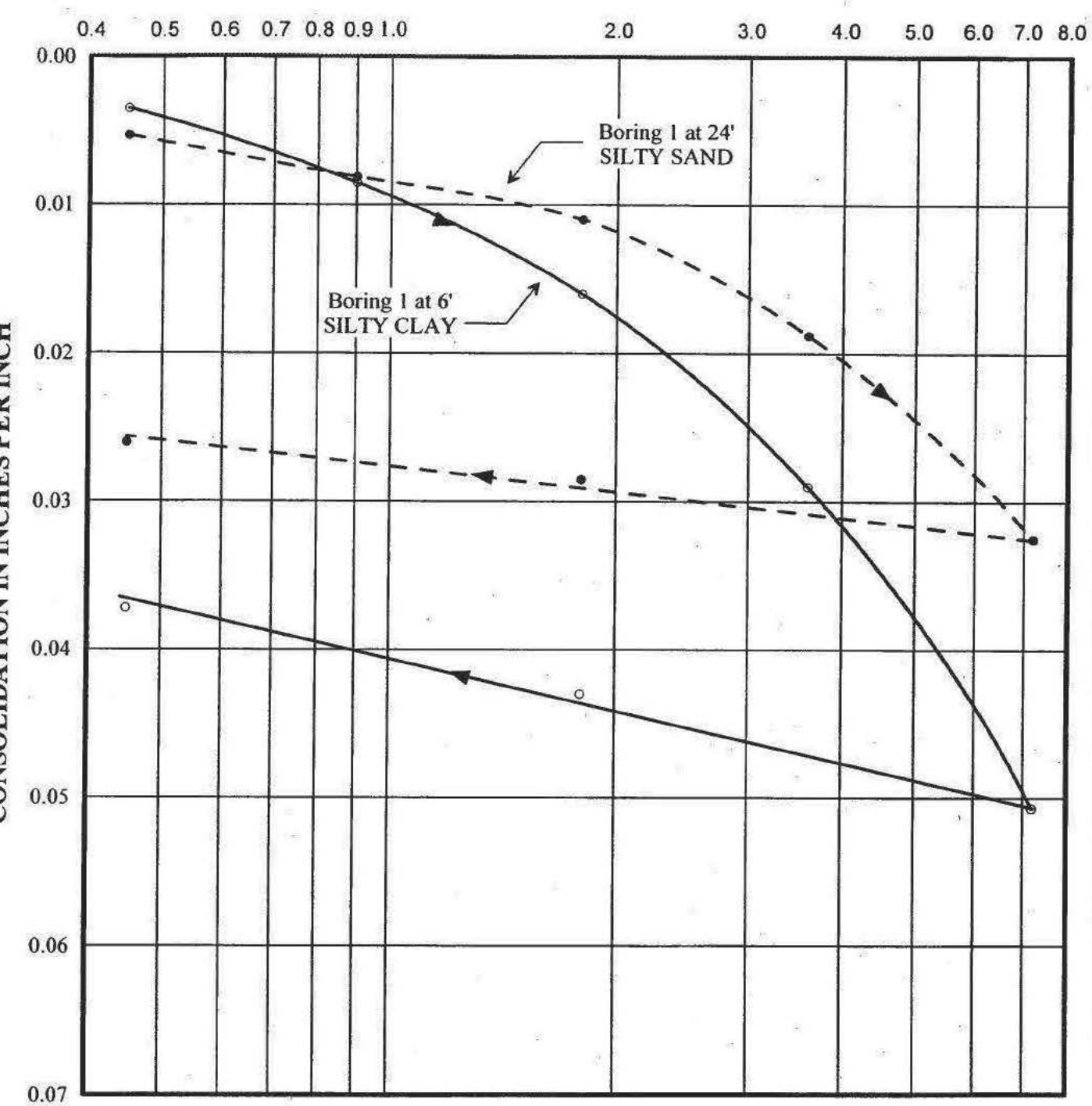


DIRECT SHEAR TEST DATA

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FIGURE 4

JOB 4953-05-2021 DATE August 3, 2005 F.T. ____ DR. LT O.E. MS CHKD MS



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

MACTEC

FIGURE 5

JOB 4953-05-2021 DATE August 3, 2005 F.T. ____ DR. ____ LT ____ O.E. ____ MS ____ CHKD ____

BORING NUMBER AND SAMPLE DEPTH:	1 at 1 to 2'
SOIL TYPE:	CLAYEY SAND
MAXIMUM DRY DENSITY: (lbs./cu.ft.)	122
OPTIMUM MOISTURE CONTENT: (%)	11.3

TEST METHOD: ASTM Designation D1557

COMPACTION TEST DATA

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FIGURE 6

JOB 4953-05-2021 DATE August 3, 2005 F.T. ____ DR. LT ____ O.E. MS ____ CHKD MS

BORING NUMBER AND SAMPLE DEPTH:	1 at 2 to 3'
SOIL TYPE:	SILTY CLAY
CONFINING PRESSURE: (lbs./sq. ft.)	144
INITIAL MOISTURE CONTENT: (% dry wt.)	16.2
FINAL MOISTURE CONTENT: (% dry wt.)	31.5
DRY DENSITY: (lbs/cu.ft.)	95.4
EXPANSION INDEX:	119

EXPANSION INDEX TEST DATA

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FIGURE 7

RESISTANCE R-VALUE TESTING RESULTS
(Cal Test 301)

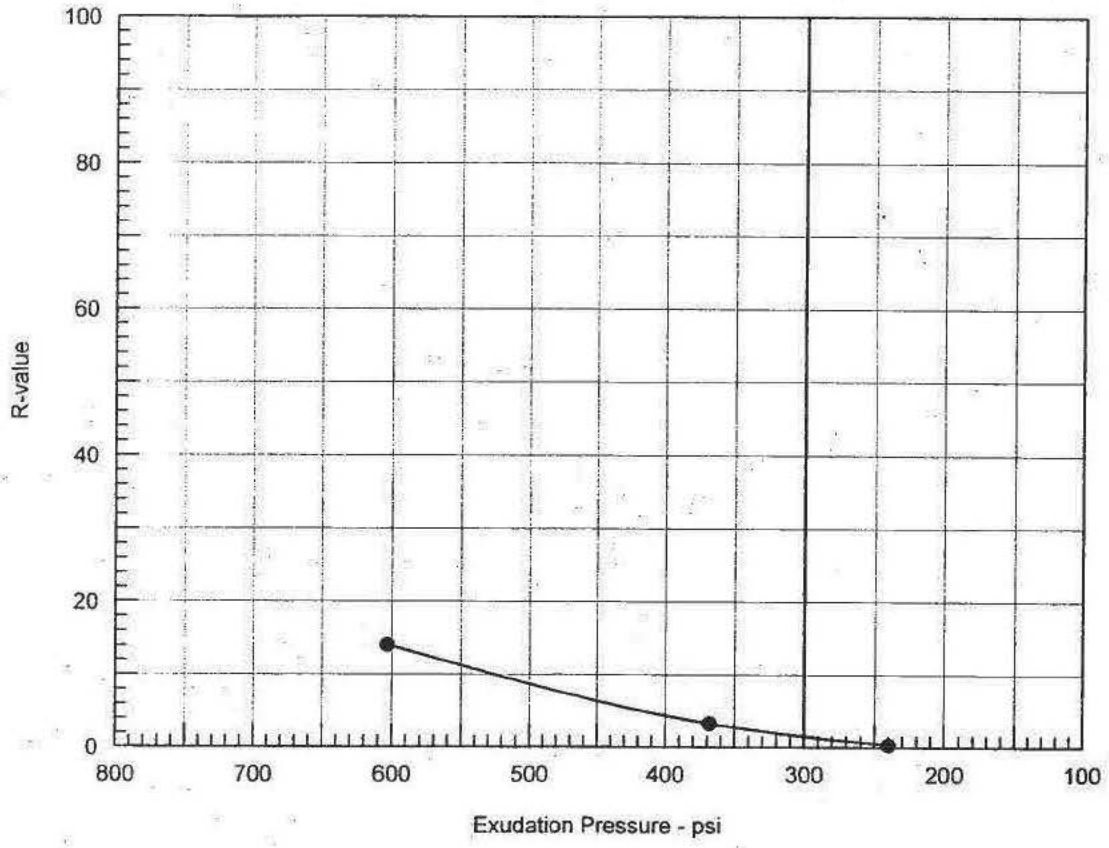
Date:	8/4/2005
Project No.:	4953-05-2021.02
Project:	JTC (for SBC)
Depth:	2-3' Sample Number: 1
Material Description:	Light brown Silty Clay
Tested by:	NS
Checked by:	LC
Remarks:	SD Lab #16768
	TI=4.5

Test specimen number	1	2	3
Compaction pressure (psi):	130	20	20
Wet weight (gms):	1320.0	1390.0	1380.0
Dry weight (gms):	1111.7	1111.7	1111.7
Tare weight (gms):	0.0	0.0	0.0
% Moisture:	18.7	25.0	24.1
Exudation load (lbs.):	7581	3012	4625
Exudation pressure (psi):	603	240	368
Total weight (gms.):	3245.0	3142.0	3247.0
Mold weight (gms.):	2126.0	2115.0	2102.0
Sample weight (gms.):	1119.0	1027.0	1145.0
Initial expansion (x10,000):	0	0	0
Final expansion (x10,000):	22	1	10
Expansion pressure (psi):	0.67	0.03	0.30
Ph at 2000 lbs.:	130	159	151
D turns:	3.53	4.57	4.52
R:	14.0	0.3	3.2
Height (in.):	2.55	2.53	2.56
Dry density (pcf):	112.0	98.4	109.2
Corrected R:	14.0	0.3	3.3

R-Value at 300 psi exudation pressure = 1.5

FIGURE 8.1

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	130	112.0	18.7	0.67	130	2.55	603	14.0	14.0
2	20	98.4	25.0	0.03	159	2.53	240	0.3	0.3
3	20	109.2	24.1	0.30	151	2.56	368	3.2	3.3

Test Results	Material Description
R-value at 300 psi exudation pressure = 1.5	Light brown Silty Clay
Project No.: 4953-05-2021.02 **Project:** JTC (for SBC) **Sample Number:** 1 **Depth:** 2-3' **Date:** 8/4/2005	**Tested by:** NS **Checked by:** LC **Remarks:** SD Lab #16768 TI=4.5
R-VALUE TEST REPORT **MACTEC Engineering and Consulting, Inc.**	**Figure** ____

FIGURE 8.2