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Celebrating 50 YEARS OF SERVICE
1971-2021

December 9, 2021
File Number 22207

Grubb Properties
4601 Park Road, Suite 450
Charlotte, North Carolina 28209

Attention: Charlie Rulick

Subject: Geotechnical Engineering Investigation
Proposed Mixed-Use Development
1200 through 1218 North Vine Street, 6245 and 6247 West Lexington Avenue
Los Angeles, California

Dear Mr. Rulick:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
GREGORIO VARELA
No. 81201
Exp. 9/30/23
CIVIL
STATE OF CALIFORNIA

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GV:ln

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LOS ANGELES, CALIFORNIA

INTRODUCTION

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

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

PROPOSED DEVELOPMENT

Information concerning the proposed development was obtained by review of the Conceptual Plans prepared by KTGY, dated October 11, 2021. The proposed development consists of construction of an eight story mixed-use structure, to be built at- or near the existing site grade. The first two levels will consist of parking and retail space, while the remaining levels will consist of residential space. The location and alignment of the proposed structure is shown on the enclosed Plot Plan.

Structural information is not available at this time. Wall loads are estimated to range between 4 and 12 kips per lineal foot. Column loads are estimated to range between 300 and 700 kips. Grading is expected to consist of excavations on the order of 5 to 7 feet for the removal and recompaction of existing unsuitable soils.

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

SITE CONDITIONS

The site is located at 1200 through 1218 North Vine Street, and 6245 and 6247 West Lexington Avenue, in the Hollywood area of the City of Los Angeles, California. The site is rectangular in shape, and just under one acre in area. The site is bounded by a two-story office building to the north, two apartment buildings to the east, Lexington Avenue to the south, and North Vine Street to the west. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The apartment buildings located to the east of the subject site are two and three stories in height. One of the buildings was built at-grade, while the other was built over a partially-subterranean parking garage. As shown in the enclosed Plot Plan, the building with the partially-subterranean garage is setback from the property line, therefore it is not anticipated that the new structure will surcharge the adjacent subterranean retaining walls.

Based on review of the Land Title Survey prepared by LG Land Surveying, Inc., dated October 13, 2020, the site grade descends gently to the southwest. The elevation relief observed across the site is in the order of 3 feet. The site is currently developed with two single-story commercial structures, and a paved parking lot. Vegetation at the site is limited, and consists of a few mature palm trees, as well as shrubbery contained in small planter areas. Drainage across the site appears to be by sheetflow to the city streets to the southwest.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on October 25, 2021 by drilling two borings. The borings were drilled to a depth of 30 and 50 feet below the existing grade, with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 and A-2.

The location of exploratory excavations was determined from hardscaped features shown in the enclosed Plot Plan. Elevations of the exploratory excavations were approximated from elevation provided in the Land Title Survey prepared by LG Land Surveying, Inc., dated October 13, 2020. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in both exploratory borings, to an approximate depth of 3 feet below the existing grade. The fill consist of sandy to clayey silt, and is dark brown in color, moist and stiff.

The fill is in turn underlain by native older alluvial soils, consisting of interlayered mixtures of sand, silt and clay. The alluvial soils are yellowish brown to dark brown in color, moist to wet, medium dense to very dense, or stiff, and fine to medium grained. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was encountered in both exploratory borings, at depths of 20 and 21½ feet below the existing grade. The historically highest groundwater level was established by review of the California Geological Survey Seismic Hazard Evaluation Report 026 Plate 1.2 entitled “Historically Highest Ground Water Contours”. Review of this plate indicates that the historically highest groundwater level is on the order of 37 feet below grade. A copy of this plate is included in the Appendix as Historically Highest Groundwater Levels Map.

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

Caving

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

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

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

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as 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.

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.

Review of the Earthquake Zones of Required Investigation Map of the Hollywood Quadrangle (CGS, 2014) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The closest zone is the Hollywood Fault Zone, which is located just over half-mile to the north of the subject site. A copy of this map is enclosed herein.

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

Liquefaction

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

As shown in the enclosed Earthquake Zones of Required Investigation Map, the State of California does not classify the site as part of a 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 conservative measure, a site-specific liquefaction analysis was performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered during exploration, at a depth of 20 and 21½ feet below the existing grade. Based on review of the seismic hazard zone report of the Hollywood 7½-minute quadrangle (CDMG, 2006), the historically highest groundwater level for the site was 37 feet below the ground surface. The enclosed liquefaction analysis is based on a groundwater level of 20 feet.

Section 11.8.3 of ASCE 7-16 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the OSHPD seismic utility program, this corresponds to a PGA_M of 0.99g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014) indicates a PGA of 0.91g (2 percent in 50 years ground motion) and a mean magnitude of 6.8 for the site. The liquefaction potential evaluation was performed by utilizing a magnitude 6.8 earthquake, and a peak horizontal acceleration of 0.99g.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on Boring 1. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory borings are presented on the enclosed E-Plate and F-Plate.

Based on CGS Special Publication 117A (CDMG, 2008) and (Bray and Sancio, 2006), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Furthermore, soils having a PI greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. The results of Atterberg Limits testing (shown on Plate F) indicate that some of soil layers below the subject site have PI greater than 18. Therefore, these soils are not considered prone to liquefaction, and the analysis of these soil layers was turned off in the liquefaction susceptibility columns.

The site-specific liquefaction analysis included in the Appendix, indicates that the site soils would not be prone to liquefaction during the ground motion expected during the design-based seismic event.

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 mapped tsunami inundation boundaries.

Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the site lies within the mapped inundation boundaries of the Mulholland Dam. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this assessment.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

During exploration, fill materials were observed to extend to a depth of 3 feet below the existing grade. The existing fill materials are considered to be unsuitable for support of foundations, floor slabs, or additional fill. However, the existing fill materials may be reused in the preparation of a compacted fill pad.

The reported fill depth was recorded at two discrete locations. Deeper fill materials may be encountered within other areas, including the portion of the site currently occupied by the existing structures. It is recommended that supplemental potholing be conducted around the perimeter of the site prior to construction. This would provide a better understanding of the fill distribution across the site, and help select a suitable temporary stabilization measure for temporary excavations which will be conducted adjacent to the property line.

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

Construction of a proper compacted fill pad may not be possible along portions of the perimeter, where the proposed structure will be built adjacent to the property lines, and the recommended compacted fill pad horizontal over-excavation may not be achievable. In areas where the horizontal over-excavation will not be possible, the proposed foundations should be deepened to bear in undisturbed alluvial soils.

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

SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

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

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2019
ASCE Design Standard	7-16
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.096g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.096g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.397g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.750g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.275g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.850g*

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

EXPANSIVE SOILS

The onsite geologic materials are in the high expansion range. The Expansion Index was found to be 94 and 106 for a representative bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slab-On-Grade" sections of this report.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on two samples representative of the onsite soils by Project X Corrosion Engineering indicate that the electrical resistivities of the soils are moderately corrosive to general metals when saturated. The soil pH value of the samples was between 8.0 and 8.1. This pH level is not detrimental to copper and aluminum alloys, but can allow corrosion of steel and iron in moist environments. Chloride levels in the samples are low and may cause insignificant corrosion of metals. Ammonia and Nitrates concentrations were not high enough to cause accelerated corrosion of copper and copper alloys, such as brass.

Sulfate content in the samples are considered negligible for corrosion of metals and cement. Special cement types need not be utilized for concrete structures in contact with the soils, since the sulfate content of the soils is negligible.

Detailed results, discussion of results and recommended mitigating measures are provided within the enclosed Corrosion Evaluation Report prepared by Project X Corrosion Engineering, dated December 7, 2021.

METHANE ZONES

This office has reviewed the City of Los Angeles Methane and Methane Buffer Zones map. Based on this review it appears that the subject property is not located within a Methane Zone or a Methane Buffer Zone, as designated by the City.

GRADING GUIDELINES

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.
- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.
- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.
- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.
- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Recommended Overexcavation and Blending

The proposed building areas shall be excavated to a minimum depth of 5 feet below the proposed grade, or 3 feet below the bottom of all foundations, whichever is greater. The excavation shall extend at least 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Once the onsite soils have been removed it is recommended that they should be blended to reduce the overall expansion index of the newly placed controlled fill. Where the site grading will result in a net export, the sandier or more granular materials should be segregated from the stockpiled soils and the more clayey or expansive materials should be exported. Samples of the segregated and/or blended soils should be tested by this office to ascertain the expansion index prior to placement and compaction.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. Based on the high expansion index of the site soils, it is recommended that fill materials are moisture conditioned to approximately 3 percent over optimum moisture content before recompaction.

All fill should be mechanically compacted in layers not more than 8 inches thick. The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Comparative compaction is defined, for purposes of these guidelines, as the ratio of the in-place density to the maximum density as determined by applicable ASTM testing.

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 (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 60. 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 (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) 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 some of the soils which will be exposed during grading and at the bottom of the excavations were locally above optimum moisture content. It is anticipated that the some of the excavated material to be placed as compacted fill, and some of the materials exposed at the bottom of excavated planes may require drying and aeration prior to recompaction.

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

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

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

Abandoned Seepage Pits

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

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

Where the seepage pit structure is to be left in place, the seepage pits should cleaned of all soil and debris. This may be accomplished by drilling. The pits should be filled with minimum 1½ sack concrete slurry to within 5 feet of the bottom of the proposed foundations. In order to provide a more uniform foundation condition, the remainder of the void should be filled with controlled fill.

Geotechnical Observations and Testing During Grading

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

LEED Considerations

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

In an effort to provide the design team with a viable option in this regard, demolition debris could be crushed onsite in order to use it in the ongoing grading operations. The environmental ramifications of this option, if any, should be considered by the team. The demolition debris should be limited to concrete, asphalt and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials and wood.

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

FOUNDATION DESIGN

Conventional

The proposed structure may be supported by conventional foundations bearing in a newly built uniform compacted fill pad. Where perimeter foundations will be built immediately adjacent to the property line, and the recommended compacted fill pad horizontal over-excavation will not be possible, the affected foundations shall be deepened through any fill to bear in undisturbed native alluvial soils.

In addition, conventional foundations proposed within the northern and eastern portion of the site shall be deepened as appropriate, to prevent the surcharge of neighboring foundations. The bottom of these foundations shall extend below a 1:1 (45 degree) surcharge plane, which is projected upward from the bottom of the neighboring foundations.

Where a foundation requires deepening to bear in native soils, the deepened portion of the proposed foundation should be backfilled with hard rock concrete having the same strength as the planned structural footing. The initial pour would not require reinforcing as it is simply passing the load through to the competent native soils. Once the initial pour has hardened, the footing may be reinforced and poured on top of the first pour. Some method of creating a positive bond between the two pours should be employed.

Continuous foundations may be designed for a bearing capacity of 2,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

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

The bearing capacity increase for each additional foot of width is 100 pounds per square foot. The bearing capacity increase for each additional foot of depth is 250 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

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

Foundation Reinforcement

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.30 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 250 pounds per cubic foot with a maximum earth pressure of 1,500 pounds per square foot.

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

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. This firm has prepared two settlement analyses, which calculate the anticipated settlement of the heaviest column foundations, for conditions where they bear in compacted fill and native alluvial soils. Copies of these analyses may be found in the Appendix of this report.

Based on these enclosed analyses, the maximum column foundation settlement anticipated for foundations bearing in compacted fill materials would be 0.99 inches, while the maximum column foundation settlement anticipated for foundations bearing in native soils would be on the order of 1.01 inches. The maximum differential settlement for the proposed foundation system is not expected to exceed ½-inch, and occur over a distance of approximately 30 feet.

Foundation Observations

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

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

RETAINING WALL DESIGN

The proposed structure is expected to be built near the existing grade. Therefore, the only retaining walls anticipated would be associated with the construction of elevator pits, planters, or shallow perimeter walls where the interior finished floor elevation will be slightly lower than the outdoor grade.

At this time, it is unknown if the proposed retaining walls will be serviced by a subdrain system. If the installation of a subdrain system will be omitted, the walls shall be designed for an undrained condition with full hydrostatic pressure. Recommendations for drained and undrained conditions are provided herein.

Additional pressure should be added to the retaining wall design, for a surcharge condition due to vehicular traffic or adjacent structures. At this time, it is not anticipated that the retaining walls will be surcharged by existing structures or traffic. For traffic surcharge, the upper 10 feet of any 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 traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Cantilever Retaining Walls

Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed utilizing the following table:

Height of Retaining Wall	Cantilever Retaining Wall **with** Wall Subdrain System Triangular Distribution of Active Earth Pressure	Cantilever Retaining Wall **without** Wall Subdrain System Triangular Distribution of Active Earth Pressure
Up to 6 feet	45 pcf	98 pcf (includes hydrostatic pressure)

The highly expansive properties of the on-site soils have been considered in the development of the recommended lateral earth pressure. For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure. Restrained retaining walls may be designed utilizing the following table:

Height of Retaining Wall	Restrained Retaining Wall **with** Wall Subdrain System Triangular Distribution of At-Rest Earth Pressure	Restrained Retaining Wall **without** Wall Subdrain System Triangular Distribution of At-Rest Earth Pressure
Up to 6 feet	68 pcf	95 pcf (includes hydrostatic pressure)

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

Dynamic (Seismic) Earth Pressure

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

Surcharge from Adjacent Structures

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2020-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

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

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

where:

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

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

Retaining Wall Drainage

If the retaining wall will be designed for a drained condition, the retaining walls should be provided with a subdrain covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. The onsite geologic materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density as determined by the most recent revision of ASTM D 1557.

As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to one-inch crushed rocks, wrapped in filter fabric. Subdrainage pipes should outlet to an acceptable location. 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.

If a drainage system is not provided, the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure. Lateral pressures based on a hydrostatic design are provided in a previous section of this report.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was encountered at a depth of 20 feet below the existing grade. Therefore, the only water which could affect the proposed retaining walls would be irrigation water and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

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

Waterproofing

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

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Based on the depths of fill encountered during exploration, and anticipating that the proposed foundations may extend to depths ranging between 2 and 4 feet, it is expected that temporary excavations in the order of 5 to 7 feet in depth will be required for the recommended grading and foundation construction. Deeper temporary excavations will be required if deeper fill materials are encountered during construction, or if deeper foundations will be required. It is recommended that potholing be conducted prior to construction, in order to anticipate the presence of deeper fill materials.

The on-site fill and native soils are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic, structures or property lines. Surcharged and unsurcharged vertical excavations may be performed to a maximum height of 7 feet with the aid of slot-cuts, as recommended in the following section. Temporary shoring will be required for vertical excavations exceeding a height of 7 feet. Trench shoring may be utilized for the deepening of foundations.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum depth of 15 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

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

Slot Cutting

Where a property line, the public right of way, an adjacent structure, or traffic will surcharge a temporary excavation, the slot cutting method may be utilized to maintain a stable excavation. The slot cutting method may also be utilized for the deepening of foundations. The height of the excavation is limited to 7 feet. The “A-B-C” slot-cutting procedure is recommended.

The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation consists of excavating the “A” slots. Alternate “A” slots of 8 feet may be worked. The remaining earth buttresses (“B” and “C” slots) should be 8 feet in width for a combined intervening length of 16 feet. The “A” slots should be properly backfilled, before the “B” slots are excavated. The height of the slots shall not exceed 7 feet in height. Calculations indicating that slots 8 feet in width will be stable for the maximum recommended height of 7 feet, including a surcharge load from adjacent walls and vehicular traffic, have been included in the appendix of this report.

Trench Shoring

Where necessary, a temporary trench shoring system may be utilized to stabilize new foundation excavations. Temporary trench shoring may consist of plywood, timber struts and angle braces, or a hydraulic trench shoring system. Temporary shoring and bracing systems up to 10 feet in height should be designed for a triangular pressure distribution with a minimum equivalent fluid pressure of 28 pounds per cubic foot. Additional active pressure should be added for a surcharge condition due to adjacent structures or vehicular traffic. It is recommended that a qualified shoring contractor be retained to determine the acceptable materials and procedures to be utilized for shoring.

The design team and contractor must be aware that the use of temporary shoring may impede the continuous construction of foundations. Foundations may require to be poured in several phases to accommodate for the removal of the trench shoring, while maintaining a stable excavation.

SHORING DESIGN

Conventional shoring may also be utilized to stabilize grading or foundation excavations. The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that Geotechnologies, Inc. review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Based on the anticipated excavation depth, it is anticipated that the soldier piles will be designed for a cantilever condition.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2½ 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 soldier pile design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot of depth, up to a maximum of 5,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

Groundwater was encountered during exploration at depths ranging between 20 and 21½ feet below the existing site grade. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength p.s.i. of 1,000 over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

Where caving occurs, it will be necessary to utilize casing or polymer drilling fluid to maintain open pile shafts. 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.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.30 based on uniform contact between the steel beam and 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 5 feet below the bottom of excavated plane whichever is deeper.

Lagging

Soldier piles and anchors should be designed for the full anticipated pressures. 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

Cantilevered shoring supporting a level backslope may be designed utilizing a triangular distribution of pressure as indicated in the following table:

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 10	28

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.

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 recommended that shoring deflection be limited to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed, provided there are no structures within a 1:1 plane drawn upward from the base of the excavation. 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.

Monitoring

Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure 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

Interior concrete floor slabs should be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over undisturbed native alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 5 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed native alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

Based on ACI 302.2R-30, Chapter 7, for projects which do not have vapor sensitive coverings or humidity-controlled areas, a vapor retarder/barrier is not necessary. Where a vapor retarder/barrier is considered necessary, 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.

Based on ACI 302.2R-30, Chapter 7, for projects with vapor sensitive coverings, a vapor retarder/ barrier should be provided. Figure 7.1 shows that the slab should be poured on the vapor retarder/barrier. The ACI guide notes in 5.2.3.2 that the decision to locate the vapor retarder/barrier in direct contact with the slab's underside had long been debated. Experience has shown, however, that the greatest level of protection for floor coverings, coating, or building environments is provided when the vapor retarder/barrier is placed in direct contact with the slab. The necessity of a vapor retarder as well as the use of dry granular material, as discussed above is not a geotechnical issue and should be confirmed by qualified members of the design team.

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, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 8 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 (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction.

Slab Reinforcing

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

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction, as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars Traffic	4	5
Moderate Truck Traffic	5	7

Concrete paving may also be used on the project. For passenger cars and moderate truck traffic, concrete paving should be 6 inches of concrete over 4 inches of compacted base. For standard crack control maximum expansion joint spacing of 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #4 steel bars on 16-inch centers each way.

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.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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

Percolation testing of the on-site soils was not conducted by this firm. However, based on the fines content of the majority of the site soils, it is the opinion of this firm that these soils will have poor infiltration capabilities. Allowing stormwater infiltration would result in a perched water condition. In addition, some of the site soils were determined to be highly expansive when saturated.

Groundwater was encountered during exploration, to depths ranging between 20 and 21½ feet below grade. Current regulations require that the bottom of infiltration systems maintain a minimum vertical separation of 10 feet above the groundwater level. Based on the required vertical separation, and the shallowest depth to groundwater observed during exploration, any potential stormwater infiltration to be conducted at the site would have to occur within the upper 10 feet of soils. Infiltration within this upper soil stratum is not recommend, as it would saturate the soils providing primary support to the proposed structure. Saturation of these soils would affect their strength.

Based on the above considerations, stormwater infiltration is not recommended for the subject site. Where infiltration of stormwater into the subgrade soils is not advisable, most Building Officials have 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 provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

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

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

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

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

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

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

EXCLUSIONS

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

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples in general accordance with the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Excavation Logs", A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

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

Laboratory Compaction Characteristics

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

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plate presented in the Appendix of this report.

Atterberg Limits

Depending on their moisture content, cohesive soils can be solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed F-Plate.

REFERENCES

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California Department of Conservation, Division of Mines and Geology, 1998 (Revised 2006), Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle, Los Angeles County, California., C.D.M.G. Seismic Hazard Zone Report 026, map scale 1:24,000.

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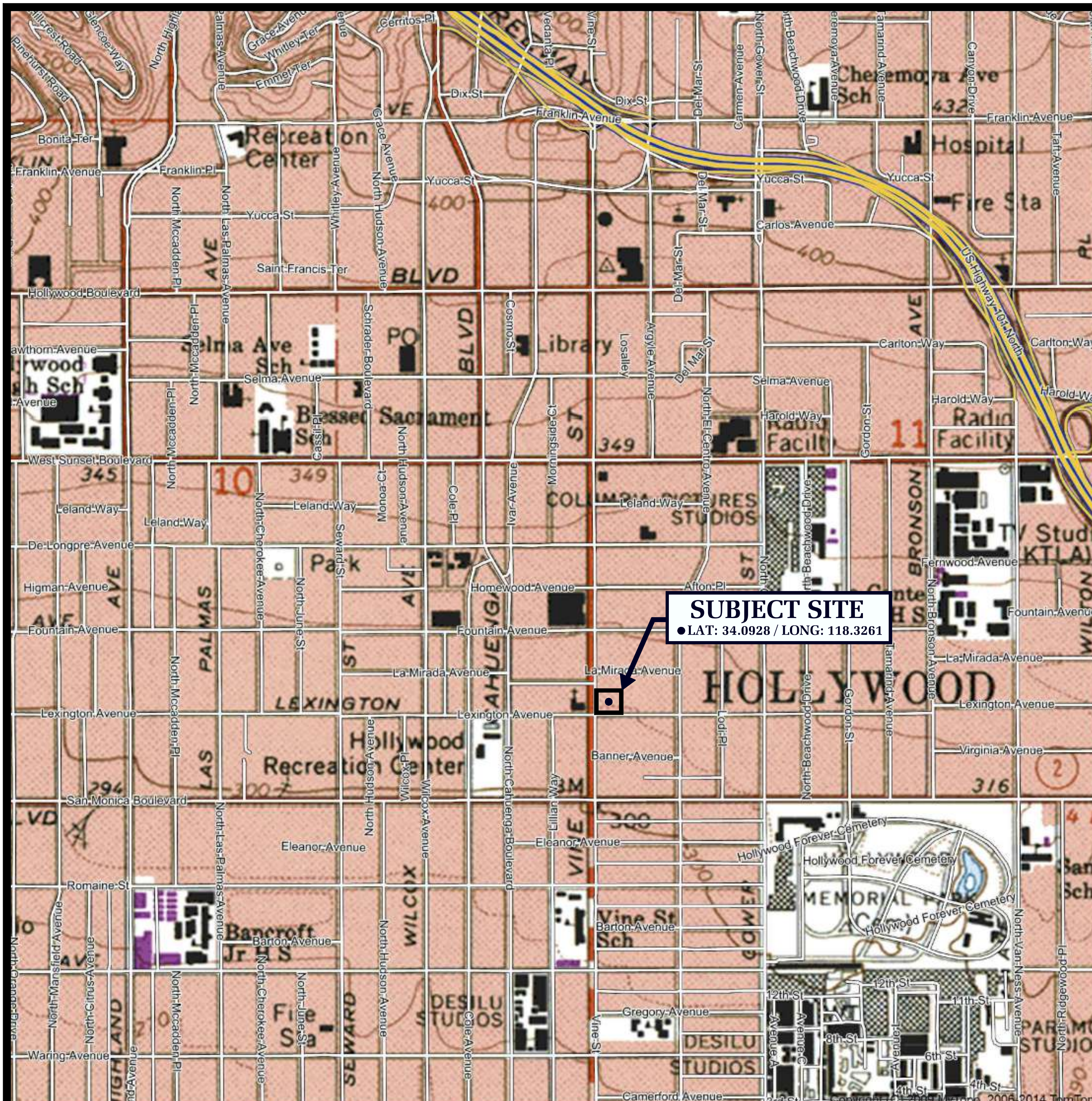


SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FEET

N

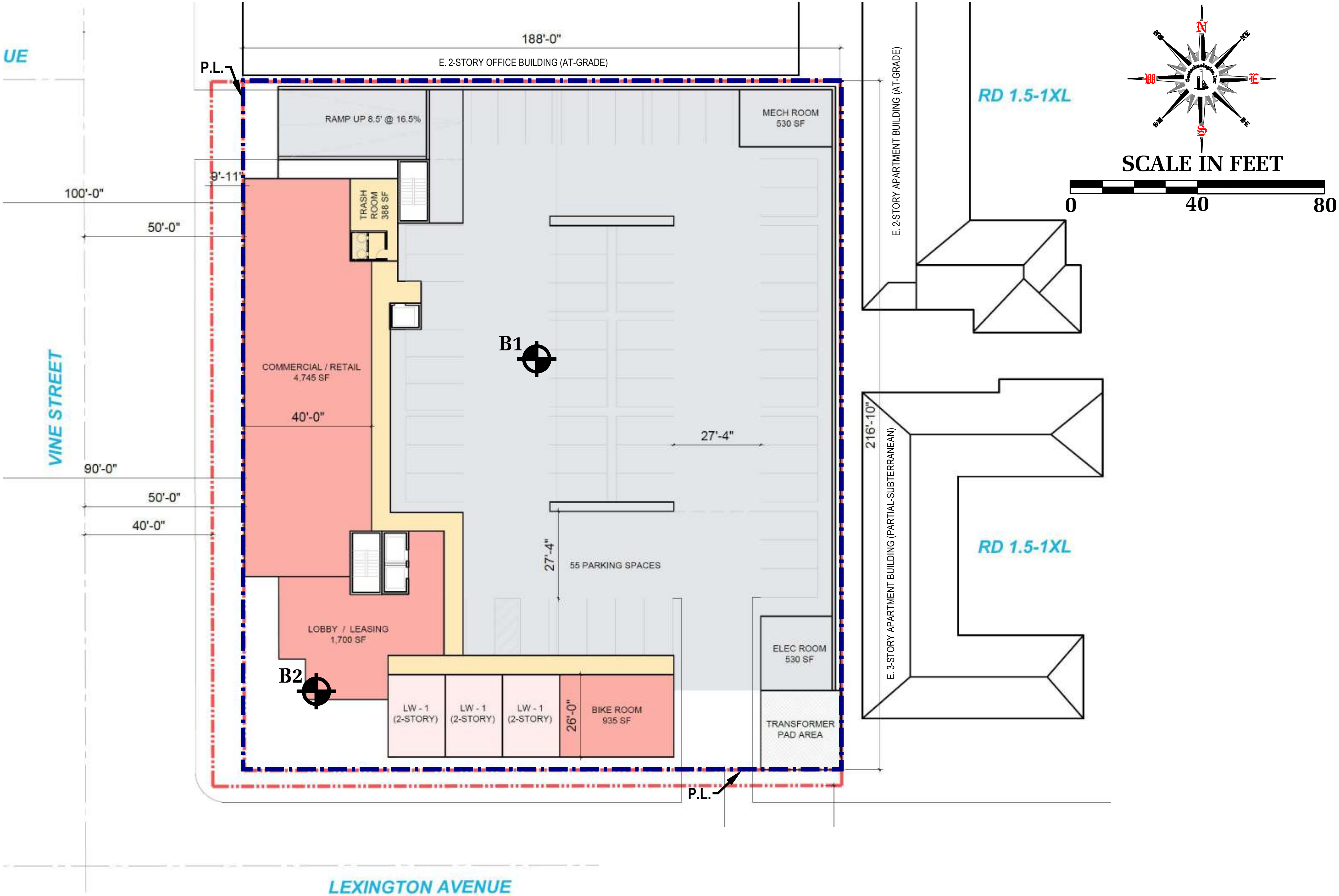
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

GRUBB PROPERTIES
1200 N. VINE ST., LOS ANGELES, CA

FILE NO. 22207



LEGEND

B2 LOCATION & NUMBER OF BORING

REFERENCE: CONCEPTUAL PLAN (LEVEL 1) BY KTGY DATED 10/11/21

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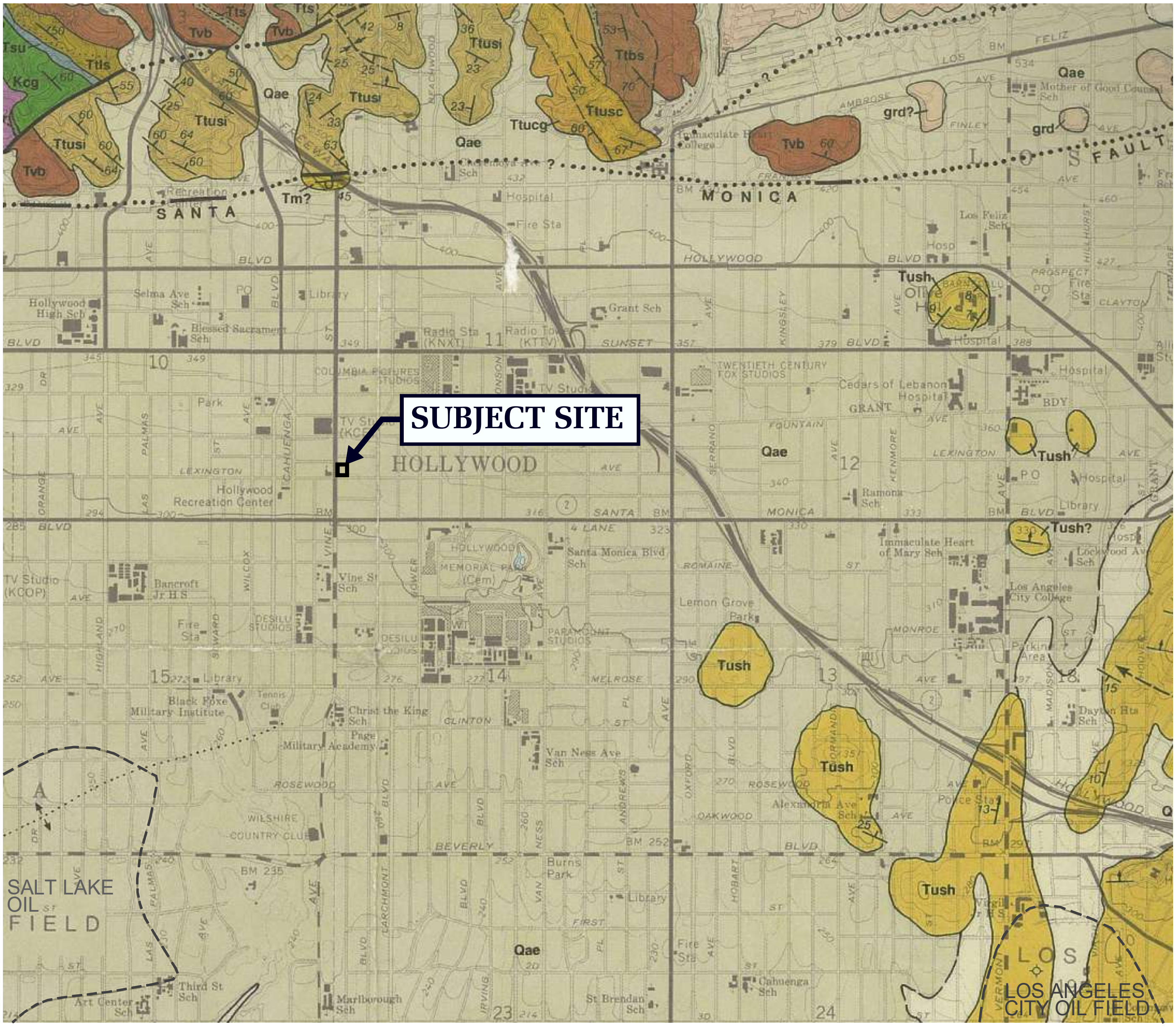
Consulting Geotechnical Engineers

PLOT PLAN

GRUBB PROPERTIES
1200 N. VINE STREET, LOS ANGELES, CA

File No.: 22207

Date: December 2021



SCALE IN FEET

0 1000 2000 4000

LEGEND

Qae: Alluvium- Clay, sand and gravel, slightly elevated and dissected

Tush: Unnamed Shale- Silty clay shale

Ttusi: Upper Topanga Formation- Clay shale or claystone

Ttus: Upper Topanga Formation- Sandstone

Ttusc: Upper Topanga Formation- Massive sandstone

Ttucg: Upper Topanga Formation- Cahuenga conglomerate

Tvb: Middle Topanga Formation- Basaltic volcanic rocks

Kcg: Unnamed Streta- Crudely bedded conglomerate

—••••? Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

N

REFERENCE:DIBBLEE, T.W., (1991), MAP #DF-30, GEOLOGIC MAP OF THE HOLLWOOD AND BURBANK (SOUTH 1/2) QUADRANGLES

LOCAL GEOLOGIC MAP

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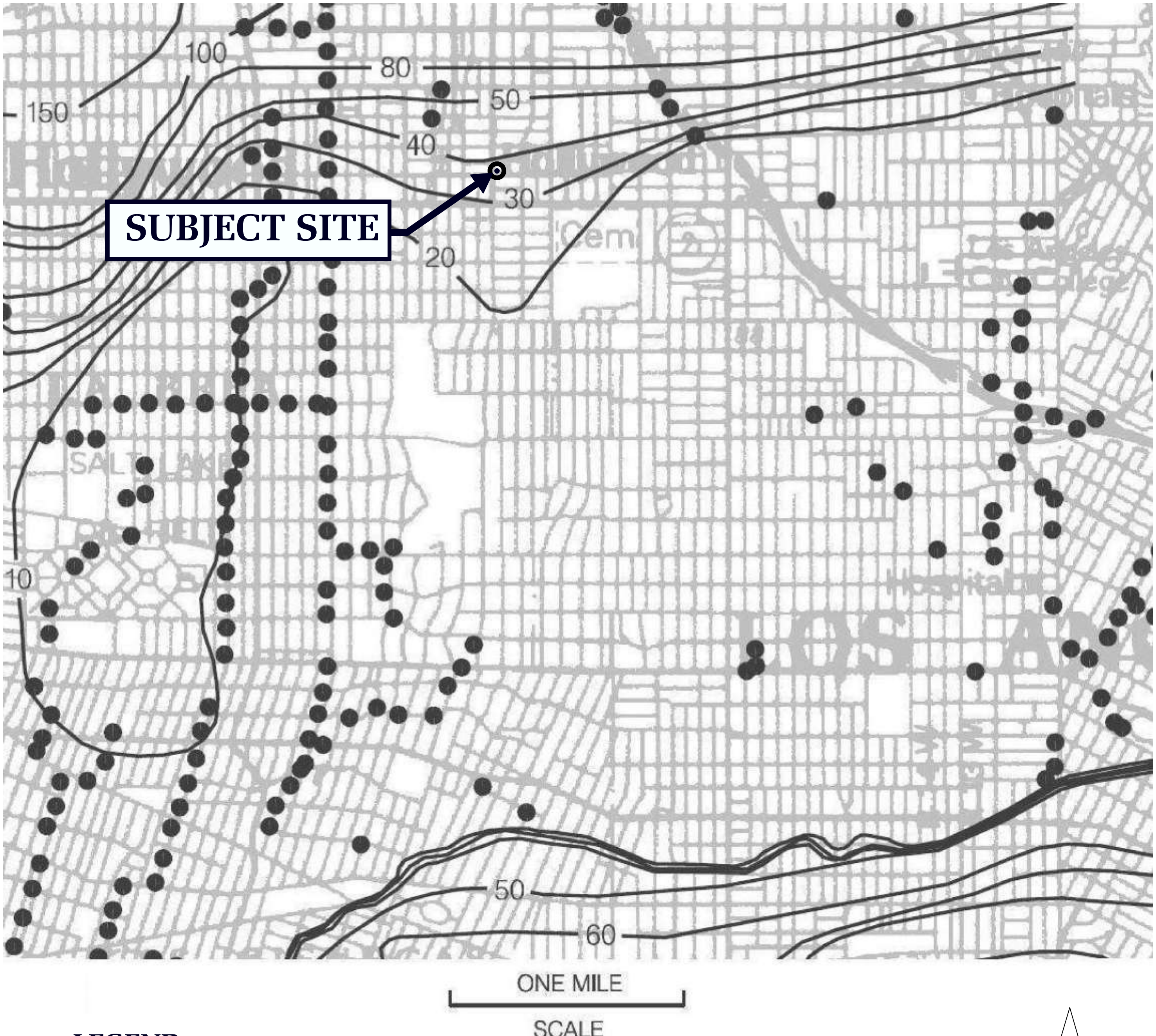
Consulting Geotechnical Engineers

GRUBB PROPERTIES
1200 N. VINE ST., LOS ANGELES, CA

FILE NO. 22207



LEGEND

20 Depth to groundwater in feet

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

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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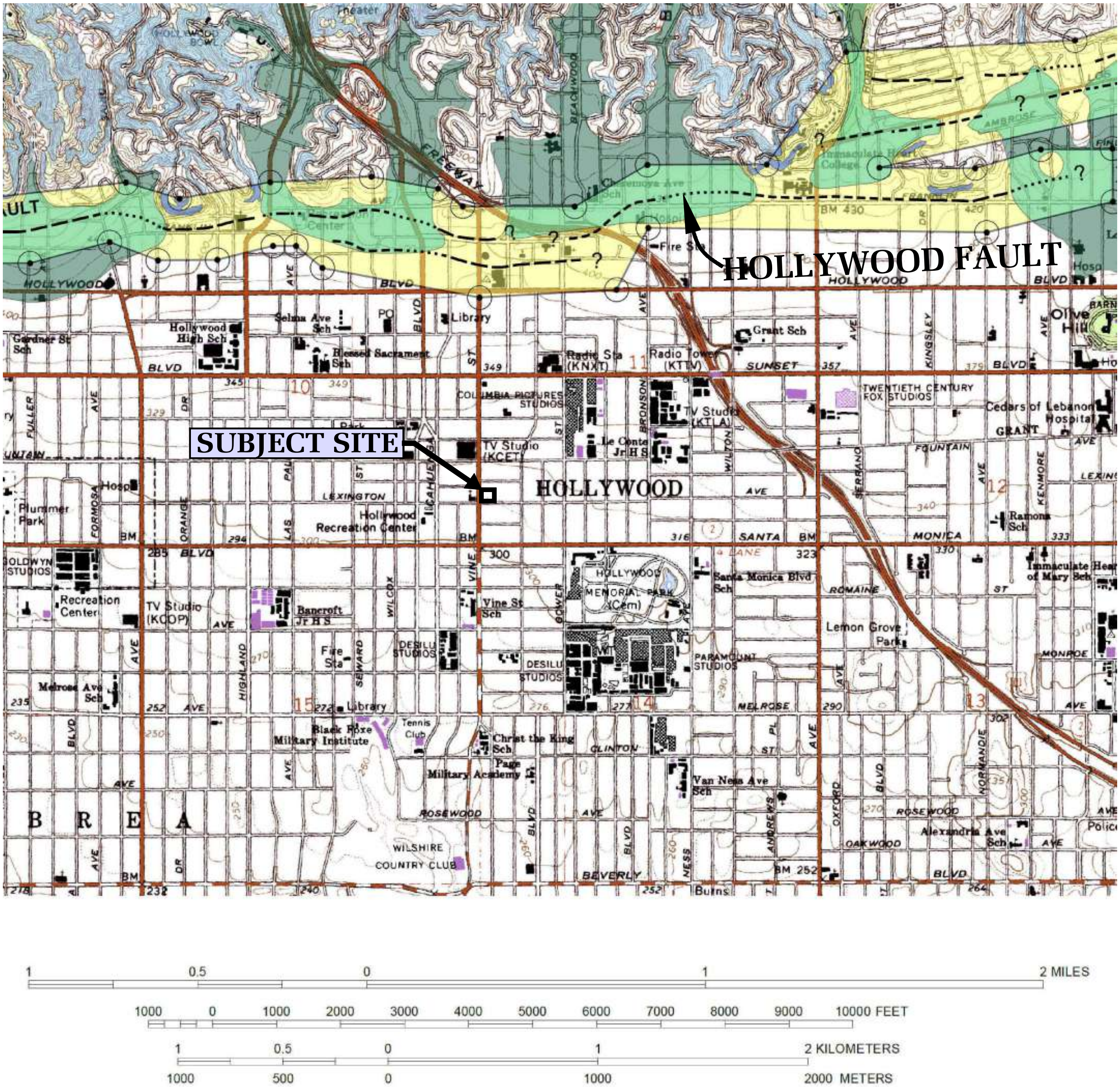
GRUBB PROPERTIES
1200 N. VINE ST., LOS ANGELES, CA

FILE NO. 22207



Contour Interval 20 Feet

Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

Liquefaction Area

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, HOLLYWOOD QUADRANGLE, CALIFORNIA GEOLOGICAAL SURVEY, NOVEMBER 6, 2014

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

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GRUBB PROPERTIES
1200 N. VINE ST., LOS ANGELES, CA

FILE NO. 22207

BORING LOG NUMBER 1

Grubb Properties **Date: 10/25/21** **Elevation: 315.0'***

File No. 22207 **Method: 8-inch diameter Hollow Stem Auger**

ln
*Reference: Land Title Survey by LG Land Surveying, Inc. dated 10/13/20

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 2-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	19	19.3	110.6	-		
				3 --		
				-	CL	NATIVE SOIL: Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	15	16.1	SPT	5 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, stiff, fine grained
				6 --		
				-		
				7 --		
7.5	26	11.4	109.5	-		
				8 --	SM	Silty Sand, dark brown, moist, medium dense, fine grained, minor pebbles
				-		
				9 --		
				-		
10	13	8.0	SPT	10 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, medium dense, fine to medium grained
				11 --		
				-		
				12 --		
12.5	22	10.8	115.5	-		
				13 --	SP	Sand, dark brown, moist, medium dense, fine to medium grained
				-		
				14 --		
				-		
15	15	8.7	SPT	15 --		
				-	SP/SM	Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
17.5	28	18.4	106.5	-		
				18 --	ML/CL	Sandy Silt to Silty Clay, dark brown, moist, stiff
				-		
				19 --		
				-		
20	19	12.7	SPT	20 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, stiff, fine grained
				21 --		
				-		
22.5	70	10.7	123.2	22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	11	24.6	SPT	25 --		
				-	CL/SC	Silty Clay to Clayey Sand, dark brown, wet, medium dense, stiff, fine grained

BORING LOG NUMBER 1

Grubb Properties

File No. 22207

In

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	22	17.8	113.5	-		
				28 --		
				-		
				29 --		
				-		
30	21	16.8	SPT	30 --		
				-	SM	Silty Sand, dark brown, very moist, medium dense, fine to medium grained
				31 --		
				-		
				32 --		
32.5	77	14.4	120.2	-		
				33 --		
				-		
				34 --		
				-		
35	31	16.3	SPT	35 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, medium dense, fine to medium grained
				36 --		
				-		
				37 --		
37.5	84	11.9	114.5	-		
				38 --	SP	Sand, dark brown, wet, very dense, fine to medium grained
				-		
				39 --		
				-		
40	26	15.2	SPT	40 --		
				-		medium dense
				41 --		
				-		
				42 --		
42.5	70	11.0	123.9	-		
				43 --	SP	Sand, dark brown, wet, very dense, fine grained, minor cobbles
				-		
				44 --		
				-		
45	35	15.4	SPT	45 --		
				-		medium dense
				46 --		
				-		
				47 --		
47.5	75	15.6	115.8	-		
				48 --	SM/ML	Silty Sand to Sandy Silt with Clay, dark brown, dense, stiff, fine to medium grained
				-		
				49 --		
				-	CL	Silty Clay, dark brown, moist, stiff
50	20	24.9	SPT	50 --		
				-		Total Depth: 50 feet Water at 20 feet Fill To 3 feet

BORING LOG NUMBER 2

Grubb Properties | **Date: 10/25/21** | **Elevation: 312.5'***

File No. 22207 | **Method: 8-inch diameter Hollow Stem Auger**

ln | *Reference: Land Title Survey by LG Land Surveying, Inc. dated 10/13/20

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 2-inch Base
				-		
				1 --		FILL: Sandy to Clayey Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	30	18.0	114.2	-		
				3 --		
				-	ML/CL	NATIVE SOILS: Clayey Silt to Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	31	14.1	113.7	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	27	11.8	119.3	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	28	5.6	106.6	15 --		
				-	SP	Sand, dark and yellowish brown, moist, medium dense fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	27	15.1	116.0	20 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	16	20.1	110.9	25 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark brown, moist, stiff

BORING LOG NUMBER 2

Grubb Properties

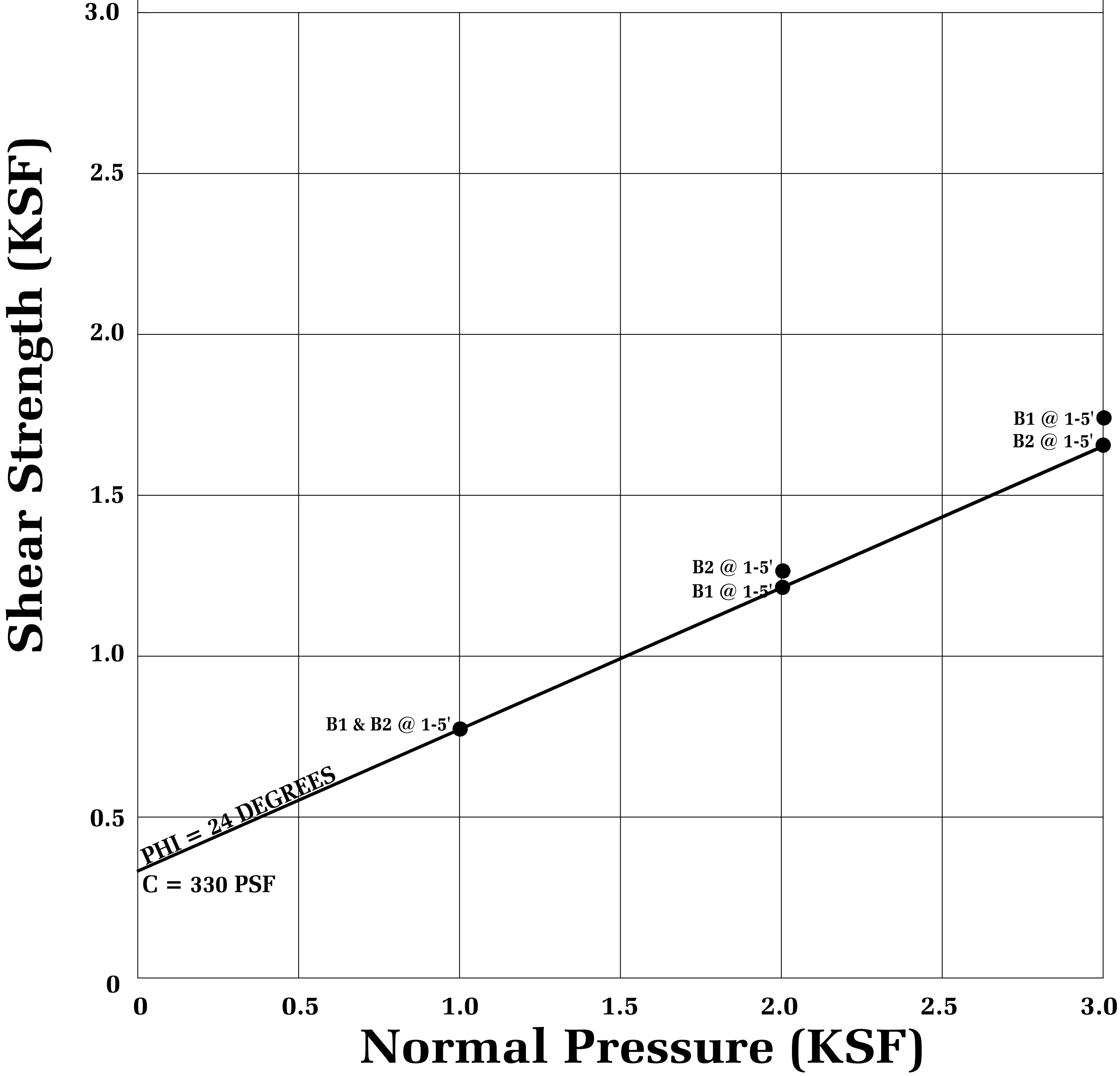
File No. 22207

ln

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 to Clayey Silt, dark and grayish brown, moist, stiff
30	81	14.8	120.9	30 --		
				-		Total Depth: 30 feet
				31 --		Water At 21.5 feet
				-		Fill To 3 feet
				32 --		
				-		NOTE: The stratification lines represent the approximate
				33 --		boundary between earth types; the transition may be gradual.
				-		
				34 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				35 --		Modified California Sampler used unless otherwise noted
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	ML/CL	119.2	13.0	24.2
B2 @ 1-5'	ML/CL	120.9	12.5	22.9



● Direct Shear, Saturated

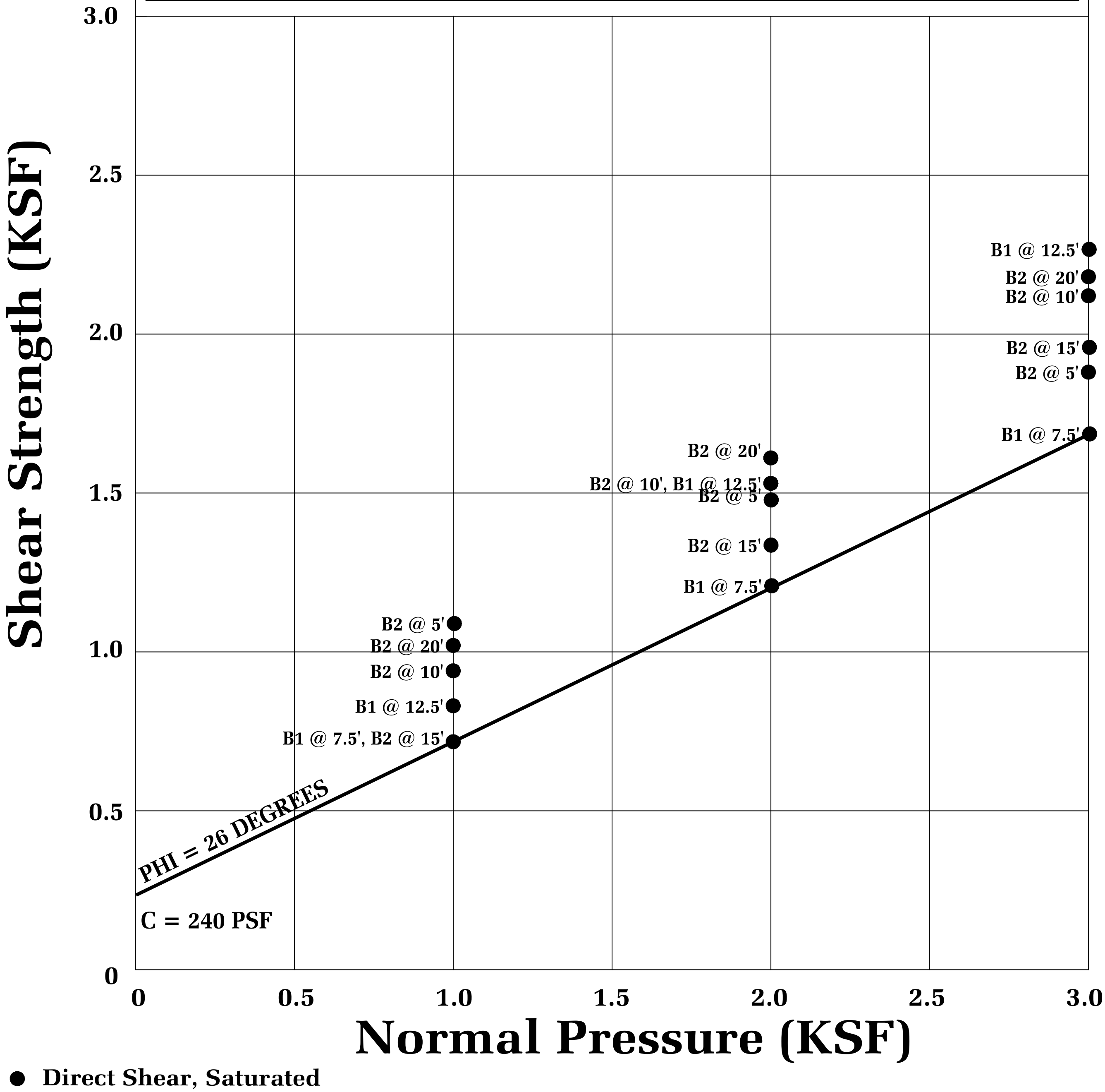
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

GRUBB PROERTIES
1200 NORTH VINE STREET, LOS ANGELES

FILE NO. 22207 | PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 5'	ML/CL	113.7	14.1	16.3
B1 @ 7.5'	SC	109.5	11.4	17.1
B2 @ 10'	SM/ML	119.3	11.8	18.5
B1 @ 12.5'	SP/SC	115.5	10.8	14.0
B2 @ 15'	SP/SC	106.6	5.6	18.8
B2 @ 20'	SC	116.0	15.1	20.5

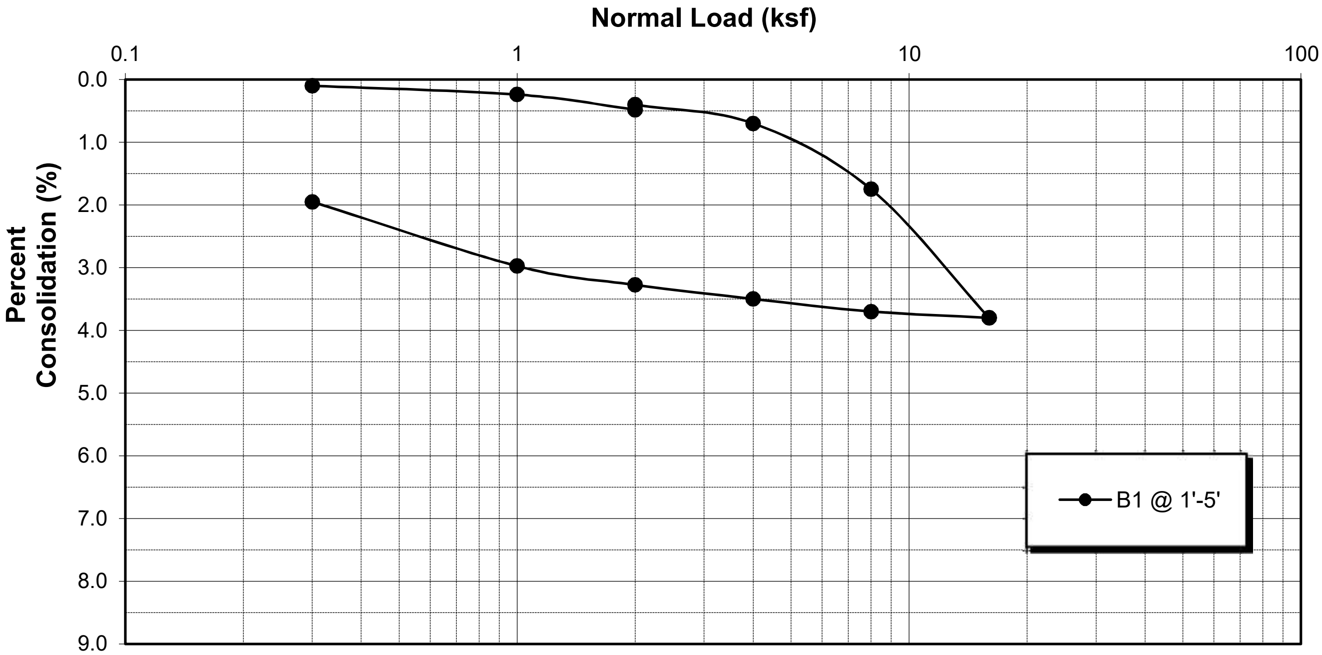


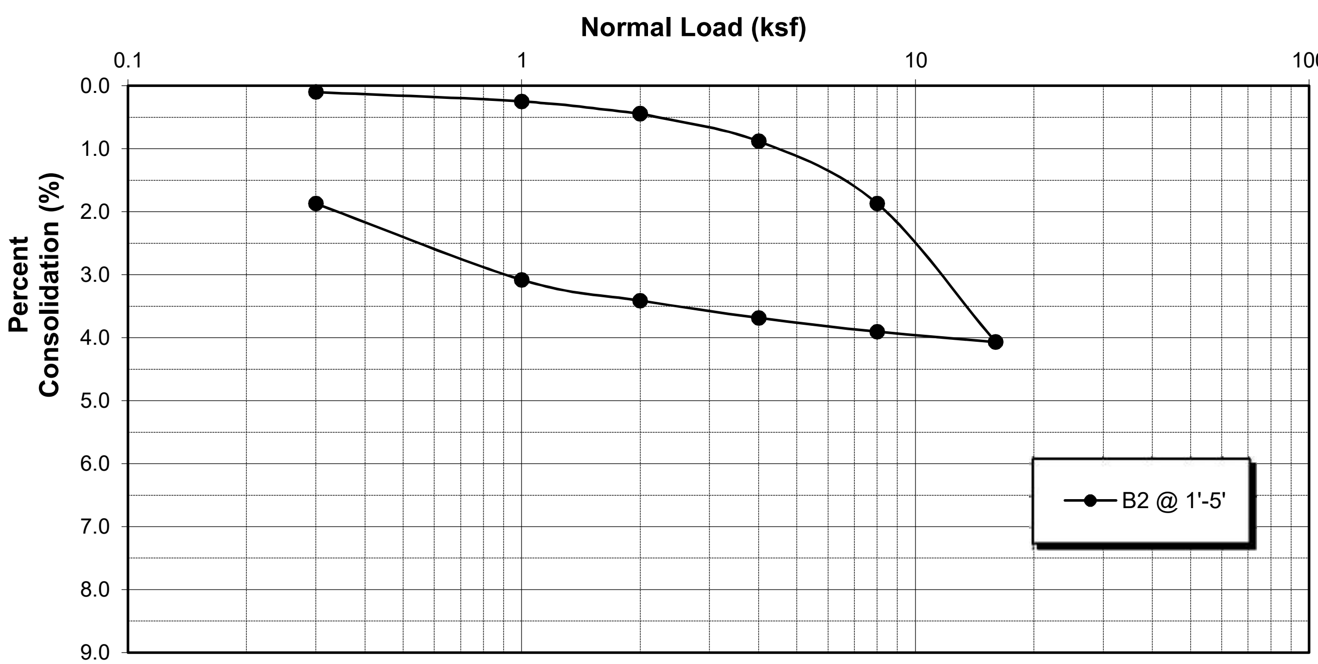
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

GRUBB PROPERTIES
1200 NORTH VINE STREET, LOS ANGELES

FILE NO. 22207 | PLATE: B-2



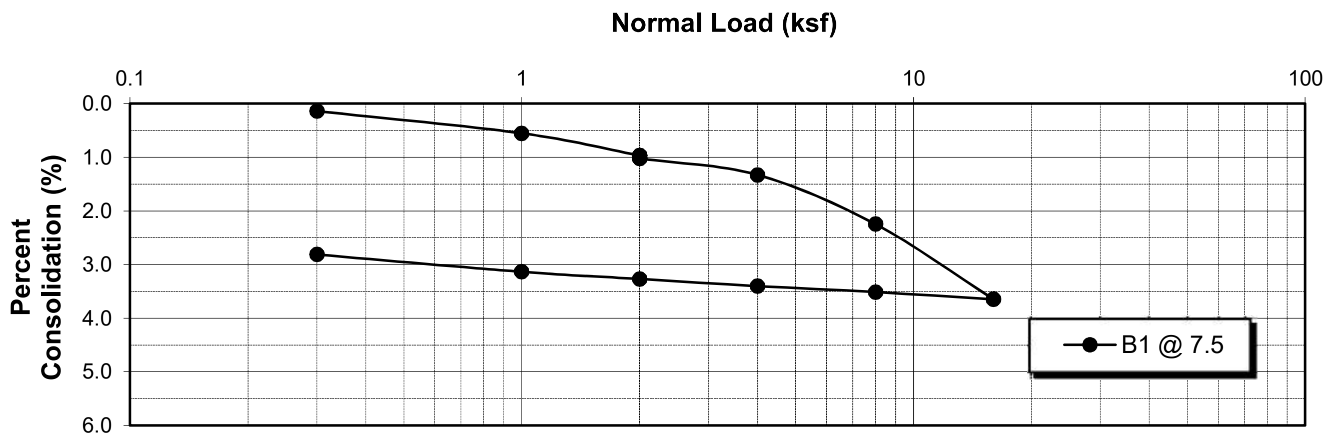


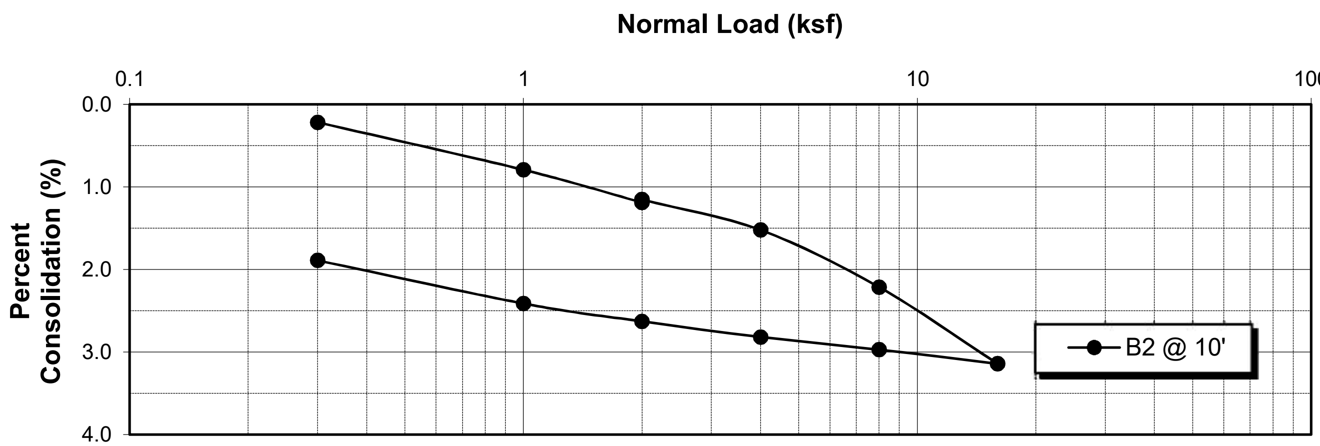
Water added at 2 KSF

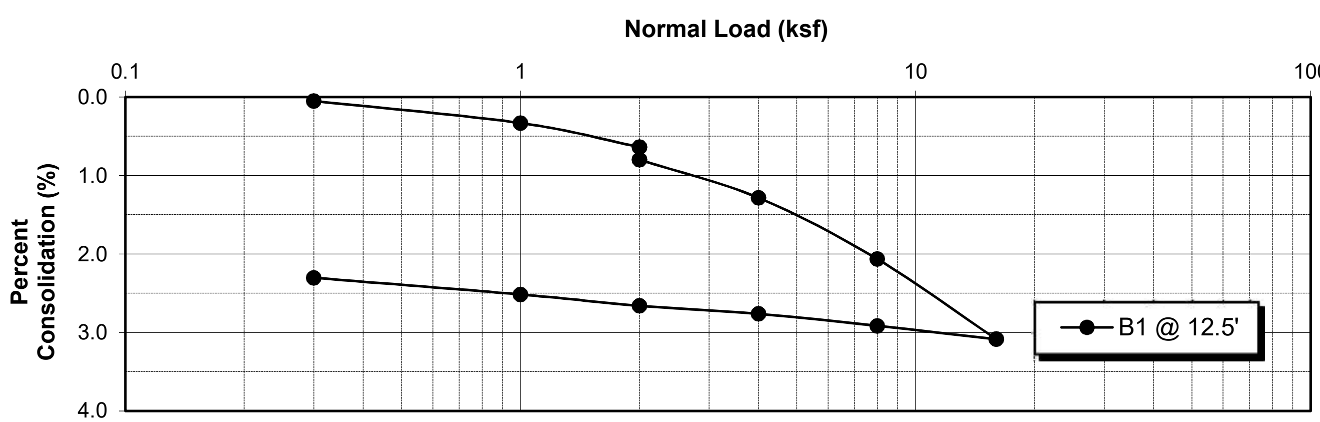
CONSOLIDATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: GRUBB PROPERTIES	
FILE NO.: 22207	PLATE: C-1





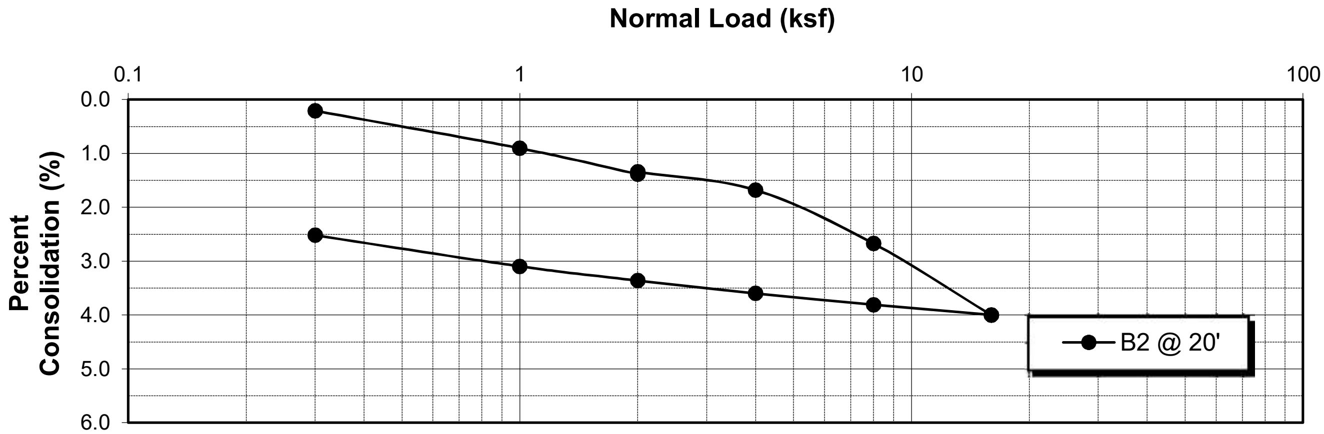


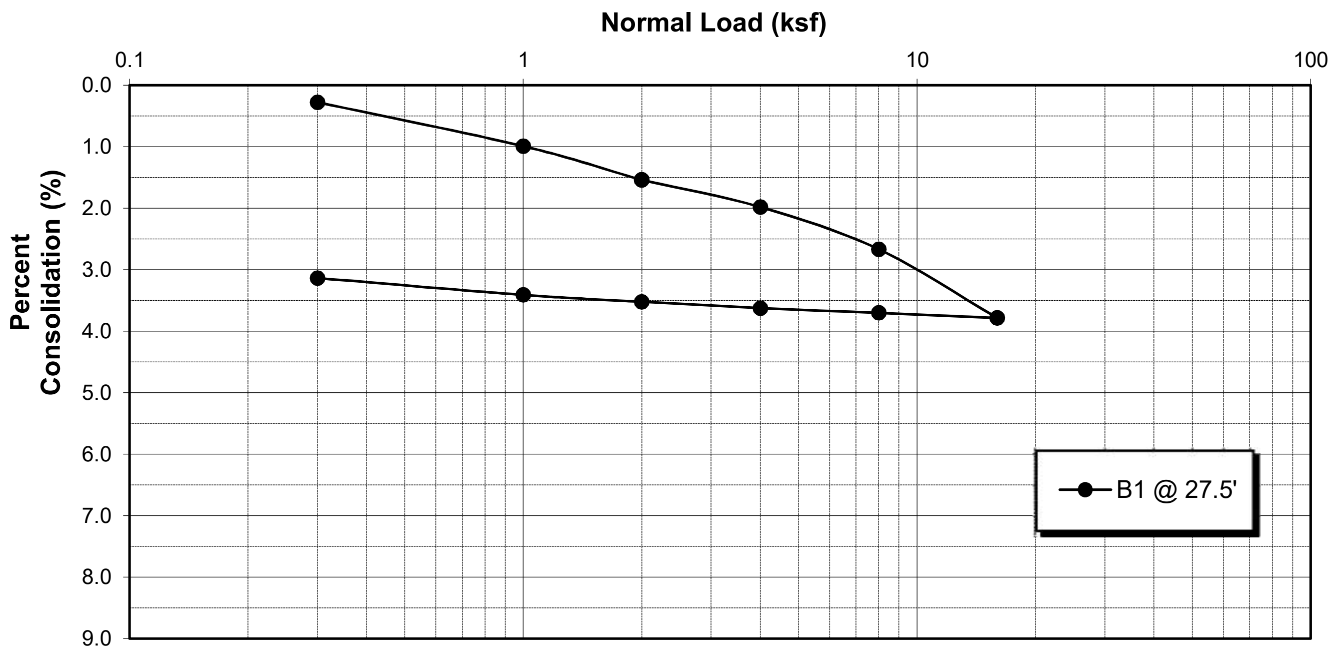
Water added at 2 KSF

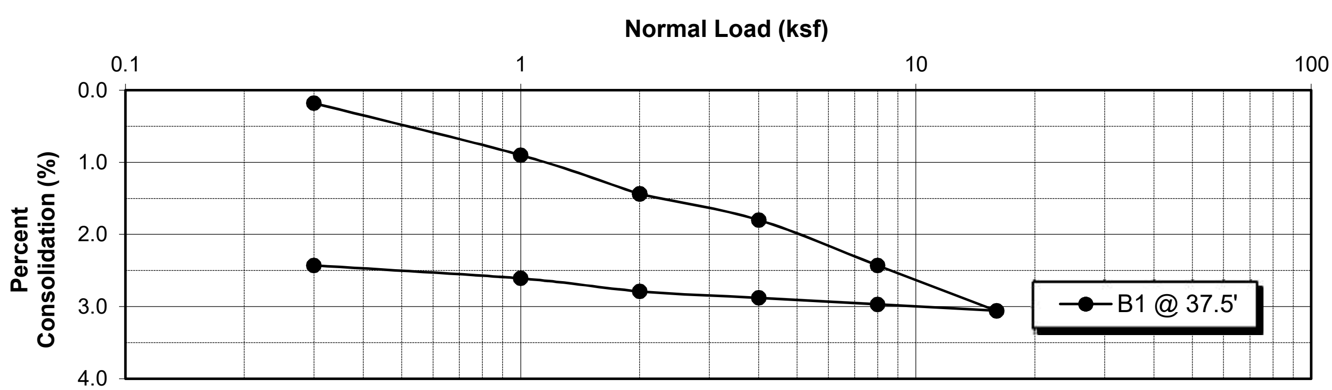
CONSOLIDATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: GRUBB PROPERTIES	
FILE NO.: 22207	PLATE: C-2







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: GRUBB PROPERTIES	
FILE NO.: 22207	PLATE: C-3

ASTM D-1557		
SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SOIL TYPE	ML/CL	ML/CL
MAXIMUM DENSITY PCF.	119.2	120.9
OPTIMUM MOISTURE %	13	12.5

ASTM D 4829		
SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SOIL TYPE	ML/CL	ML/CL
EXPANSION INDEX UBC STANDARD 18-2	94	106
EXPANSION CHARACTER	HIGH	HIGH

SULFATE CONTENT		
SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SULFATE CONTENT: (PERCENTAGE BY WEIGHT)	<0.1%	<0.1%

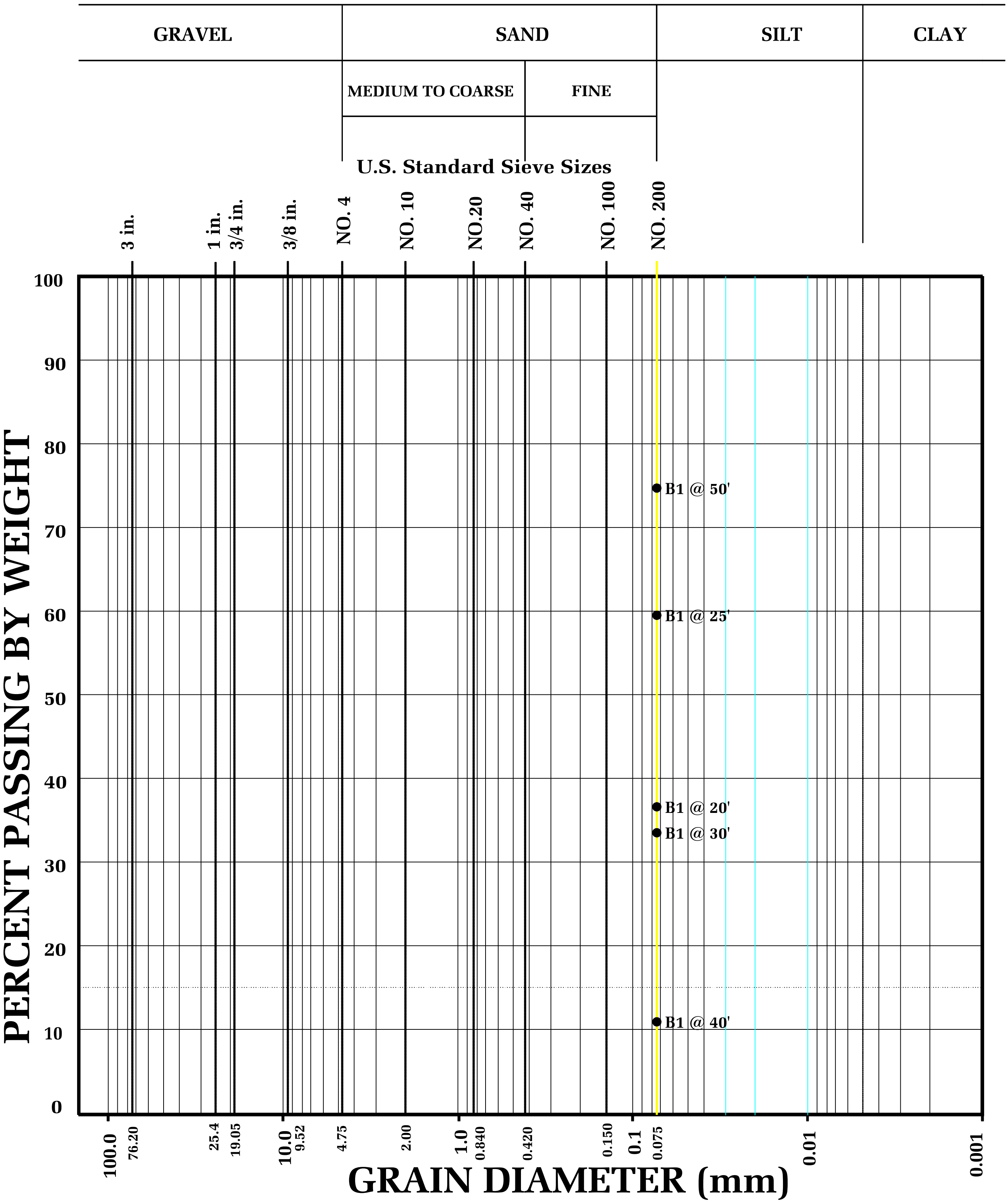
COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

GRUBB PROPERTIES
1200 NORTH VINE STREET, LOS ANGELES

FILE NO. 22207

PLATE: D-1



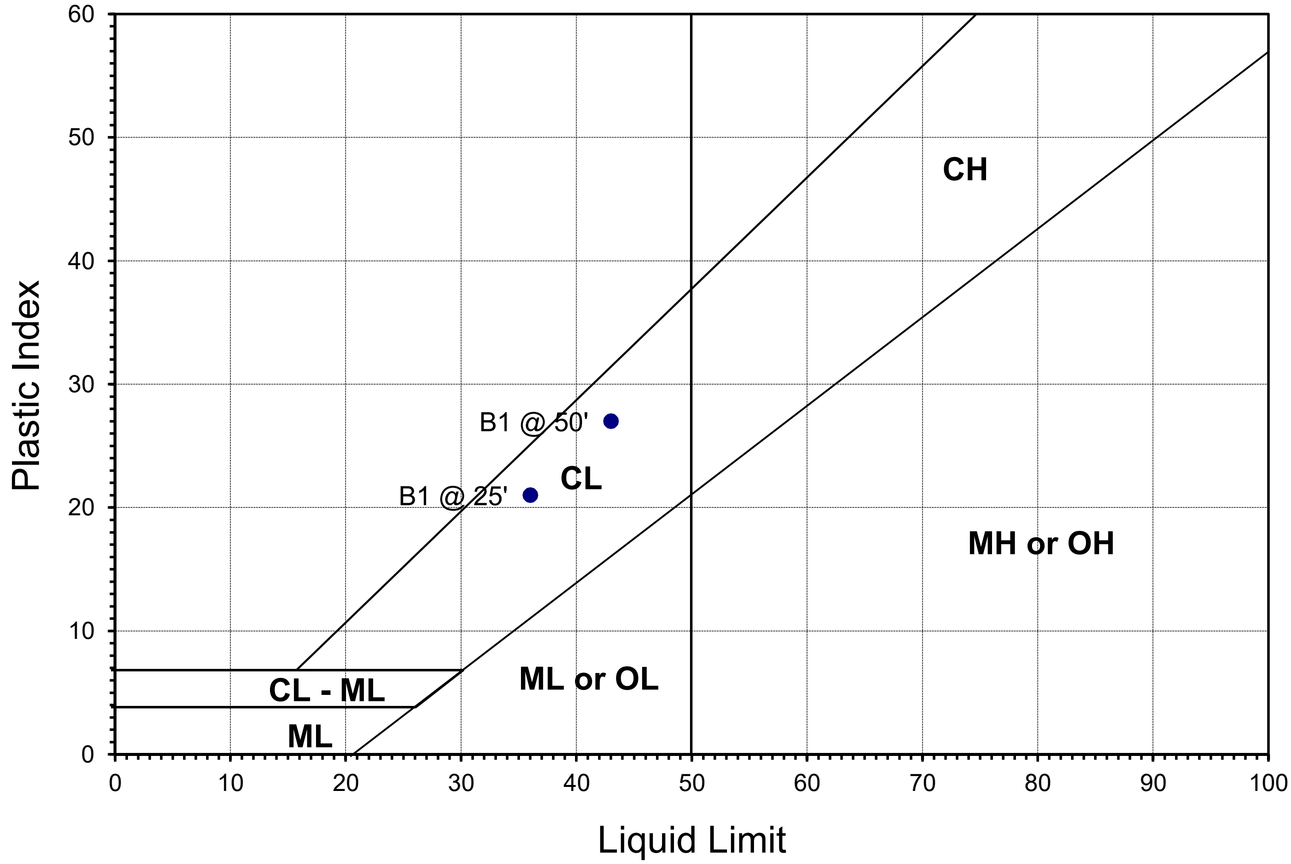
GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

GRUBB PROPERTIES
1200 NORTH VINE STREET, LOS ANGELES

FILE NO. 22207

PLATE: E



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B1 @ 25'	CL	59.5	36.0	15.0	21.0
B1 @ 50'	CL	74.7	43.0	16.0	27.0
0	0	0.0	0.0	0.0	0.0
0	0	0.0	0.0	0.0	0.0
0	0	0.0	0.0	0.0	0.0
0	0	0.0	0.0	0.0	0.0
0	0	0.0	0.0	0.0	0.0
0	0	0.0	0.0	0.0	0.0
0	0	0.0	0.0	0.0	0.0
0	0	0.0	0.0	0.0	0.0

ATTERBERG LIMITS

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: GRUBB PROPERTIES

FILE NO.: 22207

PLATE: F-1

Geotechnologies, Inc.

Project: Grubb Properties
File No.: 22207
Description: Liquefaction Analysis
Boring No: B1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.99
Calculated Mag.Wtg.Factor:	1.203

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	20.0
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	Mag. Scaling Factor (Sand) MSF	Overburden Corr. Factor K_σ	Cyclic Resist. Ratio $CRR_{M7.5,\sigma vc'=1}$	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	131.9	Unsaturated	Unsaturated	15	5	0.0	0	131.9	131.9	35.1	1.00	0.646	1.20	1.10	1.120	1.481	Non-Liq.	0.00
2	131.9	Unsaturated	Unsaturated	15	5	0.0	0	263.8	263.8	35.1	1.00	0.644	1.20	1.10	1.120	1.481	Non-Liq.	0.00
3	131.9	Unsaturated	Unsaturated	15	5	0.0	0	395.7	395.7	35.1	1.00	0.642	1.20	1.10	1.120	1.481	Non-Liq.	0.00
4	131.9	Unsaturated	Unsaturated	15	5	0.0	0	527.6	527.6	33.1	0.99	0.640	1.20	1.10	0.775	1.025	Non-Liq.	0.00
5	131.9	Unsaturated	Unsaturated	15	5	0.0	0	659.5	659.5	33.2	0.99	0.637	1.20	1.10	0.784	1.037	Non-Liq.	0.00
6	131.9	Unsaturated	Unsaturated	15	5	0.0	0	791.4	791.4	31.2	0.99	0.635	1.20	1.10	0.573	0.758	Non-Liq.	0.00
7	131.9	Unsaturated	Unsaturated	15	5	0.0	0	923.3	923.3	29.4	0.98	0.633	1.20	1.10	0.450	0.595	Non-Liq.	0.00
8	122.0	Unsaturated	Unsaturated	15	5	0.0	0	1045.3	1045.3	28.0	0.98	0.630	1.20	1.10	0.382	0.505	Non-Liq.	0.00
9	122.0	Unsaturated	Unsaturated	15	5	0.0	0	1167.3	1167.3	28.5	0.97	0.627	1.20	1.10	0.403	0.534	Non-Liq.	0.00
10	122.0	Unsaturated	Unsaturated	15	5	0.0	0	1289.3	1289.3	27.3	0.97	0.625	1.20	1.09	0.356	0.466	Non-Liq.	0.00
11	122.0	Unsaturated	Unsaturated	13	10	0.0	0	1411.3	1411.3	22.5	0.97	0.622	1.20	1.06	0.240	0.306	Non-Liq.	0.00
12	122.0	Unsaturated	Unsaturated	13	10	0.0	0	1533.3	1533.3	21.6	0.96	0.619	1.20	1.05	0.227	0.285	Non-Liq.	0.00
13	128.0	Unsaturated	Unsaturated	13	10	0.0	0	1661.3	1661.3	20.8	0.96	0.616	1.20	1.03	0.216	0.268	Non-Liq.	0.00
14	128.0	Unsaturated	Unsaturated	13	10	0.0	0	1789.3	1789.3	20.0	0.95	0.613	1.20	1.02	0.206	0.253	Non-Liq.	0.00
15	128.0	Unsaturated	Unsaturated	13	10	0.0	0	1917.3	1917.3	21.9	0.95	0.610	1.20	1.01	0.232	0.283	Non-Liq.	0.00
16	128.0	Unsaturated	Unsaturated	15	15	0.0	0	2045.3	2045.3	25.1	0.94	0.607	1.20	1.01	0.291	0.352	Non-Liq.	0.00
17	128.0	Unsaturated	Unsaturated	15	15	0.0	0	2173.3	2173.3	24.3	0.94	0.603	1.20	1.00	0.275	0.329	Non-Liq.	0.00
18	126.1	Unsaturated	Unsaturated	15	15	0.0	0	2299.4	2299.4	23.7	0.93	0.600	1.20	0.99	0.261	0.310	Non-Liq.	0.00
19	126.1	Unsaturated	Unsaturated	15	15	0.0	0	2425.5	2425.5	23.0	0.93	0.597	1.20	0.98	0.250	0.294	Non-Liq.	0.00
20	126.1	Unsaturated	Unsaturated	15	15	0.0	0	2551.6	2551.6	22.4	0.92	0.593	1.20	0.97	0.240	0.281	Non-Liq.	0.00
21	126.1	Saturated	Saturated	19	20	36.6	0	2677.7	2615.3	35.2	0.92	0.604	1.20	0.94	1.148	1.302	2.2	0.00
22	126.1	Saturated	Saturated	19	20	36.6	0	2803.8	2679.0	34.8	0.91	0.614	1.20	0.94	1.071	1.208	2.0	0.00
23	136.4	Saturated	Saturated	19	20	36.6	0	2940.2	2753.0	34.5	0.91	0.623	1.20	0.93	0.992	1.112	1.8	0.00
24	136.4	Saturated	Saturated	19	20	36.6	0	3076.6	2827.0	34.1	0.90	0.631	1.20	0.93	0.923	1.029	1.6	0.00
25	136.4	Saturated	Saturated	19	20	36.6	0	3213.0	2901.0	33.7	0.89	0.638	1.20	0.92	0.864	0.958	1.5	0.00
26	136.4	Saturated	Saturated	11	25	59.5	21	3349.4	2975.0	19.9	0.89	0.644	1.20	0.95	0.204	0.234	Non-Liq.	0.00
27	136.4	Saturated	Saturated	11	25	59.5	21	3485.8	3049.0	19.7	0.88	0.650	1.20	0.95	0.202	0.231	Non-Liq.	0.00
28	133.7	Saturated	Saturated	11	25	59.5	21	3619.5	3120.3	20.3	0.88	0.655	1.20	0.95	0.210	0.239	Non-Liq.	0.00
29	133.7	Saturated	Saturated	11	25	59.5	21	3753.2	3191.6	20.1	0.87	0.660	1.20	0.94	0.208	0.236	Non-Liq.	0.00
30	133.7	Saturated	Saturated	11	25	59.5	21	3886.9	3262.9	20.0	0.87	0.664	1.20	0.94	0.205	0.233	Non-Liq.	0.00
31	133.7	Saturated	Saturated	21	30	33.5	0	4020.6	3334.2	37.6	0.86	0.668	1.20	0.86	2.000	2.000	3.0	0.00
32	133.7	Saturated	Saturated	21	30	33.5	0	4154.3	3405.5	37.3	0.85	0.671	1.20	0.86	1.865	1.925	2.9	0.00
33	137.5	Saturated	Saturated	21	30	33.5	0	4291.8	3480.6	36.9	0.85	0.673	1.20	0.85	1.713	1.757	2.6	0.00
34	137.5	Saturated	Saturated	21	30	33.5	0	4429.3	3555.7	36.6	0.84	0.675	1.20	0.85	1.581	1.615	2.4	0.00
35	137.5	Saturated	Saturated	21	30	33.5	0	4566.8	3630.8	36.3	0.84	0.677	1.20	0.85	1.465	1.491	2.2	0.00
36	137.5	Saturated	Saturated	31	35	0.0	0	4704.3	3705.9	49.9	0.83	0.678	1.20	0.83	2.000	2.000	2.9	0.00
37	137.5	Saturated	Saturated	31	35	0.0	0	4841.8	3781.0	49.7	0.82	0.679	1.20	0.83	2.000	1.991	2.9	0.00
38	128.1	Saturated	Saturated	26	40	10.9	0	4969.9	3846.7	42.1	0.82	0.680	1.20	0.82	2.000	1.979	2.9	0.00
39	128.1	Saturated	Saturated	26	40	10.9	0	5098.0	3912.4	41.8	0.81	0.681	1.20	0.82	2.000	1.967	2.9	0.00
40	128.1	Saturated	Saturated	26	40	10.9	0	5226.1	3978.1	41.5	0.81	0.682	1.20	0.81	2.000	1.955	2.9	0.00
41	128.1	Saturated	Saturated	26	40	10.9	0	5354.2	4043.8	41.2	0.80	0.682	1.20	0.81	2.000	1.943	2.8	0.00
42	128.1	Saturated	Saturated	26	40	10.9	0	5482.3	4109.5	40.9	0.79	0.682	1.20	0.80	2.000	1.932	2.8	0.00
43	137.4	Saturated	Saturated	35	45	0.0	0	5619.7	4184.5	54.6	0.79	0.681	1.20	0.80	2.000	1.919	2.8	0.00
44	137.4	Saturated	Saturated	35	45	0.0	0	5757.1	4259.5	54.4	0.78	0.681	1.20	0.79	2.000	1.906	2.8	0.00
45	137.4	Saturated	Saturated	35	45	0.0	0	5894.5	4334.5	54.1	0.78	0.680	1.20	0.79	2.000	1.894	2.8	0.00
46	137.4	Saturated	Saturated	35	45	0.0	0	6031.9	4409.5	53.9	0.77	0.678	1.20	0.78	2.000	1.882	2.8	0.00
47	137.4	Saturated	Saturated	35	45	0.0	0	6169.3	4484.5	53.6	0.76	0.677	1.20	0.78	2.000	1.870	2.8	0.00
48	133.9	Saturated	Saturated	35	45	0.0	0	6303.2	4556.0	53.4	0.76	0.676	1.20	0.77	2.000	1.859	2.8	0.00
49	133.9	Saturated	Saturated	35	45	0.0	0	6437.1	4627.5	53.2	0.75	0.674	1.20	0.77	2.000	1.848	2.7	0.00
50	133.9	Saturated	Saturated	20	50	74.7	27	6571.0	4699.0	30.7	0.75	0.673	1.20	0.83	0.529	0.530	Non-Liq.	0.00

Geotechnologies, Inc.

Project: Grubb Properties

File No.: 22207

Settlement Calculation - Column Footing

Description: Column footing bearing in compacted fill

Soil Unit Weight	120.0 pcf	Column Footing	
Bearing Value	5000.0 psf	690 kips	
Depth of Footing	3.0 feet		
Width of Footing	11.75 feet		

* Influence Values are based on Westergaard's Analyses (Ref: Sowers)

Depth Below Basement Surbgrade (feet)	Average Depth Below Ground Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (a/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
3.0													
	4.5	1.5	7.8	83%	4153.875	540	4693.875	B1 @ 1-5'	0.80	0.25	0.55	3.0	0.20
6.0													
	8.0	5.0	2.4	50%	2493.75	960	3453.75	B1 @ 7.5'	1.25	0.60	0.65	4.0	0.31
10.0													
	12.5	9.5	1.2	29%	1429	1500	2929	B1 @ 12.5'	1.05	0.50	0.55	5.0	0.33
15.0													
	21.3	18.3	0.6	10%	499.5	2550	3049.5	B2 @ 20'	1.50	1.40	0.10	12.5	0.15
27.5													

Settlement: 0.99

Total Settlement in inches: ***0.99***

Geotechnologies, Inc.

Project: Grubb Properties

File No.: 22207

Settlement Calculation - Column Footing

Description: Column footing bearing in native soils

Soil Unit Weight	120.0 pcf	Column	Footing
Bearing Value	5000.0 psf	690	kips
Depth of Footing	3.0 feet		
Width of Footing	11.75 feet		

* Influence Values are based on Westergaard's Analyses (Ref: Sowers)

Depth Below Basement Surbgrade (feet)	Average Depth Below Ground Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (a/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
3.0													
	6.5	3.5	3.4	63%	3154.5	780	3934.5	B1 @ 7.5'	1.30	0.55	0.75	7.0	0.63
10.0													
	12.5	9.5	1.2	29%	1429	1500	2929	B1 @ 12.5'	1.07	0.52	0.55	5.0	0.33
15.0													
	21.3	18.3	0.6	10%	499.5	2550	3049.5	B2 @ 20'	1.50	1.47	0.03	12.5	0.05
27.5													

Settlement: 1.01

Total Settlement in inches: ***1.01***

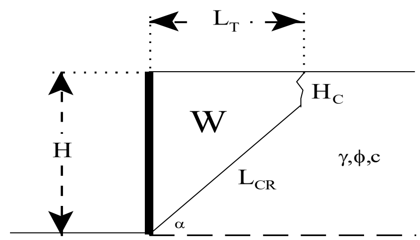
Geotechnologies, Inc.

Project: Grubb Properties
File No.: 22207
Description: Drained Catilever Retaining Wall (up to 6 feet)

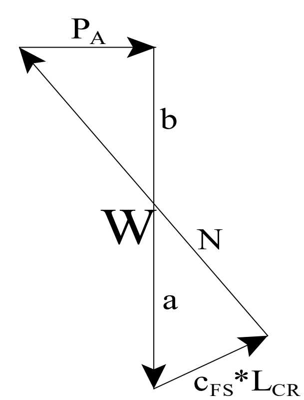
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	6.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	26.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	18.0	degrees
	84.2	160.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.4	10	1176.6	2.5	998.3	178.3	72.0
41	4.3	10	1207.4	2.6	1008.4	199.0	84.4
42	4.2	10	1225.1	2.7	1008.5	216.6	96.4
43	4.1	10	1232.5	2.8	1001.2	231.2	107.8
44	4.0	10	1231.3	2.8	988.4	242.9	118.4
45	4.0	10	1223.1	2.9	971.4	251.8	128.2
46	3.9	10	1209.3	2.9	951.2	258.1	137.1
47	3.8	10	1190.7	3.0	928.8	261.9	145.1
48	3.8	10	1168.3	3.0	904.8	263.5	152.1
49	3.8	10	1142.7	3.0	879.5	263.1	158.0
50	3.7	9	1114.3	3.0	853.5	260.8	162.9
51	3.7	9	1083.7	3.0	826.9	256.9	166.7
52	3.7	9	1051.3	2.9	799.9	251.4	169.5
53	3.7	8	1017.2	2.9	772.7	244.5	171.1
54	3.7	8	981.8	2.9	745.4	236.4	171.7
55	3.7	8	945.2	2.8	718.0	227.2	171.2
56	3.7	8	907.7	2.8	690.6	217.1	169.5
57	3.7	7	869.2	2.7	663.1	206.1	166.8
58	3.7	7	829.9	2.7	635.6	194.3	163.0
59	3.8	7	789.9	2.6	608.0	181.9	158.1
60	3.8	6	749.3	2.6	580.2	169.1	152.2
61	3.8	6	708.0	2.5	552.2	155.8	145.2
62	3.9	6	666.0	2.4	523.8	142.2	137.2
63	4.0	5	623.4	2.3	495.0	128.4	128.3
64	4.0	5	580.1	2.2	465.6	114.5	118.5
65	4.1	4	536.2	2.1	435.5	100.7	107.9



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$
$b = W-a$
$P_A = b*\tan(\alpha-\phi_{FS})$
$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 171.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 9.5 pcf

Design Wall for an Equivalent Fluid Pressure: 45 pcf (High E.I.)

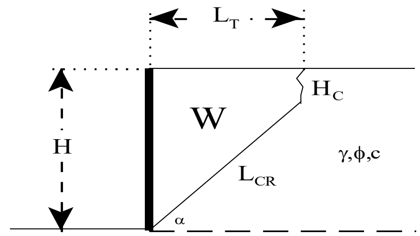
Geotechnologies, Inc.

Project: Grubb Properties
File No.: 22207
Description: Drained Catilever Retaining Wall (up to 6 feet)

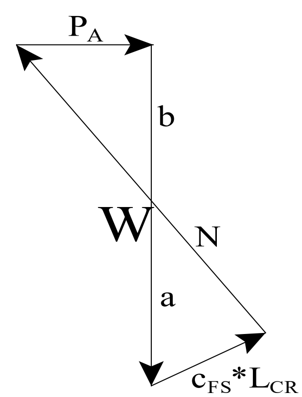
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	6.00 feet	
Unit Weight of Retained Soils	(γ)	57.6 pcf	**(Buoyant)**
Friction Angle of Retained Soils	(ϕ)	26.0 degrees	
Cohesion of Retained Soils	(c)	240.0 psf	
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	18.0 degrees	
	84.2	160.0 psf	



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	9.2	-29	-1675.9	-5.0	-2029.7	353.7	0.0
41	9.0	-25	-1468.6	-4.5	-1759.4	290.8	0.0
42	8.7	-22	-1293.9	-4.1	-1534.7	240.8	0.0
43	8.6	-20	-1146.2	-3.7	-1347.1	200.9	0.0
44	8.4	-18	-1021.1	-3.4	-1190.2	169.0	0.0
45	8.2	-16	-915.0	-3.2	-1058.5	143.5	0.0
46	8.1	-14	-825.0	-2.9	-948.1	123.0	0.0
47	8.0	-13	-748.8	-2.7	-855.4	106.6	0.0
48	7.9	-12	-684.3	-2.6	-777.8	93.5	0.0
49	7.8	-11	-630.0	-2.4	-713.0	83.0	0.0
50	7.8	-10	-584.5	-2.3	-659.1	74.7	0.0
51	7.7	-9	-546.6	-2.2	-614.8	68.2	0.0
52	7.7	-9	-515.6	-2.1	-578.7	63.2	0.0
53	7.7	-9	-490.5	-2.1	-549.9	59.5	0.0
54	7.6	-8	-470.7	-2.0	-527.6	56.8	0.0
55	7.7	-8	-455.7	-2.0	-510.9	55.2	0.0
56	7.7	-8	-445.0	-2.0	-499.5	54.5	0.0
57	7.7	-8	-438.2	-2.0	-492.9	54.6	0.0
58	7.8	-8	-435.1	-2.1	-490.6	55.6	0.0
59	7.8	-8	-435.2	-2.1	-492.5	57.3	0.0
60	7.9	-8	-438.5	-2.2	-498.4	59.9	0.0
61	8.0	-8	-444.8	-2.3	-508.1	63.3	0.0
62	8.1	-8	-453.9	-2.4	-521.6	67.7	0.0
63	8.2	-8	-465.8	-2.5	-538.8	73.0	0.0
64	8.4	-8	-480.5	-2.6	-560.0	79.5	0.0
65	8.5	-9	-497.9	-2.8	-585.1	87.2	0.0



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 0.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 0.0 pcf

Design Wall for an Equivalent Fluid Pressure: 98 pcf (Includes Hydrostatic Pressure)

Geotechnologies, Inc.

Project: Grubb Properties **DRAINED RESTRAINED RETAINING WALL**
File No.: 22207

Soil Weight	γ	120	pcf
Internal Friction Angle	ϕ	26	degrees
Cohesion	c	0	psf
Height of Retaining Wall	H	6	feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

$\sigma'_h = K_o\sigma'_v$

$K_o = 1 - \sin\phi$ 0.562

$\sigma'_v = \gamma H$ 720.0 psf

$\sigma'_h =$ 404.4 psf

EFP = 67.4 pcf

$P_o =$ 1213.1 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 68 pcf

Geotechnologies, Inc.

Project:	Grubb Properties	**UNDRAINED RESTRAINED RETAINING WALL**
File No.:	22207	

Soil Weight	γ	57.6 pcf	**(Buoyant)**
Internal Friction Angle	ϕ	26 degrees	
Cohesion	c	0 psf	
Height of Retaining Wall	H	6 feet	

Restrained Retaining Wall Design based on At Rest Earth Pressure

$\sigma'_h = K_o\sigma'_v$

$K_o = 1 - \sin\phi$	0.562
$\sigma'_v = \gamma H$	345.6 psf

σ'_h =	194.1 psf	
EFP =	32.3 pcf	
P_o =	582.3 lbs/ft	(based on a triangular distribution of pressure)

Design wall for an EFP of 93 pcf **(Includes Hydrostatic Pressure)**

Geotechnologies, Inc.

Project: GRUBB PROPERTIES
File No.: 22207
Description: Slot Cut

Slot Cut Calculation

Input:

Height of Slots	(H)	7	feet
Unit Weight of Soils	(γ)	120.0	pcf
Friction Angle of Soils	(ϕ)	26.0	degrees
Cohesion of Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.5	

Surcharge Pressure:

Line Load	(q_L)	2500.0	plf
Distance Away from Edge of Excavation	(X)	0.0	feet

Design Equations

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

$A = 0.5*H*b$

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

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

$F_2 = d*L$

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

$R_2 = 2*\Delta F$

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

FS = Resistance Force/Driving Force

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

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
60	4.0	14	1697.4	1817.5	1481.8	4438.0	11.4
61	3.9	14	1629.7	1751.1	1404.7	4260.8	11.0
62	3.7	13	1563.2	1684.3	1330.1	4087.1	10.7
63	3.6	12	1498.0	1617.2	1257.9	3916.6	10.4
64	3.4	12	1433.9	1550.0	1188.1	3749.1	10.1
65	3.3	11	1370.9	1482.7	1120.6	3584.4	9.9
66	3.1	11	1309.0	1415.3	1055.3	3422.4	9.7
67	3.0	10	1248.0	1348.0	992.2	3262.8	9.5
68	2.8	10	1187.8	1280.9	931.2	3105.7	9.4
69	2.7	9	1128.6	1214.0	872.2	2950.7	9.2
70	2.5	9	1070.1	1147.4	815.2	2797.8	9.1
71	2.4	8	1012.3	1081.2	760.0	2646.8	9.0
72	2.3	8	955.3	1015.5	706.8	2497.6	9.0
73	2.1	7	898.8	950.3	655.3	2350.1	8.9
74	2.0	7	843.0	885.8	605.6	2204.1	8.9
75	1.9	7	787.8	821.9	557.6	2059.7	8.8
76	1.7	6	733.0	758.9	511.2	1916.5	8.8
77	1.6	6	678.8	696.7	466.3	1774.6	8.8
78	1.5	5	624.9	635.5	423.0	1633.9	8.9
79	1.4	5	571.5	575.3	381.1	1494.2	8.9
80	1.2	4	518.4	516.2	340.6	1355.4	9.0
81	1.1	4	465.7	458.2	301.5	1217.5	9.1
82	1.0	3	413.2	401.5	263.6	1080.3	9.1
83	0.9	3	361.0	346.1	227.0	943.8	9.3
84	0.7	3	309.0	292.0	191.5	807.9	9.4
85	0.6	2	257.2	239.4	157.2	672.5	9.5

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

d_{allow} 8.8 feet

The proposed excavation may be made using the A-B-C Slot-Cutting Method with a Maximum Allowable Slot Width of 8 Feet, and up to 7 Feet in Height, with a Factor of Safety Equal or Exceeding 1.25.

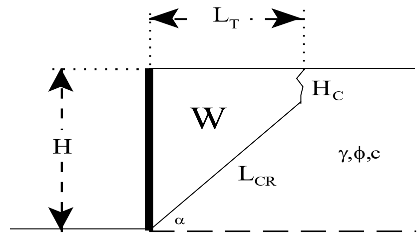
Geotechnologies, Inc.

Project: Grubb Properties
File No.: 22207
Description: Temporary Shoring (up to 10 feet)

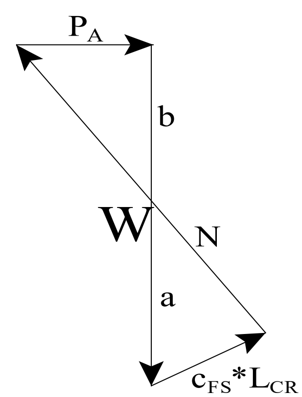
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	26.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	21.3	degrees
	84.2	192.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.1	38	4512.8	6.1	3410.4	1102.3	372.8
41	5.9	38	4529.4	6.3	3348.3	1181.2	422.6
42	5.7	38	4515.1	6.5	3270.5	1244.6	469.9
43	5.5	37	4476.7	6.6	3182.9	1293.8	514.5
44	5.4	37	4419.6	6.7	3089.3	1330.3	556.1
45	5.2	36	4347.8	6.7	2992.6	1355.2	594.5
46	5.1	36	4264.6	6.8	2894.9	1369.7	629.6
47	5.0	35	4172.4	6.8	2797.3	1375.1	661.3
48	5.0	34	4073.2	6.8	2701.0	1372.1	689.7
49	4.9	33	3968.5	6.8	2606.6	1361.9	714.6
50	4.8	32	3859.6	6.7	2514.4	1345.1	736.0
51	4.8	31	3747.3	6.7	2424.8	1322.6	753.9
52	4.7	30	3632.6	6.7	2337.7	1294.9	768.4
53	4.7	29	3516.0	6.6	2253.3	1262.7	779.4
54	4.7	28	3398.0	6.6	2171.6	1226.4	786.9
55	4.7	27	3278.9	6.5	2092.3	1186.6	790.9
56	4.7	26	3159.1	6.4	2015.4	1143.7	791.5
57	4.7	25	3038.8	6.3	1940.8	1098.0	788.5
58	4.7	24	2918.1	6.2	1868.2	1049.9	782.1
59	4.7	23	2797.2	6.1	1797.5	999.7	772.2
60	4.8	22	2676.1	6.0	1728.4	947.7	758.9
61	4.8	21	2554.9	5.9	1660.6	894.2	742.0
62	4.9	20	2433.5	5.8	1594.1	839.5	721.7
63	4.9	19	2312.0	5.7	1528.3	783.7	697.9
64	5.0	18	2190.3	5.5	1463.2	727.1	670.6
65	5.1	17	2068.3	5.4	1398.3	670.0	639.9



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 791.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 15.8 pcf

Design Shoring for an Equivalent Fluid Pressure: 28 pcf

Soil Corrosivity Evaluation Report for Grubb Properties

December 7, 2021

Prepared for:
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Project X Job #: S211203F
Client Job or PO #: 22207

Contents

1 Executive Summary

A corrosion evaluation of the soils at Grubb Properties was performed to provide corrosion control recommendations for general construction materials. The site is located at 1200 North Vine St, Los Angeles, CA. Two (2) samples were tested to a depth of 20 ft. Site ground water and topography information was provided by Geotechnologies, Inc.. Groundwater depth was determined to be 20 feet below finished grade.

Every material has its weakness. Aluminum alloys, galvanized/zinc coatings, and copper alloys do not survive well in very alkaline or very acidic pH environments. Copper and brasses do not survive well in high nitrate or ammonia environments. Steels and irons do not survive well in low soil resistivity and high chloride environments. High chloride environments can even overcome and attack steel encased in normally protective concrete. Concrete does not survive well in high sulfate environments. And nothing survives well in high sulfide and low redox potential environments with corrosive bacteria. This is why Project X tests for these 8 factors to determine a soil's corrosivity towards various construction materials. **Depending solely on soil resistivity or Caltrans corrosion guidelines (which concentrate on concrete/steel highways), will over-simplify descriptions as corrosive or non-corrosive. This approach will not detect these other factors attacking other metals because it is possible to have bad levels of corrosive ions and still have greater than 1,100 ohm-cm soil resistivity. We have observed this fact on thousands of soil samples tested in our laboratory.**

It should not be forgotten that import soil should also be tested for all factors to avoid making your site more corrosive than it was to begin with.

The recommendations outlined herein are not a substitute for any design documents previously prepared for the purpose of construction and apply only to the depth of samples collected.

Soil samples were tested for minimum resistivity, pH, chlorides, sulfates, ammonia, nitrates, sulfides and redox.

As-Received soil resistivities ranged between 14,740 ohm-cm and 20,770.0 ohm-cm. This data would be similar to a Wenner 4 pin test in the field and used in the design of a cathodic protection or grounding bed system. This resistivity can change seasonally depending on the weather and moisture in the ground. This reading alone can be misleading because condensation or minor water leaks will occur underground along pipe surfaces creating a saturated soil environment in the trench on infrastructure surfaces. This is why minimum or saturated soil resistivity measurements are more important than as-received resistivities.

Saturated soil resistivities ranged between 2,010 ohm-cm to 2,211 ohm-cm. The worst of these values is considered to be moderately corrosive to general metals.

PH levels ranged between 8.0 to 8.1 pH. PH levels were determined to be at levels not detrimental to copper or aluminum alloys. The pH of these samples can allow corrosion of steel and iron in moist environments.

Chlorides ranged between 6 mg/kg to 17 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals.

Sulfates ranged between 33 mg/kg to 36 mg/kg. Sulfate levels in these samples are negligible for corrosion of cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 0.6 mg/kg to 1.8 mg/kg. Nitrates ranged between 24.4 mg/kg to 36.8 mg/kg. Concentrations of these elements were not high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 210 mV to + 215 mV. The probability of corrosive bacteria was determined to be low due to the sulfide and positive REDOX levels determined in these samples.

2 Corrosion Control Recommendations

The following recommendations are based upon the results of soil testing.

2.1 Cement

The highest reading for sulfates was 36 mg/kg or 0.0036 percent by weight.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in these samples categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement not containing steel or other metal can be used.

2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C)

Chlorides in soil can overcome the corrosion inhibiting property of cement for steel, as it can also break through passivated surfaces of aluminum and stainless steels.[1,2] The highest concentration of chlorides was 17 mg/kg.

Chloride levels in these samples are not significantly corrosive to metals not in tension. Standard cement cover may be used in these soils.

Though soils at some locations are significantly corrosive to various metals, per ACI 318-14 Chapter 19 Table 19.3.1.1, all slabs on this site exposure categories and class for **Corrosion Protection of Reinforcement (C) would be considered C1** as Concrete exposed to moisture [mud/rain] (slab sides and bottom) but not to an external source of chlorides. Though there are chlorides in the soil, ACI 318's definition of "external source of chlorides" consists of deicing chemicals, salt, brackish water, seawater, or spray from these sources. The chloride levels in seawater are typically over 19,000 mg/L or 19,000 ppm.

When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Per ACI 318-14 Table 5.3.2.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.[3]

[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65
[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*

[3] ACI 381-14., BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)

2.3 Stainless Steel Pipe/Conduit/Fittings

Stainless steels derive their corrosion resistance from their chromium content and oxide layer which needs oxygen to regenerate if damaged. Thus stainless steel is not good for deep soil applications where oxygen levels are extremely low. Stainless steels should not be installed deeper than a plant root zone. Stainless steels typically have the same nobility as copper on the galvanic series and can be connected to copper. If stainless steel must be used, it must be backfilled with soil having greater than 10,000 ohm-cm resistivity and excellent drainage. 304 Stainless steel will also corrode if in contact with carbon materials such as activated carbon. Stainless steel welds should be pickled.

The soil at this site has low probability for anaerobic corrosive bacteria and low chloride levels. Per Nickel Institute guidelines, 304 or 316 Stainless steels can be used in these soils.

2.4 Steel Post Tensioning Systems

The proper sealing of stressing holes is of utmost importance in PT Systems. Cut off excess strand 1/2" to 3/4" back in the hole. Coat or paint exposed anchorage, grippers, and stub of strands with "Rust-o-leum" or equal. After tendons have been coated, the cement contractor shall dry pack blockouts within ten (10) days. A non-shrink, non-metallic, non-porous moisture-insensitive grout (Master EMACO S 488 or equivalent), or epoxy grout shall be used for this purpose. If an encapsulated post-tension system is used, regular non-shrink grout can be used.

Due to the low chloride concentrations measured on samples obtained from this site, post-tensioned slabs should be protected in accordance with soil considered normal (non-corrosive).[4,5] Addition of grease caps to the cut strand at live end anchors can deter construction defect accusations but are not needed.

2.5 Steel Piles

Steel piles are most susceptible to corrosion in disturbed soil where oxygen is available. Further, a dissimilar environment corrosion cell would exist between the steel embedded in cement, such as pile caps and the steel in the soil. In the cell, the steel in the soil is the anode (corroding metal), and the steel in cement is the cathode (protected metal). This cell can be minimized by coating the part of the steel piles that will be embedded in cement to prevent contact with cement and reinforcing steel.

Piles driven into soils without disturbing soils will avoid oxygen introduction and low corrosion rates unless there is a probability for corrosive anaerobic bacteria. Galvanized steel's zinc coating can provide significant protection for driven piles. In corrosive soils in which normal zinc coatings are not enough, the life of piles can be extended by increasing zinc coating thickness, using sacrificial metal, or providing a combination of epoxy coatings and cathodic protection. Corrosion has been observed to be extremely localized even at and below underground water tables. Pit depths of this magnitude do not have an appreciable effect on the strength or useful life of piling structures because the reduction in pile cross section is not

[4] *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12,Table 4.1, pg 16*
[5] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

significant.[6] Pitting is of more importance to pipes transporting liquids or gases which should not be leaked into the ground.

The following recommendations are recommended to achieve desired life. We defer to structural engineers to use our estimated corrosion rates and to choose from the corrosion control options listed below.

1) Sacrificial metal by use of thicker piles per non-disturbed soil corrosion rates, or
2) Galvanized steel piles per non-disturbed soil corrosion rates, or
3) Combination of galvanized and sacrificial metal per non-disturbed soil corrosion rates, or
4) For no loss of metal, coat entire pile with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or
5) Use high yield steel which will corrode at the same rate as mild steel but have greater yield strength and thus be able to suffer more material loss than mild steel.

2.5.1 Expected Corrosion Rate of Steel and Zinc in disturbed soil

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

In Melvin Romanoff's NBS Circular 579, the corrosion rates of carbon steels and various metals was studied over long term periods. Various metals were placed in various soil types to gather corrosion rate data of all metals in all soil types. Samples were collected and material loss measured over the course of 20 years in some sites. The following corrosion rates were estimated by comparing the worst results of soils tested with similar soils in Romanoff's studies and Highway Research Board's publications.[7] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[8]

Expected Corrosion Rate for Steel = 1.53 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.34 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

In undisturbed soils, a corrosion rate of 1 mil/year for steel is expected with little change in the corrosion rate of zinc due to it's low nobility in the galvanic series.

Per CTM 643: Years to perforation of corrugated galvanized steel culverts

- 33.9 Years to Perforation for a 18 gage metal culvert
- 44.1 Years to Perforation for a 16 gage metal culvert

[6] Melvin Romanoff, Corrosion of Steel Pilings in Soils, National Bureau of Standards Monograph 58, pg 20.

[7] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962

[8] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

- 54.3 Years to Perforation for a 14 gage metal culvert
- 74.6 Years to Perforation for a 12 gage metal culvert
- 94.9 Years to Perforation for a 10 gage metal culvert
- 115.3 Years to Perforation for a 8 gage metal culvert

2.5.2 ***Expected Corrosion Rate of Steel and Zinc in Undisturbed soil***

Expected Corrosion Rate for Steel = 1 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.34 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

2.6 Steel Storage tanks

Underground fuel tanks must be constructed and protected in accordance with California Underground Storage Tank Regulations, CCR, Title 23, Division 3, Chapter 16. Metals should be protected with cathodic protection or isolated from backfill material with an epoxy coating.

2.7 Steel Pipelines

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines

6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286 to avoid galvanic corrosion cells. **These are especially important for fire risers.**

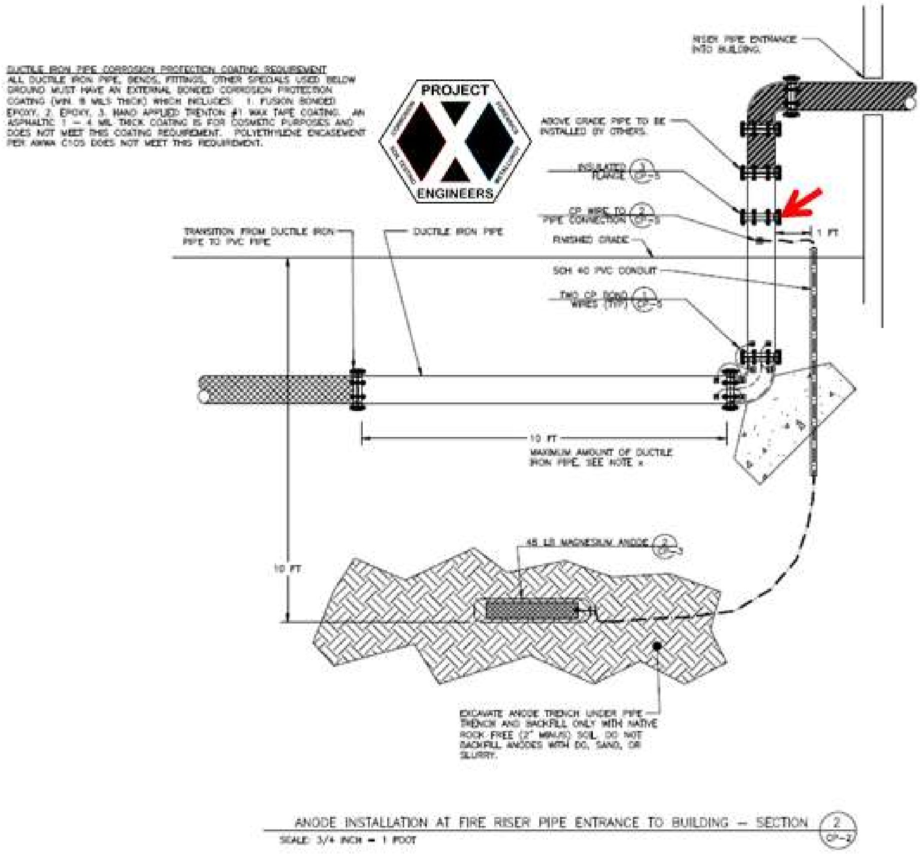


Figure 1- Fire Riser Detail: Install Isolation joint at red arrow

The bare steel surfaces, the corrosivity at this site is mildly corrosive to steel. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213, or
6) For bare steel surfaces, such as welded pipe joints, apply 3 inch thick field coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide. (For CML&C pipes, CML&C factory applied 3/4 inch thick coating is equivalent and needs no extra thickness added.)

It is critical for the life of the pipe that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.8 Steel Fittings

The corrosivity at this site is mildly corrosive to steel. The corrosion control options for this site can be one of the following:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Use powder coated steel with minimum 60 micron (2-3 mil) thick coating[9], or
7) Galvanized steel, or
8) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.9 Ductile Iron (DI) & Cast Iron Fittings

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 1 out of 25.5. A score greater or equal

[9] Manish Kumar Bhadu, Akshya Kumar Guin, Veena Singh, Shyam K. Choudhary, "Corrosion Study of Powder-Coated Galvanised Steel", International Scholarly Research Notices, vol. 2013, Article ID 464710, 9 pages, 2013

to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[10]

The corrosivity at this site is mildly corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.10 Ductile Iron & Cast Iron Pipe

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 1 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[11]

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection. **If using thermite, perform one**

[10] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

[11] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

test bond using a half-charge then pressure test to confirm excess heat and pinholes were not created.

Pea gravel is used by plumbers to lay pipes and establish slopes. If the gravel has more than 200 ppm chlorides or is not tested, a 25 mil plastic should be placed between the gravel and pipe to avoid corrosion.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286**. These are especially important for fire risers.**

The corrosivity at this site is mildly corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11 Copper Materials

Copper is an amphoteric material which is susceptible to corrosion at very high and very low pH. It is one of the most noble metals used in construction thus typically making it a cathode when connected to dissimilar metals. Copper's nobility can change with temperature, similar to the phenomenon in zinc. When zinc is at room temperature, it is less noble than steel and can provide cathodic protection to steel. But when zinc is at a temperature above 140F such as in a water heater, it becomes more noble than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Cold copper has one native potential, but when heated it develops a more electronegative electro-potential aka open circuit potential. Thus hot and cold copper pipes should be electrically isolated from each other to avoid creation of a thermo-galvanic corrosion cell.

2.11.1 __Copper Pipes__

The lowest pH for this area was measured to be 8.0. Copper is greatly affected by pH, ammonia and nitrate concentrations[12]. The highest nitrate concentration was 36.8 mg/kg and the highest ammonia concentration was 1.8 mg/kg at this site.

These soils were determined mildly corrosive to copper and copper alloys such as brass.

Underground, aboveground, cold water, and hot water pipes should be electrically isolated from each other by use of dielectric unions and plastic in-wall pipe supports per NACE SP0286. The following are corrosion control options for underground copper water pipes.

1) Cover cold copper piping with minimum 8 mil polyethylene and backfill with clean sand with 2 inch minimum cover above and below tubing. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm
2) Heat increases corrosion rates. Hot water pipes should be installed within PVC piping to prevent soil contact, or
3) Cover hot water pipes with minimum 8 mil polyethylene sleeve or incase in double 4-mil thick polyethylene sleeves over a suitable primer

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.2 __Brass Fittings__

Brass fittings should be electrically isolated from dissimilar metals by use of dielectric unions or isolation joint kits per NACE SP0286.

[12] Corrosion Data Handbook, Table 6, Corrosion Resistance of copper alloys to various environments, 1995

These soils were determined to be mildly corrosive to copper and copper alloys such as brass.

The following are corrosion control options for underground brass.

1) Cover with minimum 10 mil polyethylene or other impermeable coating and backfill with clean sand with 4 inch minimum cover above and below brass. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm, or
2) Wrap fitting or valves in wax tape

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.3 Bare Copper Grounding Wire

It is assumed that corrosion will occur at all sides of the bare wire, thus the corrosion rate is calculated as a two sided attack determining the time it takes for the corrosion from two sides to meet at the center of the wire. The estimated life of bare copper wire for this site is the following:[13]

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
14	64.1	1068.3
13	72	1200.0
12	80.8	1346.7
11	90.7	1511.7
10	101.9	1698.3
9	114.4	1906.7
8	128.5	2141.7
7	144.3	2405.0
6	162	2700.0
5	181.9	3031.7
4	204.3	3405.0
3	229.4	3823.3
2	257.6	4293.3
1	289.3	4821.7

If the bare copper wire is being used as a grounding wire connected to less noble metals such as galvanized steel or carbon steel, the less noble metals will provide additional cathodic protection to the copper reducing the corrosion rate of the copper.

[13] Soil-Corrosion studies 1946 and 1948: Copper Alloys, Lead, and Zinc, Melvin Romanoff, National Bureau of Standards, Research Paper RP2077, 1950

It is recommended that a corrosion inhibiting and water-repelling coating be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion. This can be wax tape, or other epoxy coating.

Tinned copper wiring or laying copper wire in conductive concrete can protect against chemical attack in soils with high nitrates, ammonia, sulfide and severely low soil electrical resistivity.

2.12 Aluminum Pipe/Conduit/Fittings

Aluminum is an amphoteric material prone to pitting corrosion in environments that are very acidic or very alkaline or high in chlorides.

Conditions at this site are safe for aluminum.

Aluminum derives its corrosion resistance from its oxide layer which needs oxygen to regenerate if damaged, similar to stainless steels. Thus aluminum is not good for deep soil applications. Since aluminum corrodes at very alkaline environments, it cannot be encased or placed against cement or mortar such as brick wall mortar up against an aluminum window frame.

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints per NACE SP0286. Pooling of water at post bottoms or surfaces should be avoided by integrating good drainage.

2.13 Carbon Fiber or Graphite Materials

Carbon fiber or other graphite materials are extremely noble on the galvanic series and should always be electrically isolated from dissimilar metals. They can conduct electricity and will create corrosion cells if placed in contact within a moist environment with any metal.

2.14 Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping from a corrosion viewpoint.

Protect all metallic fittings and pipe restraining joints with wax tape per AWWA C217, cement if previously recommended, or epoxy.

3 CLOSURE

In addition to soils chemistry and resistivity, another contributing influence to the corrosion of buried metallic structures is stray electrical currents. These electrical currents flowing through the earth originate from buried electrical systems, grounding of electrical systems in residences, commercial buildings, and from high voltage overhead power grids. Therefore, it is imperative that the application of protective wraps and/or coatings and electrical isolation joints be properly applied and inspected.

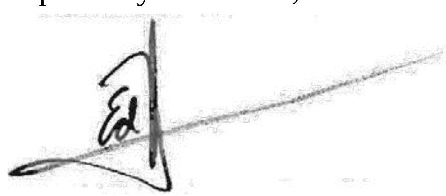
It is the responsibility of the builder and/or contractor to closely monitor the installation of such materials requiring protection in order to assure that the protective wraps or coatings are not damaged.

The recommendations outlined herein are in conformance with current accepted standards of practice that meet or exceed the provisions of the Uniform Building Code (UBC), the International Building Code (IBC), California Building Code (CBC), the American Cement Institute (ACI), Nickel Institute, National Association of Corrosion Engineers (NACE International), Post-Tensioning Institute Guide Specifications and State of California Department of Transportation, Standard Specifications, American Water Works Association (AWWA) and the Ductile Iron Pipe Research Association (DIPRA).

Our 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,



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4 SOIL ANALYSIS LAB RESULTS

Client: Geotechnologies, Inc.
Job Name: Grubb Properties
Client Job Number: 22207
Project X Job Number: S211203F
December 7, 2021

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	ASTM D4658	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B1 ML/CL	1-5	33.0	0.0033	6.3	0.0006	20,770	2,211	8.1	215	0.16	24.4	1.8	0.02	19.1	2.3	14.8	54.5	0.8	1.4
B2 ML/CL	1-5	35.7	0.0036	16.6	0.0017	14,740	2,010	8.0	210	0.15	36.8	0.6	0.02	88.4	5.6	19.3	20.4	4.1	2.2

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract
Anions and Cations tested via Ion Chromatograph except Sulfide.

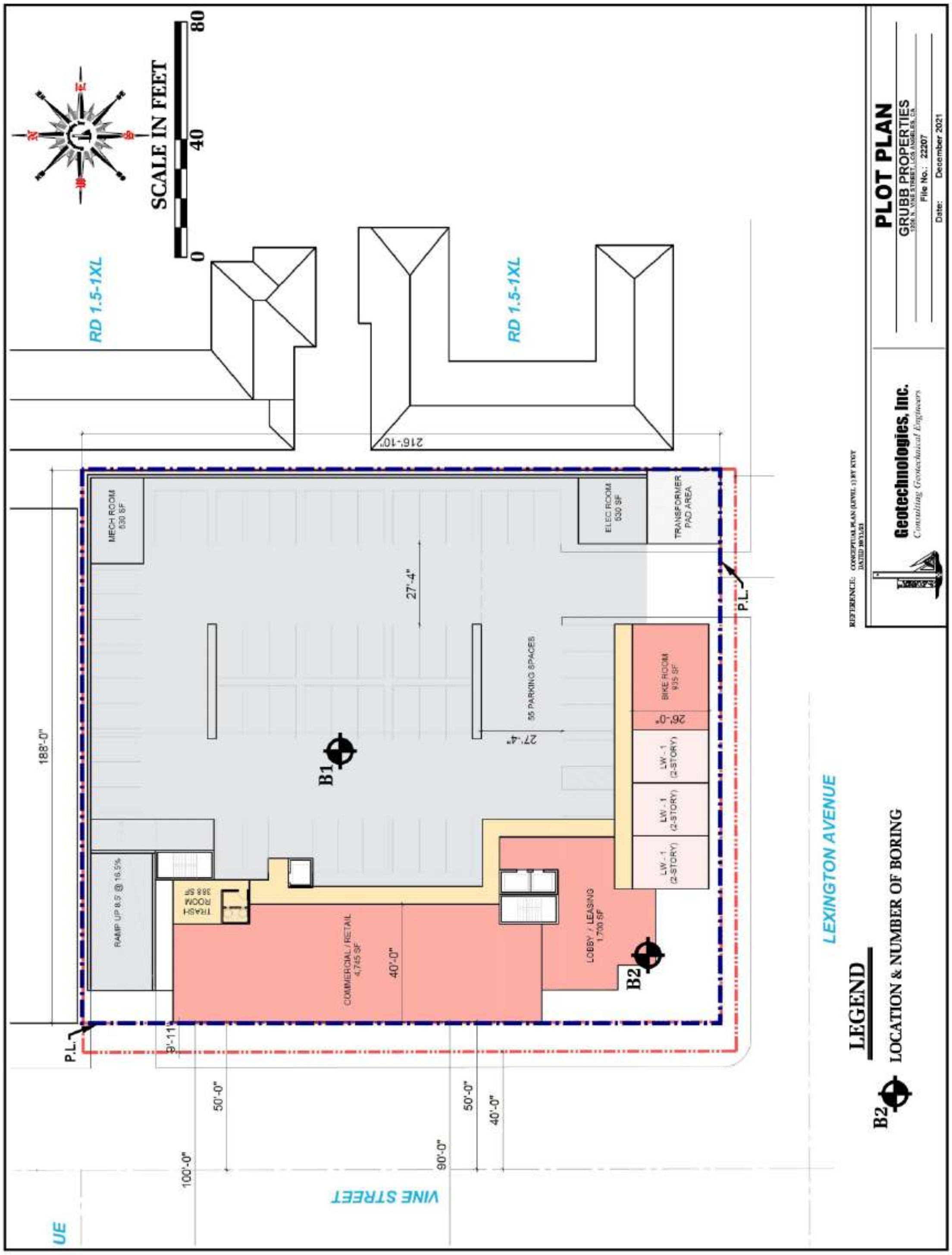

SCALE IN FEET
0
40
80
RD 1.5-1XL
RD 1.5-1XL
216'-10"
188'-0"
MECH ROOM 530 SF
ELEC ROOM 530 SF
TRANSFORMER PAD AREA
27'-4"
27'-4"
55 PARKING SPACES
BIKE ROOM
26'-0"
LW - 1 (2-STORY)
LW - 1 (2-STORY)
LW - 1 (2-STORY)
RAMP UP 8.5' @ 16.5%
TRASH ROOM 388 SF
COMMERCIAL / RETAIL 4,745 SF
40'-0"
LOBBY / LEASING 1,700 SF
B1
B2
P.L.
P.L.
9'-11"
50'-0"
50'-0"
40'-0"
100'-0"
90'-0"
VINE STREET
LEXINGTON AVENUE
LEGEND
B2 LOCATION & NUMBER OF BORING
PLOT PLAN
GRUBB PROPERTIES
File No.: 22207
Date: December 2021
Geotechnologies, Inc.
Consulting Geotechnical Engineers

Figure 2- Soil Sample Locations, 1200 North Vine St, Los Angeles, CA

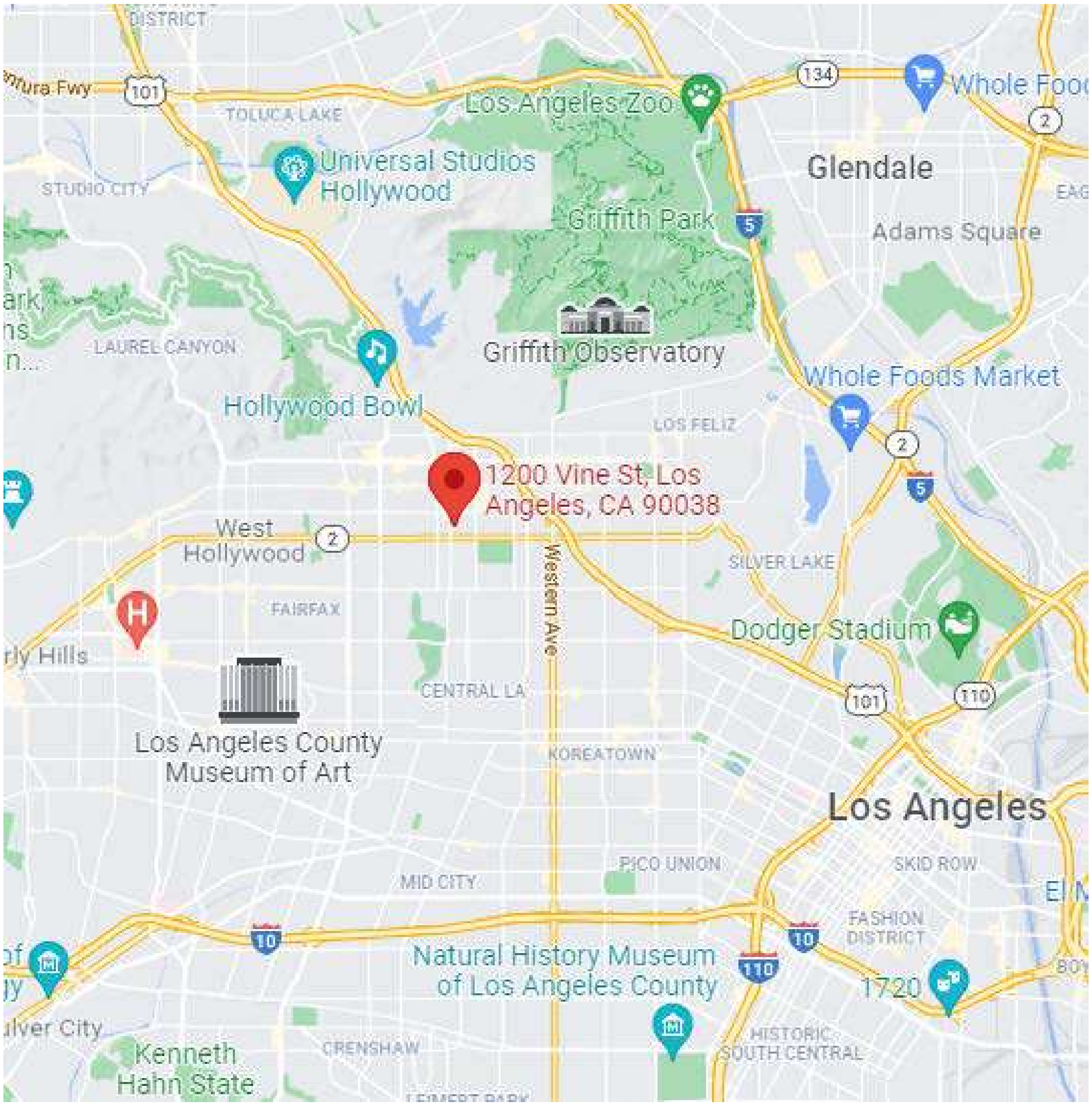


Figure 3- Vicinity Map, 1200 North Vine St, Los Angeles, CA

5 Corrosion Basics

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils should be considered disturbed simply because of their nature from dry to wet seasons.

5.1 Pourbaix Diagram – In regards to a material's environment

All metals are unique and have a weakness. Some metals do not like acidic (low pH) environments. Some metals do not like alkaline (high pH) environments. Some metals don't like either high or low pH environments such as aluminum. These are called amphoteric materials. Some metals become passivated and do not corrode at high pH environments such as steel. These characteristics are documented in Marcel Pourbaix's book "Atlas of electrochemical equilibria in aqueous solutions"

In the mid 1900's, Marcel Pourbaix developed the Pourbaix diagram which describes a metal's reaction to an environment dependent on pH and voltage conditions. It describes when a metal remains passive (non-corroding) and in which conditions metals become soluble (corrode). Steels are passive in pH over 12 such as the condition when it is encased in cement. If the cement were to carbonate and its pH reduce to below 12, the cement would no longer be able to act as a corrosion inhibitor and the steel will begin to corrode when moist.

Some metals such as aluminum are amphoteric, meaning that they react with acids and bases. They can corrode in low pH and in high pH conditions. Aluminum alloys are generally passive within a pH of 4 and 8.5 but will corrode outside of those ranges. This is why aluminum cannot be embedded in cement and why brick mortar should not be laid against an aluminum window frame without a protective barrier between them.

5.2 Galvanic Series – In regards to dissimilar metal connections

All metals have a natural electrical potential. This electrical potential is measured using a high impedance voltmeter connected to the metal being tested and with the common lead connected to a copper copper-sulfate reference electrode (CSE) in water or soil. There are many types of reference electrodes. In laboratory measurements, a Standard Hydrogen Electrode (SHE) is commonly used. When different metal alloys are tested they can be ranked into an order from most noble (less corrosion), to least noble (more active corrosion). When a more noble metal is connected to a less noble metal, the less noble metal will become an anode and sacrifice itself through corrosion providing corrosion protection to the more noble metal. This hierarchy is known as the galvanic series named after Luigi Galvani whose experiments with electricity and muscles led Alessandro Volta to discover the reactions between dissimilar metals leading to the early battery. The greater the voltage difference between two metals, the faster the corrosion rate will be.

Table 1- Dissimilar Metal Corrosion Risk

	Zinc	Galvanized Steel	Aluminum	Cast Iron	Lead	Mild Steel	Tin	Copper	Stainless Steel
Zinc	None	Low	Medium	High	High	High	High	High	High
Galvanized Steel	Low	None	Medium	Medium	Medium	High	High	High	High
Aluminum	Medium	Medium	None	Medium	Medium	Medium	Medium	High	High
Cast Iron	High	Medium	Medium	None	Low	Low	Low	Medium	Medium
Lead	High	Medium	Medium	Low	None	Low	Low	Medium	Medium
Mild Steel	High	High	Medium	Low	Low	None	Low	Medium	Medium
Tin	High	High	Medium	Low	Low	Low	None	Medium	Medium
Copper	High	High	High	Medium	Medium	Medium	Medium	None	Low
Stainless Steel	High	High	High	Medium	Medium	Medium	Medium	Low	None

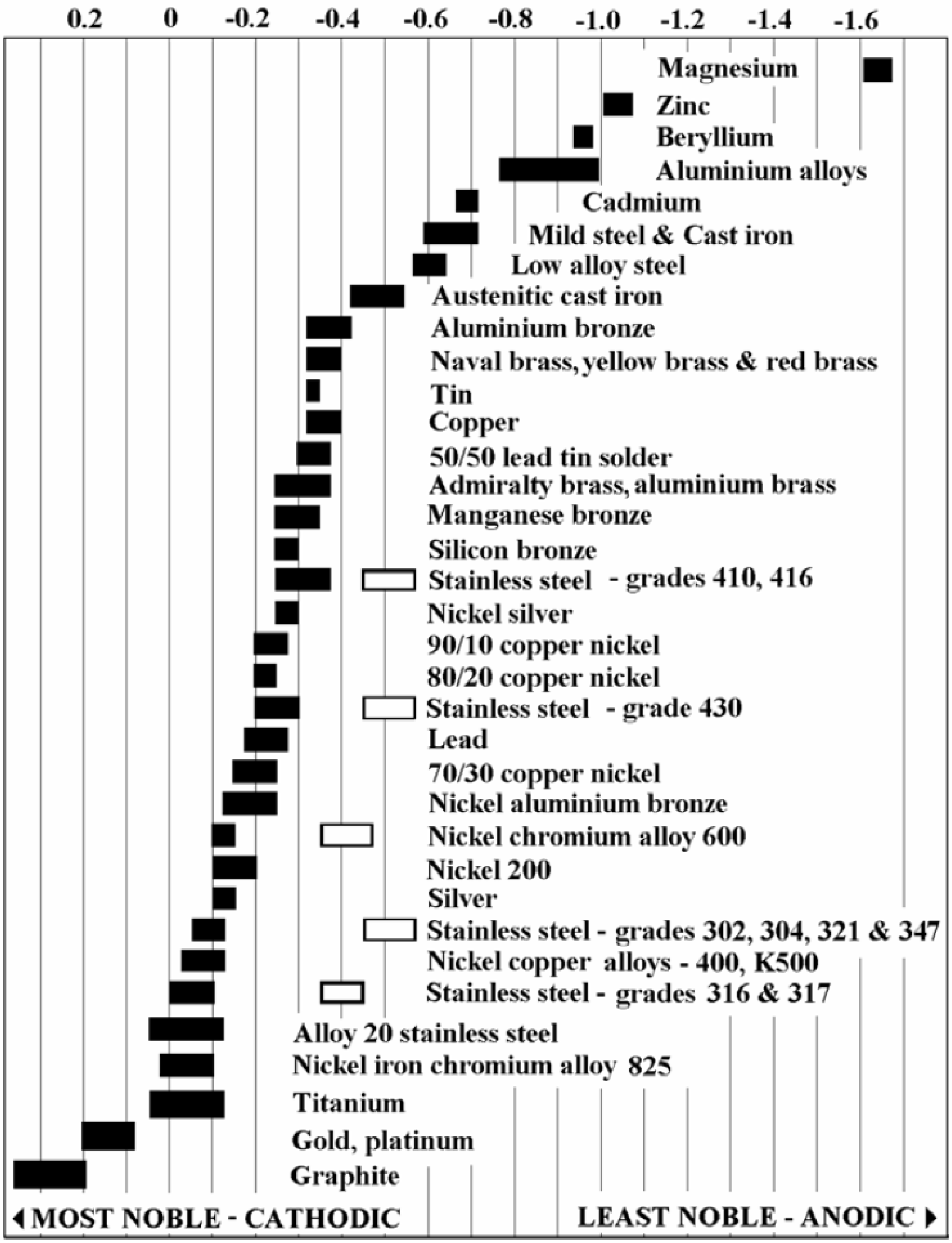
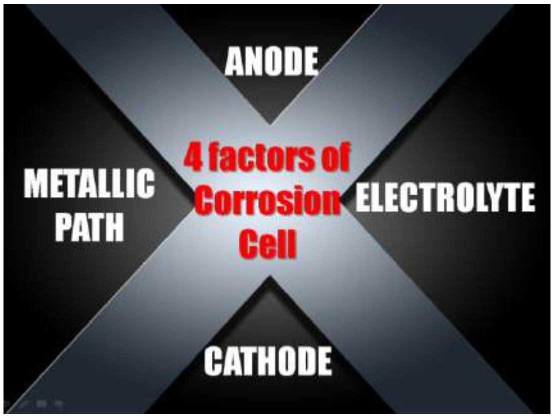
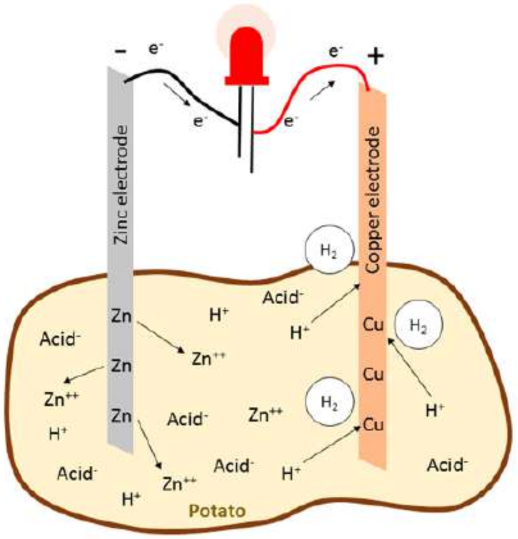


Figure 4 - Galvanic series of metals relative to CSE half cell.

5.3 Corrosion Cell



In order for corrosion to occur, four factors must be present. (1) The anode (2) the cathode (3) the electrolyte and (4) the metallic or conductive path joining the anode and the cathode. If any one of these is removed, corrosion activity will stop. This is how a simple battery produces electricity. An example of a non-metallic yet conductive material is graphite. Graphite is similar in nobility to gold. Do not connect graphite to anything in moist environments.



The anode is where the corrosion occurs, and the cathode is the corrosion free material. Sometimes the anode and cathode are different materials connected by a wire or union. Sometimes the anode and cathode are on the same pipe with one area of the pipe in a low oxygen zone while the other part of the pipe is in a high oxygen zone. A good example of this is a post in the ocean that is repeatedly splashed. Deep underwater, corrosion is minimal, but at the splash zone, the corrosion rate is greatest.

Low oxygen zones and crevices can also harbor corrosive bacteria which in moist environments will lead to corrosion. This is why pipes are laid on backfill instead of directly on native cut soil in a trench. Filling a trench slightly with backfill before installing pipe then finishing the backfill creates a uniform environment around the entire surface of the pipe.

The electrolyte is generally water, seawater, or moist soil which allows for the transfer of ions and electrical current. Pure water itself is not very conductive. It is when salts and minerals dissolve into pure water that it becomes a good conductor of electricity and chemical reactions. Metal ores are turned into metal alloys which we use in construction. They naturally want to return to their natural metal ore state but it requires energy to return to it. The corrosion cell, creates the energy needed to return a metal to its natural ore state.

The metallic or conductive path can be a wire or coupling. Examples are steel threaded into a copper joint, or an electrician grounding equipment to steel pipes inadvertently connecting electrical grid copper grounding systems to steel or iron underground pipes.

The ratio of surface area between the anode and the cathode is very important. If the anode is very large, and the cathode is very small, then the corrosion rate will be very small and the anode may live a long life. An example of this is when short copper laterals were connected to a large and long steel pipeline. The steel had plenty of surface area to spread the copper's attack, thus corrosion was not noticeable. But if the copper was the large pipe and the steel the short laterals, the steel would corrode at an amazing rate.

5.4 Design Considerations to Avoid Corrosion

The following recommendations are based upon typical observations and conclusions made by forensic engineers in construction defect lawsuits and NACE International (Corrosion Society) recommendations.

5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3)

As previously mentioned, different factors can cause corrosion. The most useful and common test for categorizing a soil's corrosivity has been the measure of soil resistivity which is typically measured in units of (ohm-cm) by corrosion engineers and geologists. Soil resistivity is the ability of soil to conduct or resist electrical currents and ion transfer. The lower the soil resistivity, the more conductive and corrosive it is. The following are "generally" accepted categories but keep in mind, the question is not "Is my soil corrosive?", the question should be, "What is my soil corrosive to?" and to answer that question, soil resistivity and chemistry must be tested. Though **soil resistivity is a good corrosivity indicator for steel materials, high chlorides or other corrosive elements do not always lower soil resistivity, thus if you don't test for chlorides and other water soluble salts, you can get an unpleasant surprise.** The largest contributing factor to a soil's electrical resistivity is its clay, mineral, metal, or sand make-up.

Table 2 - Corrosion Basics- An Introduction, NACE, 1984, pg 191

(Ohm-cm)	Corrosivity Description
0-500	Very Corrosive
500-1,000	Corrosive
1,000-2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively less corrosive

Testing a soil's pH provides information to reference the Pourbaix diagram of specific metals. Some elements such as ammonia and nitrates can create localized alkaline conditions which will greatly affect amphoteric materials such as aluminum and copper alloys.

Excess sulfates can break-down the structural integrity of cement and high concentrations of chlorides can overcome cement's corrosion inhibiting effect on encased ferrous metals and break down protective passivated surface layers on stainless steels and aluminum.

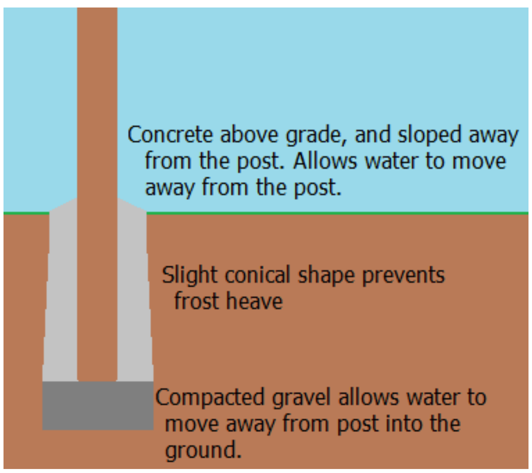
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction (REDOX) electro-potential and by testing for the presence of sulfides.

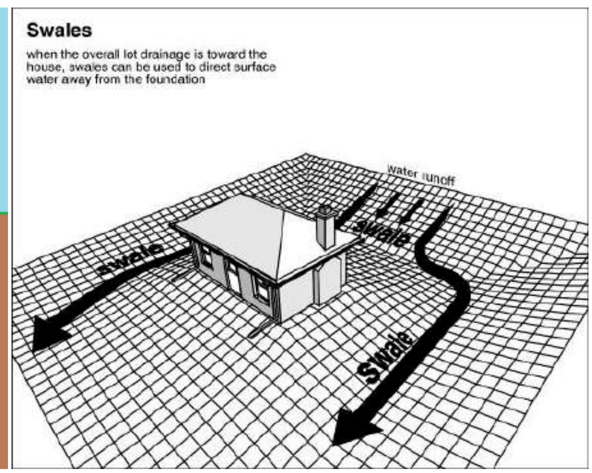
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and concrete.

5.4.2 ***Proper Drainage***

It cannot be emphasized enough that pooled stagnant water on metals will eventually lead to corrosion. This stands for internal corrosion and external corrosion situations. In soils, providing good drainage will lower soil moisture content reducing corrosion rates. Attention to properly sealing polyethylene wraps around valves and piping will avoid water intrusion which would allow water to pool against metals. Above ground structures should not have cupped or flat surfaces that will pond water after rain or irrigation events.

Buildings typically are built on pads and have swales when constructed to drain water away from buildings directing it towards an acceptable exit point such as a driveway where it continues draining to a local storm drain. Many homeowners, landscapers and flatwork contractors appear to not be aware of this and destroy swales during remodeling. The majority of garage floor and finished grade elevations are governed by drainage during design. [14,15]



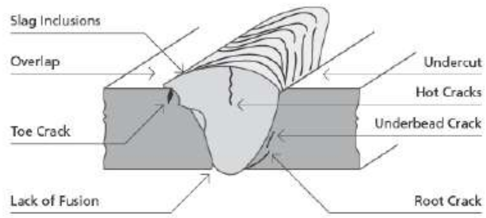


5.4.3 ***Avoiding Crevices***

Crevices are excellent locations for oxygen differential induced corrosion cells to begin. Crevices can also harbor corrosive bacteria even in the most chemically treated waters. Crevices will also gather salts. If water's total alkalinity is low, its ability to maintain a stable pH can also become more difficult within a crevice allowing the pH to drop to acidic levels continuing a pitting process. Welds in extremely corrosive environments should be complete and well filleted without sharp edges to avoid crevices. Sharp edges should be avoided to allow uniform coating of protective epoxy. Detection of crevices in welds should be treated immediately. If pressures and loads are low, sanding and rewelding or epoxy patching can be suitable repairs. Damaged coatings can usually be repaired with Direct to Metal paints. **Scratches and crevice corrosion are like infections, they should not be left to fester or the infection will spread making things worse.**

[14] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

[15] http://southdownstudio.co.uk/problme-drainage-maison.html



BAD **GOOD**

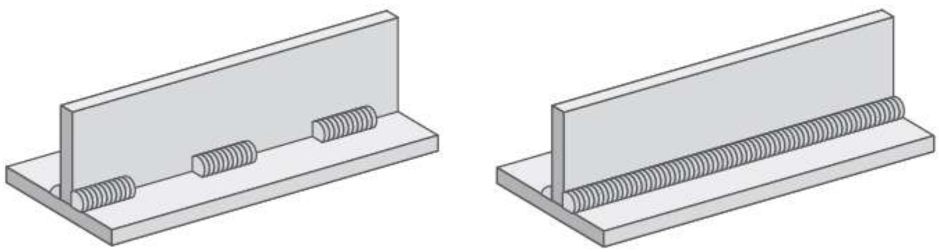


Figure 5 Defects which form weld crevices[16]

5.4.4 Coatings and Cathodic Protection

When faced with a corrosive environment, the best defense against corrosion is removing the electrolyte from the corrosion cell by applying coatings to separate the metal from the soil. During construction and installation, there is always some scratch or damage made to a coating. NACE training recommends that coatings be used as a first line of defense and that sacrificial or impressed current cathodic protection is used as a 2nd line of defense to protect the scratched areas. Use of a good coating dramatically reduces the amount of anodes a CP system would need. If CP is not installed as a 2nd line of defense in an extremely corrosive environment, the small scratched zones will suffer accelerated corrosion. CP details such as anode installation instructions must be designed by corrosion engineers or vessel manufacturers on a per project basis because it depends on electrolyte resistivity, surface area of infrastructure to be protected, and system geometry.

There are two types of cathodic protection systems, a Galvanic Anode Cathodic Protection (GACP) system and an Impressed Current Cathodic Protection (ICCP) system. A Galvanic Anode Cathodic Protection (GACP) system is simpler to install and maintain than an Impressed Current Cathodic Protection (ICCP) system. To protect the metals, they must all be electrically continuous to each other. In a GACP system, sacrificial zinc or magnesium anodes are then buried at locations per the CP design and connected by wire to a structure at various points in system. At the connection points, a wire connecting to the structure and the wire from the anode are joined in a Cathodic Protection Test Station hand hole which looks similar in size and shape to an irrigation valve pull box. By coating the underground structures, one can reduce the number of anodes needed to provide cathodic protection by 80% in many instances.

An ICCP system requires a power source, a rectifier, significantly more trenching, and more expensive type anodes. These systems are typically specified when bare metal is requiring protection

[16] http://www.daroproducts.co.uk/makes-good-weld/

in severely corrosive environments in which galvanic anodes do not provide enough power to polarize infrastructure to -850 mV structure-to-soil potential or be able to create a 100 mV potential shift as required by NACE SP169 to control corrosion. In severely corrosive environments, a GACP system simply may not last a required lifetime due to the high rate of consumption of the sacrificial anodes. ICCP system rectifiers must be inspected and adjusted quarterly or at a minimum bi-annually per NACE recommendations. Different anode installations may be possible but for large sites, anodes are placed evenly throughout the site and all anode wires must be trenched to the rectifier. For a large site, it may be beneficial to use two or more rectifiers to reduce wire lengths or trenching.

To simplify, a GACP system can be installed and practically forgotten with minor trenching because the anodes can be installed very close to the structures. An ICCP system must be inspected annually and anode wires run back to the rectifier which itself connects to the pile system. If any type of trenching or development is expected to occur at the site during the life of the site, it is a good idea to inspect the anode connections once a year to make sure wires are not cut and that the infrastructure is still being provided adequate protection. A common situation that occurs with ICCP systems is that a contractor accidently cuts the wires during construction then reconnects them incorrectly, turning the once cathode, into a sacrificing anode.

Design of a cathodic protection system protecting against soil side corrosion requires that Wenner Four Pin ground resistance measurements per ASTM G57 be performed by corrosion engineers at various locations of the site to determine the best depths and locations for anode installations. Ideally, a sample pile is installed and experiments determining current requirement are conducted. Using this data, the decision is made whether a GACP system is feasible or if an ICCP must be used.

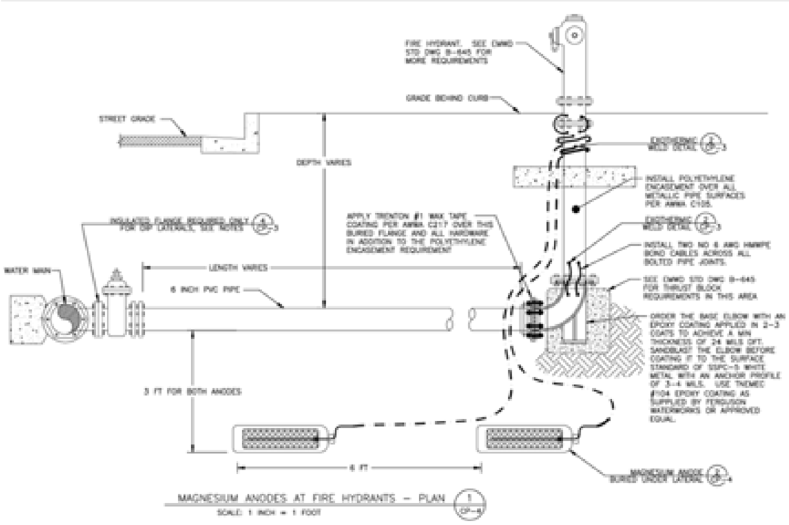


Figure 6 Sample anode design for fire hydrant underground piping

Vessels such as water tanks will have protective interior coatings and anodes to protect the interior surfaces. Anodes can also be buried on site and connected to system skid supports to protect the metal in contact with soil. A good example of a vessel cathodic protection system exists in all home water heaters which contain sacrificial aluminum or magnesium anodes. In environments that exceed 140F, zinc anodes cannot be used with carbon steel because they become the aggressor (Cathodic) to the steel instead of sacrificial (anodic). Anodes in vessels containing extremely brackish water with chloride levels over 2,000 ppm should inspect or change out their anodes every 6 months.

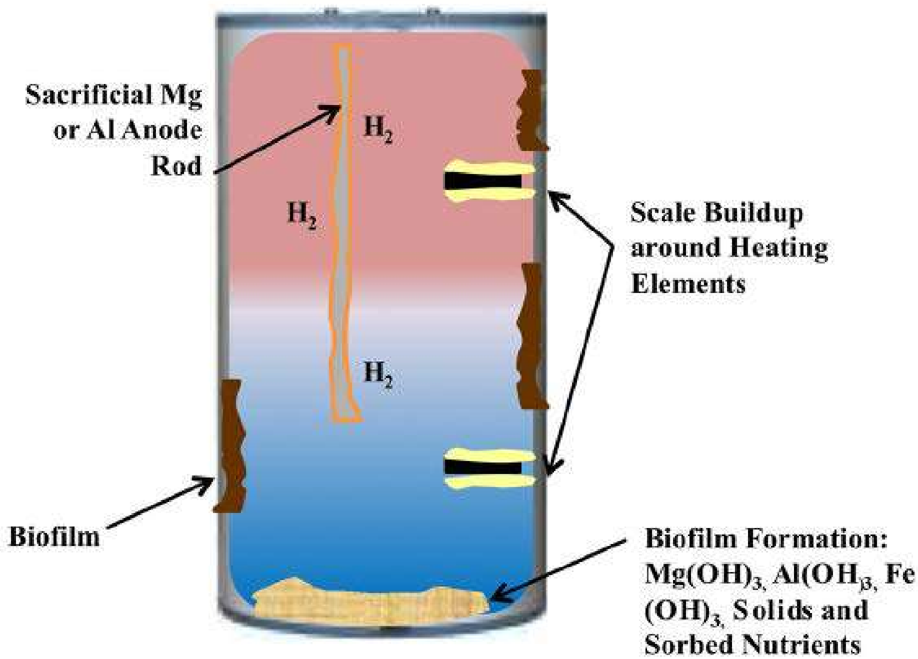


Figure 7 Cross section of boiler with anode

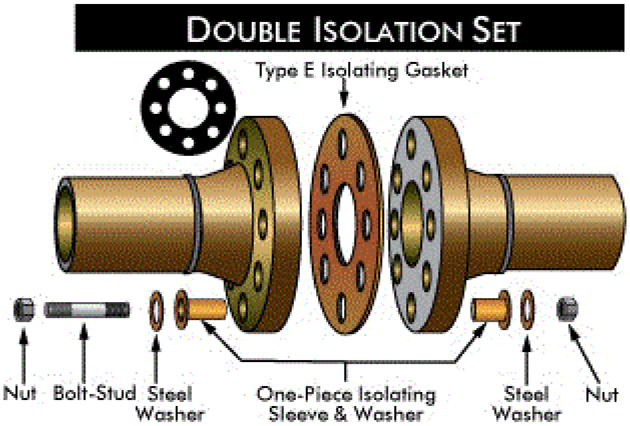
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

5.4.5 Good Electrical Continuity

In order for cathodic protection to protect a long pipeline or system of pipes from external soil side corrosion, they must all be electrically continuous to each other so that the electric current from the anode can travel along the pipes, then return through the earth to the anode. Electrical continuity is achieved by welding or pin brazing #8 AWG copper strand bond cable to the end of pipe sticks which have rubber gaskets at bell and spigots. If steel pipes are joined by full weld, bonding wires are not needed.

Electrical continuity between dissimilar metals is not desirable. Isolation joints or di-electric unions should be installed between dissimilar metals, such as steel pipes connecting to a brass valve per NACE SP0286. Bonding wires should then be welded onto the steel pipes by-passing the brass valve so that the cathodic protection system's current can continue to travel along the steel piping but isolate the brass valve from the steel pipeline. Another option would be to provide a separate cathodic protection system for steel pipes on both sides of the brass valve.

Typically, water heater inlets and outlets, gas meters and water meters have dielectric unions installed in them to separate utility property from homeowner property. This also protects them in the case that a home owner somehow electrically connects water pipes or gas pipes to a neighborhood electrical grounding system which can potentially have less noble steel in soil now connected to much more noble copper in soil which will then create a corrosion cell. This is exactly how a lemon powered clock works when a galvanized zinc nail and a steel nail are inserted into a lemon then connected to a clock. The clock is powered by the corrosion cell created.



5.4.6 Bad Electrical Continuity

Bad electrical continuity is when two different materials or systems are made electrically continuous (aka shorted) when they were not designed to be electrically continuous. Examples of this would be when gas lines are shorted to water lines or to electrical grounding beds. Very often, fire risers are shorted to electrical grounding systems, and water pipes at business parks. Since fire risers usually have a very short ductile iron pipe in the ground which connects to PVC pipe systems, they tend to experience leaks after 7 to 10 years of being attacked by underground copper systems.

It is absolutely imperative that any copper water piping or other metal conduits penetrating cement slab or footings, not come in contact with the reinforcing steel or post-tensioning tendons to avoid creation of galvanic corrosion cells.

5.4.7 Corrosion Test Stations

Corrosion test stations should be installed every 1,000 feet along pipelines in order to measure corrosion activity in the future. For a simple pipeline, two #8 AWG copper strand bond cable welded or pin brazed onto the pipeline are run up to finished grade and left in a hand hole. Corrosion test stations are used to measure pipe-to-soil electro potential relative to a copper copper-sulfate reference electrode to determine if the pipe is experiencing significant corrosion activity. By measuring test stations along a pipeline, hot spots can be determined, if any. The wires also allow for electrical continuity testing, condition assessment, and a multitude of other types of tests.

At isolation joints and pipe casings, two wires should be welded to either side of the isolation joint for a total of 4 wires to be brought up to the hand hole. This allows for future tests of the isolation joint, casing separation confirmation, and pipe-to-soil potential readings during corrosion surveys.

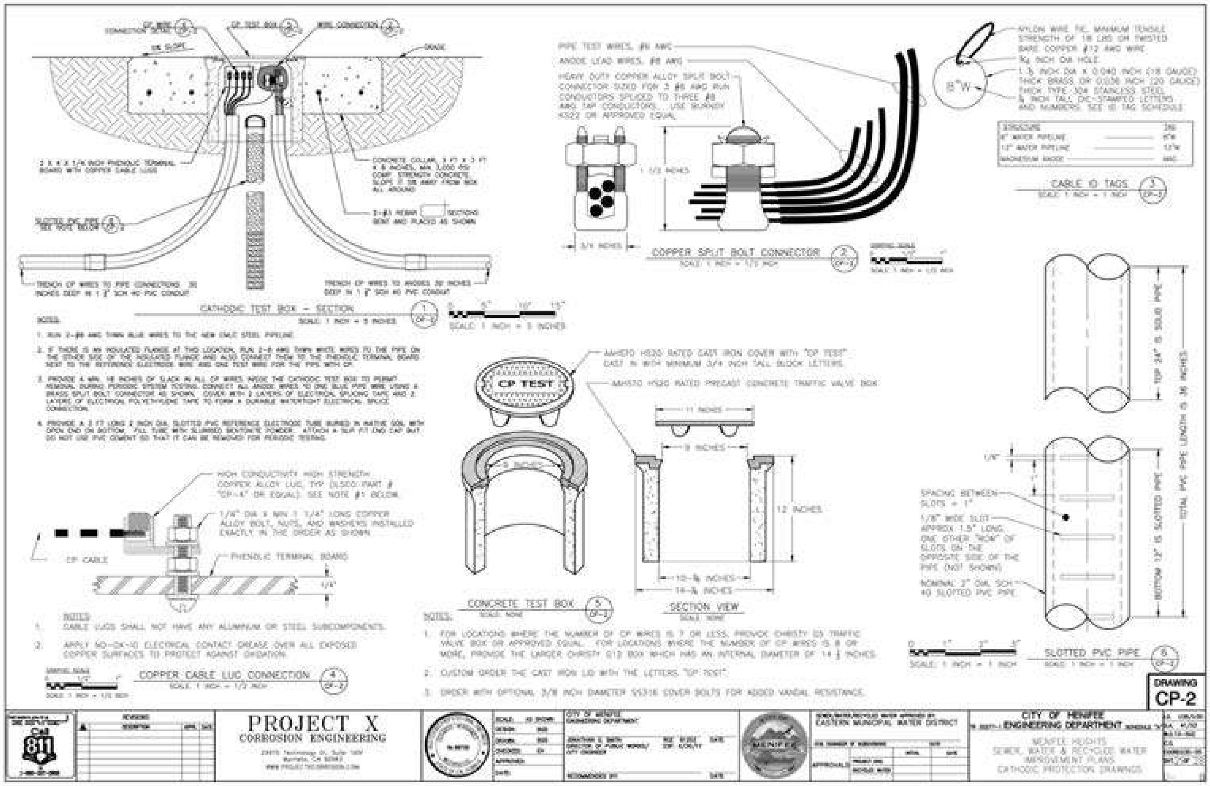


Figure 8 Sample of corrosion test station specification drawing

5.4.8 Excess Flux in Plumbing

Investigations of internal corrosion of domestic water plumbing systems almost always finds excess flux to be the cause of internal pitting of copper pipes. Some people believe that there is no such thing as too much flux. Flux runs have been observed to travel up to 20 feet with pitting occurring along the flux run. Flushing a soldered plumbing system with hot water for 15 minutes can remove significant amounts of excess flux left in the pipes. If a plumbing system is expected to be stagnant for some time, it should be drained to avoid stagnant water conditions that can lead to pitting and dezincification of yellow brasses.

5.4.9 Landscapers and Irrigation Sprinkler Systems

A significant amount of corrosion of fences is due to landscaper tools scratching fence coatings and irrigation sprinklers spraying these damaged fences. Recycled water typically has a higher salt content than potable drinking water, meaning that it is more corrosive than regular tap water. The same risk from damage and water spray exists for above ground pipe valves and backflow preventers. Fiber glass covers, cages, and cement footings have worked well to keep tools at an arm's length.

5.4.10 Roof Drainage splash zones

Unbelievably, even the location where your roof drain splashes down can matter. We have seen drainage from a home's roof valley fall directly down onto a gas meter causing it's piping to corrode at an accelerated rate reaching 50% wall thickness within 4 years. It is the same effect as a splash zone in the ocean or in a pool which has a lot of oxygen and agitation that can remove material as it corrodes.

5.4.11 Stray Current Sources

Stray currents which cause material loss when jumping off of metals may originate from direct-current distribution lines, substations, or street railway systems, etc., and flow into a pipe system or other steel structure. Alternating currents may occasionally cause corrosion. The corrosion resulting from stray currents (external sources) is similar to that from galvanic cells (which generate their own current) but different remedial measures may be indicated. In the electrolyte and at the metal-electrolyte interfaces, chemical and electrical reactions occur and are the same as those in the galvanic cell; specifically, the corroding metal is again considered to be the anode from which current leaves to flow to the cathode. Soil and water characteristics affect the corrosion rate in the same manner as with galvanic-type corrosion.

However, stray current strengths may be much higher than those produced by galvanic cells and, as a consequence, corrosion may be much more rapid. Another difference between galvanic-type currents and stray currents is that the latter are more likely to operate over long distances since the anode and cathode are more likely to be remotely separated from one another. Seeking the path of least resistance, the stray current from a foreign installation may travel along a pipeline causing severe corrosion where it leaves the line. Knowing when stray currents are present becomes highly important when remedial measures are undertaken since a simple sacrificial anode system is likely to be ineffectual in preventing corrosion under such circumstances.[17] Stray currents can be avoided by installing proper electrical shielding, installation of isolation joints, or installation of sacrificial jump off anodes at crossings near protected structures such as metal gas pipelines or electrical feeders.

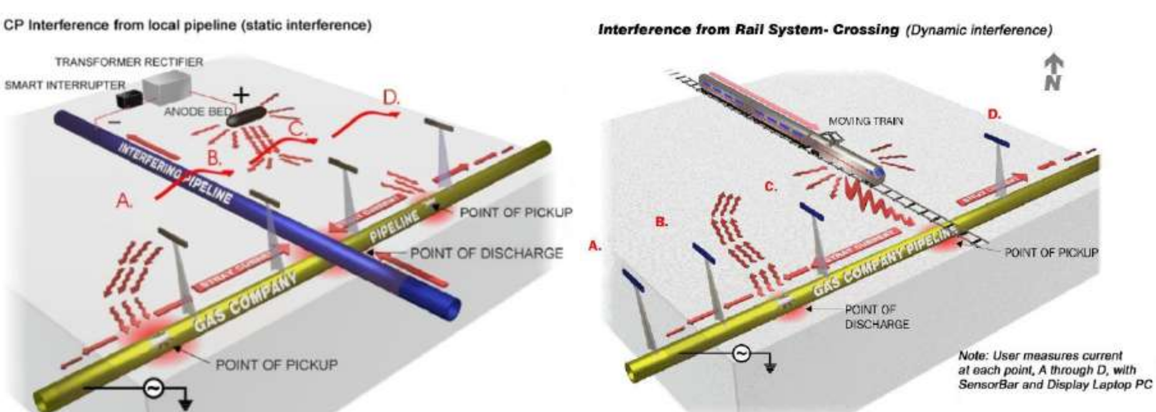


Figure 9 Examples of Stray Current[18]

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[17] http://corrosion-doctors.org/StrayCurrent/Introduction.htm

[18] http://www.eastcomassoc.com/