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August 21, 2019
File No. 21827

Legendary East Pasadena, LLC
116 South Euclid Avenue
Pasadena, California 91101

Attention: Surjit Soni

Subject: Geotechnical Engineering Investigation
Proposed Mixed-Use Development
380 South Rosemead Boulevard, Pasadena, California

Ladies and Gentlemen:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
380 SOUTH ROSEMEAD BOULEVARD
PASADENA, 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 three exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, and the preparation of this report. The exploratory excavation locations are indicated on the enclosed Survey Plan and Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The proposed development consists of a six-story, mixed-use building with two levels of subterranean parking. A small portion of the structure at the southeast end of the site will be at-grade. The proposed structure will likely extend to the property lines. Column loads are estimated to be between 600 and 750 kips. Wall loads are estimated to be between 10 and 15 kips per lineal foot. It is anticipated that grading will consist of excavations to a depth of approximately 30 feet below the existing grade for construction of the proposed subterranean parking levels including foundation elements and elevator pits. The proposed development is illustrated on the Plot Plan and Cross Sections A-A' and B-B' in the Appendix of this report.

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 380 South Rosemead Boulevard, in the City of Pasadena, California. The site is bounded by Walnut Drive and Oakdale Avenue to the north, single-story residential structures to the east, two-story commercial structures to the south, and South Rosemead Boulevard towards the west.

The L-shaped site consists of two lots with a total estimated surface area of 0.7 acres. The subject site is currently occupied by a 3-story church in the northwest corner of the site, two single-story residential structures near the middle of the site, and a parking lot in the southwestern region of the site. The balance of the site is occupied by barren ground and open field areas. The existing structures will be demolished. The site is indicated relative to nearby topographic features in the enclosed Vicinity Map and Survey Plan.

The existing grade across the site descends toward the southeast. The site ranges in elevation from 620 feet at the northwest corner to elevation 614 feet in the southeast corner for a total elevation difference of 6 feet. The overall site gradient is 50 to 1 (horizontal to vertical).

Vegetation consists of lawns and landscaping with larger trees located near the north and south property lines. Drainage across the site is by sheetflow toward southeast. The neighboring developments consist primarily of commercial and residential structures.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on June 6 and June 7, 2019, by drilling three borings to a maximum depth of 68 feet below existing ground surface. The site exploration was performed with a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The boring locations are indicated on the Survey Plan and Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-3.

Soil samples were taken with a California-modified, split-spoon sampler lined with 2.5-inch diameter brass rings. The sampler was advanced with a 140-pound weight dropped from a height of 30 inches using an automatic trip hammer. Samples were taken at regular depth intervals. In Boring 1, at depths between the California Modified samples, Standard Penetration test equipment was used.

The locations of the borings were determined from hardscape features indicated on the attached Survey Plan and Plot Plan. The locations of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill soil was encountered in the borings to a maximum depth of three feet below the existing site grade. The soil consists of silty sand that is dark brown in color, moist, medium dense and fine grained.

The fill is underlain by alluvial soil to the maximum depth of the borings of 68 feet below ground surface. The alluvium consists of silty sand to sand and ranges in color from dark brown to yellowish brown, and is slightly moist to moist, medium dense to very dense, and fine to medium grained. From a depth of 35 feet to 50 feet, the alluvium coarsens to include gravel and cobbles.

The geologic materials observed in the borings were consistent with the geologic conditions indicated on the Local Geologic Map provided in the Appendix of this report. More detailed descriptions of the geologic materials encountered may be obtained from the individual logs of the subsurface excavations.

Groundwater

Groundwater was not encountered during site exploration to a maximum excavated depth of 68 feet.

State Well Number 01N12W36H003S (ground surface elevation of 606 feet) located 0.9 miles southwest of the site indicate existing groundwater levels on the order of 225 feet below ground surface (attached herein).

The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report 030 Plate 1.2 entitled “Historically Highest Ground Water Contours”. Review of this plate indicates that the historically highest groundwater level is approximately 28 feet below grade.

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

Caving

Caving could not be directly observed during exploration due to the continuously cased design of the hollowstem augers. However, based on the experience of this firm, large diameter excavations that encounter granular, cohesionless soils, and excavations below the groundwater table, will most likely experience caving.

The alluvial soil contains layers of clean, dry sand. Large diameter borings that drill within these materials may experience caving. If significant caving is encountered, polymeric drilling muds may be needed.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

REGIONAL FAULTING

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

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

LOCAL FAULTING

The Raymond fault is located approximately 1,600 feet southeast of the subject site as indicated by the attached Earthquake Fault Map. Much of the geomorphic evidence for the Raymond fault has been obliterated by urbanization of the San Gabriel Valley. However, a discontinuous escarpment can be traced from Monrovia to the Arroyo Seco in South Pasadena. The very bold, "knife edge" escarpment in Monrovia parallel to Scenic Drive is believed to be a fault scarp of the Raymond fault. Trenching of the Raymond fault is reported to have revealed Holocene movement (Weaver and Dolan, 1997). The Raymond fault has been found to be an effective groundwater barrier which divides the San Gabriel Valley into groundwater sub-basins.

The recurrence interval for the Raymond fault is probably slightly less than 3,000 years, with the most recent documented event occurring approximately 1,600 years ago (Crook, et al, 1978). However, historical accounts of an earthquake that occurred in July 1855 as reported by Toppozada and others, 1981, place the epicenter of a Richter Magnitude 6 earthquake within the Raymond fault. It is believed that the Raymond fault is capable of producing a 6.8 magnitude earthquake. The Raymond Fault is considered active by the California Geological Survey.

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

Surface rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The nearest Alquist-Priolo Fault Zone is located approximately 1,100 feet to the southeast of the site as indicated on the attached Earthquake Fault Map. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

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

The Seismic Hazards Maps of the State of California (CDMG, 1999), classifies the site as part of the potentially "Liquefiable" area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

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

Groundwater was not encountered during exploration to a maximum excavated depth of 68 feet below ground surface. According to the Seismic Hazard Zone Report of the Mount Wilson 7½-Minute Quadrangle (CDMG, 1999), the historic high groundwater level for the subject site is estimated at 28 feet below ground surface. A groundwater level of 28 feet below ground surface was conservatively utilized for the enclosed liquefaction analysis.

The peak ground acceleration (PGA_M) and modal magnitude were obtained from USGS associated websites using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) and the Office of Statewide Health Planning and Development (OSHPD), 2019, ground motion utility tool. A Site Class "D" (Stiff Soil Profile) was utilized in the USGS seismic and OSHPD ground motion utility tools. A modal magnitude (M_W) of 6.5 was obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration of 1.06g, corresponding to a seismic event with a mean return interval of 2,475 years (2% exceedance in 50 years) was obtained using the OSHPD seismic hazard utility tool. These parameters were used in the enclosed liquefaction analysis.

The enclosed "Liquefaction Evaluation" calculation sheet is based on the results from 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 for representative soil samples are presented on the enclosed E-Plate. Based on CGS Special Publication 117A (CDMG, 2008), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Furthermore, cohesive soils with PI between 7 and 12 and moisture content greater than 85 percent of the liquid limit are susceptible to liquefaction.

Based on the adjusted blow count data, results of laboratory testing, and the calculated factor of safety against the occurrence of liquefaction, it is the opinion of this firm that the potential for liquefaction at the site is considered to be remote.

Dynamic Dry Settlement

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

Seismic dry sand settlements were calculated utilizing Tokimatsu and Seed's procedure for the soils encountered in Boring B1 (Tokimatsu and Seed, 1987). A ground acceleration of 1.06g and a mean magnitude of 6.5, as determined from the USGS Probabilistic Seismic Hazard Deaggregation program and the Office of Statewide Health Planning and Development (OSHPD), 2019, ground motion utility tool, were utilized in the dynamic dry settlement calculation.

Based on these parameters, the total seismically-induced dry sand settlement was calculated to be 0.14 inches for Boring B1. Differential dynamic dry settlement would not be expected to exceed 0.1 inches. The calculated settlements are expected to be within the tolerance of structures designed based on current building codes.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. The site is high enough and far enough from the ocean to preclude being prone to hazards of a tsunami.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within the potential mapped inundation boundaries due to a seiche or a breached upgradient reservoir. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the relatively moderate gradient across 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 development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

The site is underlain by uncertified fill soil and medium dense to very dense, fine grained and granular alluvium. The fill was observed to extend to depths of up to three feet below the ground surface during exploration. The underlying alluvium consists of silty sand to sand. At a depth range of 35 to 50 feet the alluvium coarsens to gravelly sand with cobbles.

Groundwater was not observed during site exploration to a maximum excavated depth of 68 feet below existing ground surface. The historically highest groundwater level is approximately 28 feet below grade. Temporary construction dewatering will not be necessary. However, retaining walls and floor slabs extending deeper than 28 feet below existing ground surface would require structural design to resist hydrostatic pressure.

The fill soil is not suitable for support of the proposed foundations, floor slabs or additional fill. Excavation of the proposed subterranean levels will remove the unsuitable fill materials in the building area. The proposed structure may be supported by conventional foundations bearing in alluvial soil exposed at the base of the proposed excavation. For sections of the proposed building to be constructed at existing site grade, existing soils should be removed and recompacted as certified fill for building slab support. Foundations should be extended through the certified fill to bear in undisturbed alluvial soil.

The proposed development will have a portion with subterranean levels and an at-grade portion that will have footings in close proximity to the subterranean levels. The retaining walls must be designed for the surcharge pressure of the onsite, at-grade footings and those footings from the adjacent structures.

Excavation of the proposed subterranean levels will require shoring measures to provide a stable working area due to the proposed excavation depth, and the proximity of adjacent surface streets and property lines. It is anticipated that the shoring piles will encounter gravelly and cobbley soils below the proposed finish floor elevation.

The location for stormwater disposal devices have not been specifically addressed on this site. It is the opinion of this office that stormwater infiltration is feasible within the site. Should stormwater infiltration be implemented within the site, this office should conduct site specific percolation test corresponding to the anticipated infiltration basin depth and location.

The following statement is made in regard to Los Angeles County Code Sections 110, 111 and J105: Fill slope surfaces have been compacted and buttress fills or similar stabilization measures have been installed in accordance with our recommendations as approved by the Building Official. It is the opinion of the undersigned that, provided our recommendations are followed, the proposed development will be safe for its intended use against hazard from landsliding, settlement or slippage. The proposed development will have no adverse effect on the stability of the site or adjoining properties.

SEISMIC DESIGN CONSIDERATION

California Building Code Seismic Parameters

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

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS		
California Building Code	2016	2019
ASCE Design Standard	7-10	7-16
Risk Category	II	II
Site Class	D	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.743g	2.058g
Site Coefficient (F_a)	1.0	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.743g	2.058g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.829g	1.372g
Mapped Spectral Acceleration at One-Second Period (S_1)	1.003g	0.753g
Site Coefficient (F_v)	1.5	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.505g	1.280g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	1.003g	0.853g*

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

FILL SOIL

As much as three feet of fill soil was encountered during exploration. The existing fill soils are not suitable for the support of foundations, floor slabs or additional fill but may be reused as compacted fill. It is anticipated that the existing fill will be removed during excavation of the subterranean parking levels anticipated for the proposed development.

For sections of the proposed building to be constructed at existing site grade, the fill soil should be removed and recompacted as certified fill for building slab support. Foundations may then be extended through the certified fill to bear in competent undisturbed alluvial soil. Conventional foundations bearing in competent alluvial soils are recommended for support of the proposed mixed-use structure.

EXPANSIVE SOILS

The onsite geologic materials are in the very low expansion range. The Expansion Index was found to be a value of 7 for a bulk sample taken from a depth of 1 to 5 feet. Recommended reinforcement is provided in the “Foundation Design” and “Slab-on-Grade” sections of this report.

WATER-SOLUBLE SULFATES

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

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

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

GRADING GUIDELINES

The following guidelines are provided for any miscellaneous compaction that may be required, such as retaining wall backfill or subgrade preparation.

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 structure should be removed during grading.
- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.
- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D1557.

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

Acceptable Materials

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

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

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

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D1557.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 3 and 7 percent should be anticipated when excavating and recompacting the existing fill and underlying alluvial soils on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

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

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

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

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

Abandoned Seepage Pits

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

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

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

Geotechnical Observations and Testing During Grading

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

FOUNDATION DESIGN

Conventional Foundations

The proposed structure may be supported by conventional foundations bearing in competent undisturbed alluvial soil. It is anticipated that the excavation for the proposed subterranean parking levels will remove the existing fill materials and expose competent undisturbed alluvium at the subgrade. For sections of the proposed building to be constructed at existing site grade, existing fill may be removed and recompacted as certified fill for building slab support. Foundations may then be extended through the certified fill to bear in competent undisturbed alluvial soil.

Continuous foundations may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 3,500 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 300 pounds per square foot. The bearing capacity increase for each additional foot of depth is 800 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

A minimum factor of safety of 3 was utilized in determining the allowable bearing capacities. The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed structure may be supported on certified recompacted fill or may be deepened through any existing fill to bear in undisturbed alluvial soils. Continuous footings may be designed for a bearing capacity of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

Controlled Low Strength Material

Foundations may require deepening to bear in competent native alluvial soils. The deepened portion of the foundation excavations may be filled with controlled low-strength material (CLSM). This is allowable under 2016 California Building Code section 1804.7.

The foundation excavations should be cleaned of all loose materials prior to placement of the CLSM. The CLSM should consist of 3-sack slurry mix. A sample of the CLSM should be collected and checked for compressive strength. The results of the tests should indicate that the CLSM at 28 days yields a minimum of 100 pounds per square inch.

The foundation may be formed and poured on top of the cured CLSM. Some method of ensuring a good bond between the top of the CLSM and the concrete of the proposed foundation should be employed.

Foundations Adjacent to Buildings or Property Lines

Where new foundations are proposed adjacent to a deep adjacent foundation, the new foundations should be deepened to match the depth of the adjacent foundation. Where foundation excavations will leave an adjacent foundation unsupported, the foundation excavation should be shored.

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.

Foundation 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 0.40 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed alluvium or certified, recompacted soil may be computed as an equivalent fluid having a density of 275 pounds per cubic foot with a maximum earth pressure of 2,750 pounds per square foot.

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

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is not expected to exceed one inch below the heaviest loaded columns bearing on alluvium. Differential settlement is not expected to exceed 1/2 inch.

Foundation Observations

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

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

RETAINING WALL DESIGN

Historically highest groundwater is estimated at a depth of 28 feet below existing ground surface. Retaining walls extending deeper than 28 feet below existing ground surface would require design to resist hydrostatic pressures. In addition, lateral pressure due to the surcharge from the adjacent structures and that from the at-grade portion of the proposed structure shall be included.

Cantilever Retaining Walls

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

HEIGHT OF RETAINING WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Less than 15	32
15 to 20	36
20 to 25	39
25 to 30	40

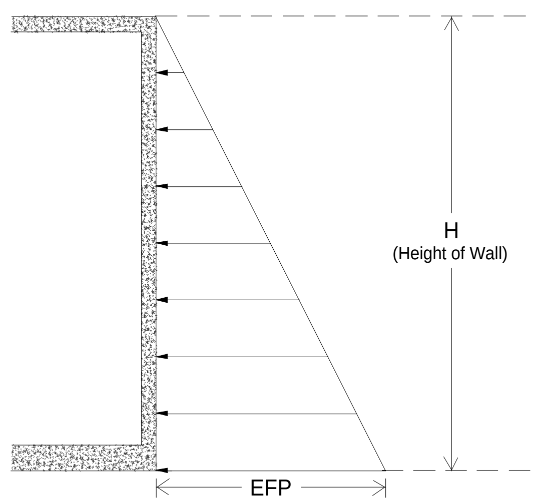
For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

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

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

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. The at-rest pressure for design purposes is 52 pounds per cubic foot. Additional earth pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.



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

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

Retaining Wall Drainage

Subdrains may consist of 4-inch diameter perforated pipes, placed with perforated surface facing down. The pipe shall be encased in at least one-foot of gravel around the pipe. The gravel shall be wrapped in non-decomposable filter fabric. The gravel may consist of three-quarter inch to one inch crushed rock. As an alternative for outdoor retaining walls, 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 one cubic foot in dimension, and may consist of three-quarter inch to one inch crushed rock, wrapped in non-decomposable filter fabric.

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

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

Where shoring will not allow the installation of a standard subdrainage system outside the wall rock pockets may be utilized. The rock pockets with should drain through the wall. The pockets should be a minimum of 12 inches in length, width and depth. The pocket should be filled with ¾-inch to one-inch gravel wrapped in non-decomposable filter fabric. The rock pockets should be spaced no more than 8 feet on center. It should be noted, earth retaining walls associated with subterranean ramps or stairwells should be connected to the water drainage system.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration to a depth of 68 feet. Historically highest groundwater is estimated at 28 feet below ground surface. For retaining walls extending no deeper than 28 feet below the ground surface, the water anticipated from the wall drainage system will be from rainfall, irrigation and leaky pipes. A pump capacity of 5 gallons per minute is considered sufficient.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 29.8 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Waterproofing

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

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the most recent revision of ASTM D1557 method of compaction. Flooding is not permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Excavations on the order of 30 feet in vertical height may be required for the anticipated subterranean parking levels, elevator pits, and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum vertical height of 20 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.

Excavations Adjacent to Existing Foundations, Buildings or Property Lines

Where foundation excavations will leave an adjacent foundation unsupported the foundation excavation should be slot cut or shored. The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. Alternate "A" slots of 8 feet may be worked. The remaining earth buttresses ("B" and "C" slots) should each be 8 feet in width for a combined intervening length of 16 feet. The foundation should be poured in the "A" slots before the "B" slots are excavated. After completing the foundation in the "B" slots, finally the "C" slots may be excavated. Slot cut excavations shall not exceed 5 feet in vertical height.

Excavation Observations

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

SHORING DESIGN

The following information regarding 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. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tied-back anchors or raker braces.

Soldier Piles – Drilled and Poured

Drilled cast-in-place soldier piles should be placed no closer than two 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 geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation may be assumed to be 600 pounds per square foot per foot, up to a maximum of 3,000 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 geologic materials.

Casing may be required should caving be experienced in the granular geologic materials. 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 five 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 0.40 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 7 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 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.

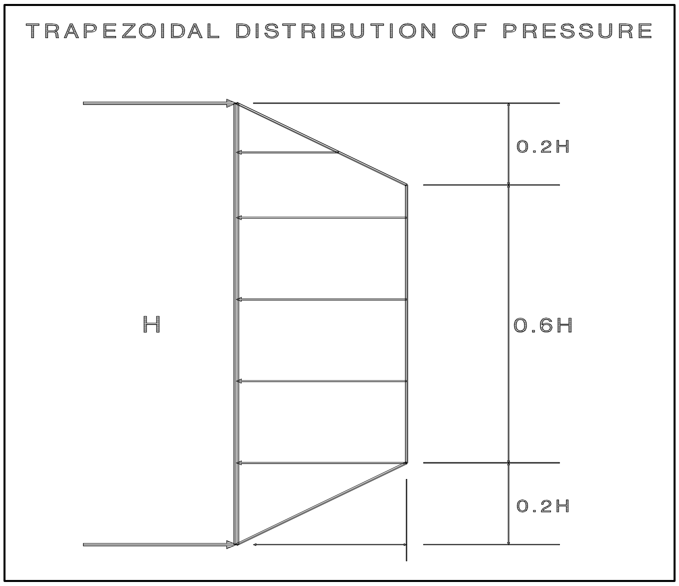
Lateral Pressures

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)
H up to 15	25
15 to 20	27
20 to 25	30
25 to 30	32
30 to 35	33

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

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs, with the trapezoidal distribution as shown in the diagram below.



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

HEIGHT OF SHORING "H" (feet)	**DESIGN SHORING FOR (Where H is the height of the wall)**
Up to 20	18H
20 to 25	19H
25 to 30	20H
30 to 35	21H

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

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge. Anchors should be placed at least 6 feet on center to be considered isolated.

Drilled friction anchors may be designed for a skin friction of 525 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,500 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchor Installation

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

Tieback Anchor Testing

At least 10 percent of the anchors should be selected for “quick” 200 percent tests. It is recommended that at least three anchors be selected for 24-hour, 200 percent tests and that the 24-hour tests be performed prior to installation of additional tiebacks. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on these initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

All of the remaining anchors should be tested to at least 150 percent of design load. The total deflection during the 150 percent test should not exceed 12 inches. The rate of creep under the

150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. Where post-grouted anchors are utilized, additional post-grouting may be required. The installation and testing of the anchors should be observed by a representative of the soils engineer.

Raker Brace Foundations

An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 4 feet in width and length as well as 4 feet in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of ½ inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

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

Concrete floor slabs should be a minimum of 4 inches in thickness. Slabs-on-grade should be cast over competent undisturbed alluvial soils or certified compacted fill. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed 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 of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

Based on ACI 302.2R-30, Chapter 7, for projects which do not have vapor sensitive coverings or humidity controlled areas, a vapor retarder is not necessary. Where a vapor retarder is considered necessary, the design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E1643 and ASTM E1745. The vapor retarder should comply with ASTM E1745 Class A requirements. The necessity of a vapor retarder is not a geotechnical issue and should be confirmed by qualified members of the design team.

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

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

Concrete Crack Control

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

For standard control of concrete cracking, a maximum crack control joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

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

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Please note: Floor slabs extending deeper than 28 feet below existing ground surface would require design to resist hydrostatic pressures.

Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-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 of the maximum density as determined by the most recent revision of ASTM D1557. 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.

Due to a wide variation which may occur during the grading process, it is recommended that R-value tests be performed near the completion of grading in order to ascertain the subgrade conditions prior to paving. The recommended paving sections shall be considered preliminary and are subject revision. For preliminary design purposes, an R-value of 40 was assumed. A preliminary paving section is provided in the following table for traffic indexes of 4, 6, and 8:

PAVING DESIGN SECTIONS				
Service Level	Asphalt Pavement		Concrete Pavement	
	Asphalt Pavement Thickness (Inches)	Asphalt Pavement Base Course (Inches)	Concrete Pavement Thickness (Inches)	Concrete Pavement Base Course (Inches)
Passenger Cars (TI=4)	3	4	6	4
Moderate Truck (TI=6)	4	6	6	4
Heavy Trucks (TI=8)	5	9	7.5	4

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should consist of Crushed Aggregate Base which conforms with Section 200-2.2 of the most recent edition of "Standard Specifications for Public Works Construction", (Green Book). Crushed Misc. Base is addressed in Section or 200-2.4.

Concrete paving may be used on the project. A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. For standard control of concrete cracking, a maximum crack control joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Concrete paving should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Construction joints should be designed by a structural engineer.

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

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

STORMWATER DISPOSAL

Introduction

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.

The Proposed System

The location for potential stormwater disposal has not been specifically addressed on this site. It is the opinion of this office that stormwater infiltration is feasible. Should stormwater infiltration be implemented, this office should conduct site-specific percolation tests corresponding to the anticipated infiltration basin depths and locations specified by the design team.

Laboratory testing indicates that the onsite soils are in the very low expansion range and that the soil strata underlying the proposed development are not susceptible to significant hydroconsolidation. Additionally, it is the assessment of this firm that soils encountered on the site should allow stormwater to percolate in a generally vertical manner. Therefore, it is unlikely that a perched water condition will develop.

With regards to deep infiltration within the site, it is the opinion of this firm that any infiltration of stormwater in close proximity to structures should occur below the influence zone of the proposed foundations. Foundation influence zones would be expected to extend to depths correlating to roughly twice the width of the largest pad footing. Assuming a typical 10 foot square pad footing that is founded at an approximate depth of 25 feet, this would correlate to an influence depth of 20 feet below the bottom of pad footing, or approximately 45 feet below the ground surface. For deep infiltration systems directly underlying the proposed structure, stormwater infiltration should occur at depths deeper than 45 feet below existing ground surface.

Where percolation 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.

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.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on samples of the onsite soils by Project X, Corrosion Engineering, Inc. indicate that the electrical resistivity of the onsite soils are mildly corrosive to general metals in the saturated condition. The soil pH value of the samples tested were found to be 7.7 to 8.7 and were determined to be at levels not detrimental to copper or aluminum alloys but can allow corrosion of steel and iron in moist environments. Chloride levels tested using site samples are low and may cause insignificant corrosion of metals. Sulfate levels are negligible for corrosion of metals and cement. Concentrations of ammonia and nitrates were high enough to cause accelerated corrosion of copper and copper alloys such as brass. Sulfides presence was determined to be negative.

In summary, copper and brass metals should be utilized with caution at shallow depths within the site. Special cement types need not be utilized for concrete structures in contact with onsite soils, since the sulfate content of the soils is negligible. Detailed results, discussion of results and recommended mitigating measures are provided within the report by Project X, Corrosion Engineers, contained in the Appendix.

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. 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.

The alluvium coarsens between 25 and 40 feet below the ground surface. Gravel and cobbles should be anticipated when excavating within this zone.

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. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

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

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

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

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

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

EXCLUSIONS

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

GEOTECHNICAL TESTING

Classification and Sampling

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

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in accordance with the most recent revision of ASTM D1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by the most recent revision of ASTM D4959 or ASTM D4643. 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 by the most recent revision of ASTM D3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. 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-Plate.

The most recent revision of ASTM D3080 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 using the most recent revision of ASTM D2435. 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-Plate.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of the most recent revision of ASTM D1557. A soil at a selected moisture content is placed in five layers into as 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.

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 D4829. 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.

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 D422 is used to determine particle sizes smaller than the Number 200 sieve. Results from the Number 200 sieve test are presented in Plate E of this report.

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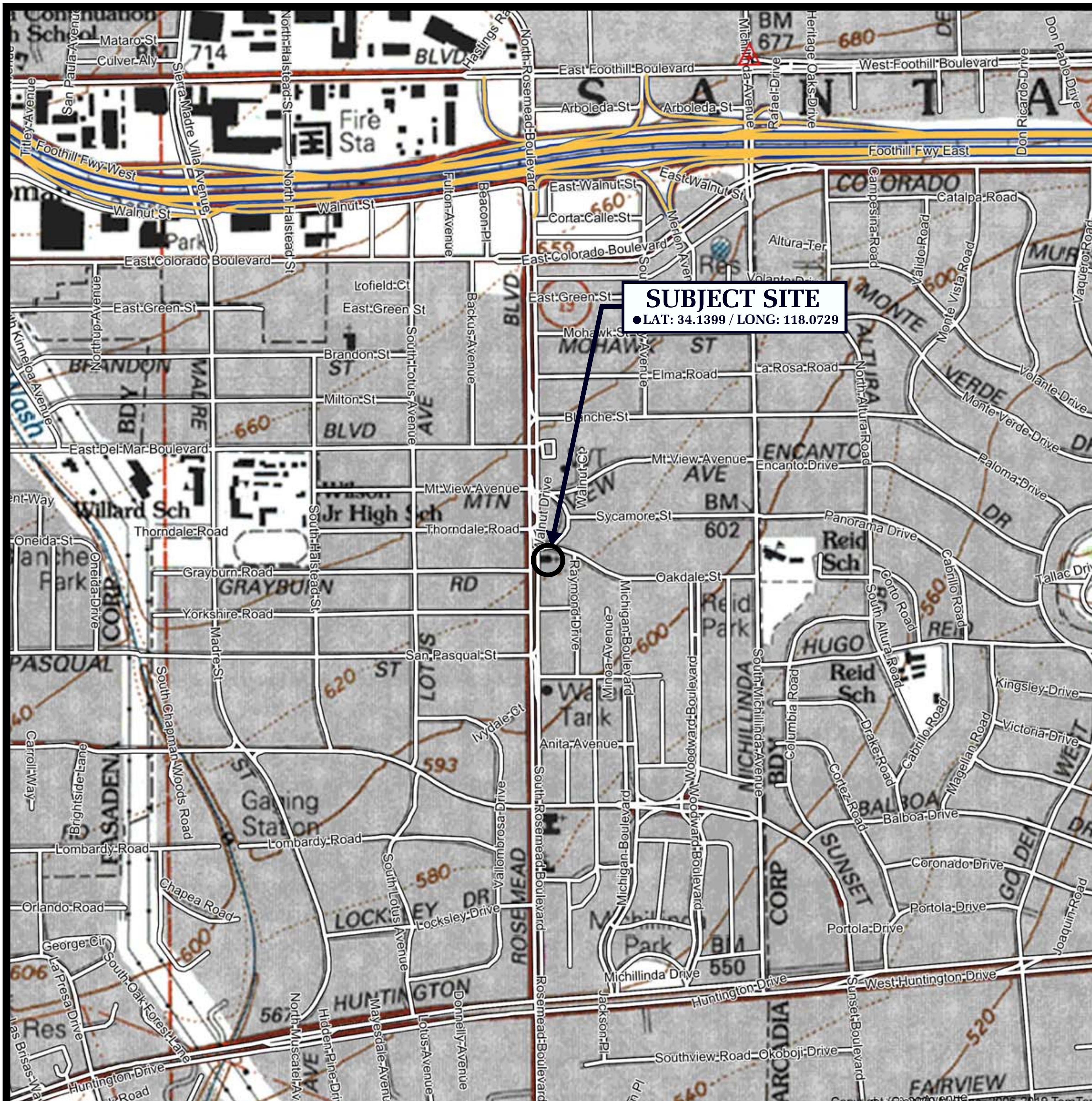
Weaver, K.D., and Dolan, J.F., 2000, Paleoseismology and Geomorphology of the Raymond Fault, Los Angeles County, California: Seismol. Soc. America Bull. 90:1409-1429.



SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6 MILES

0 1000 2000 3000 FEET

N

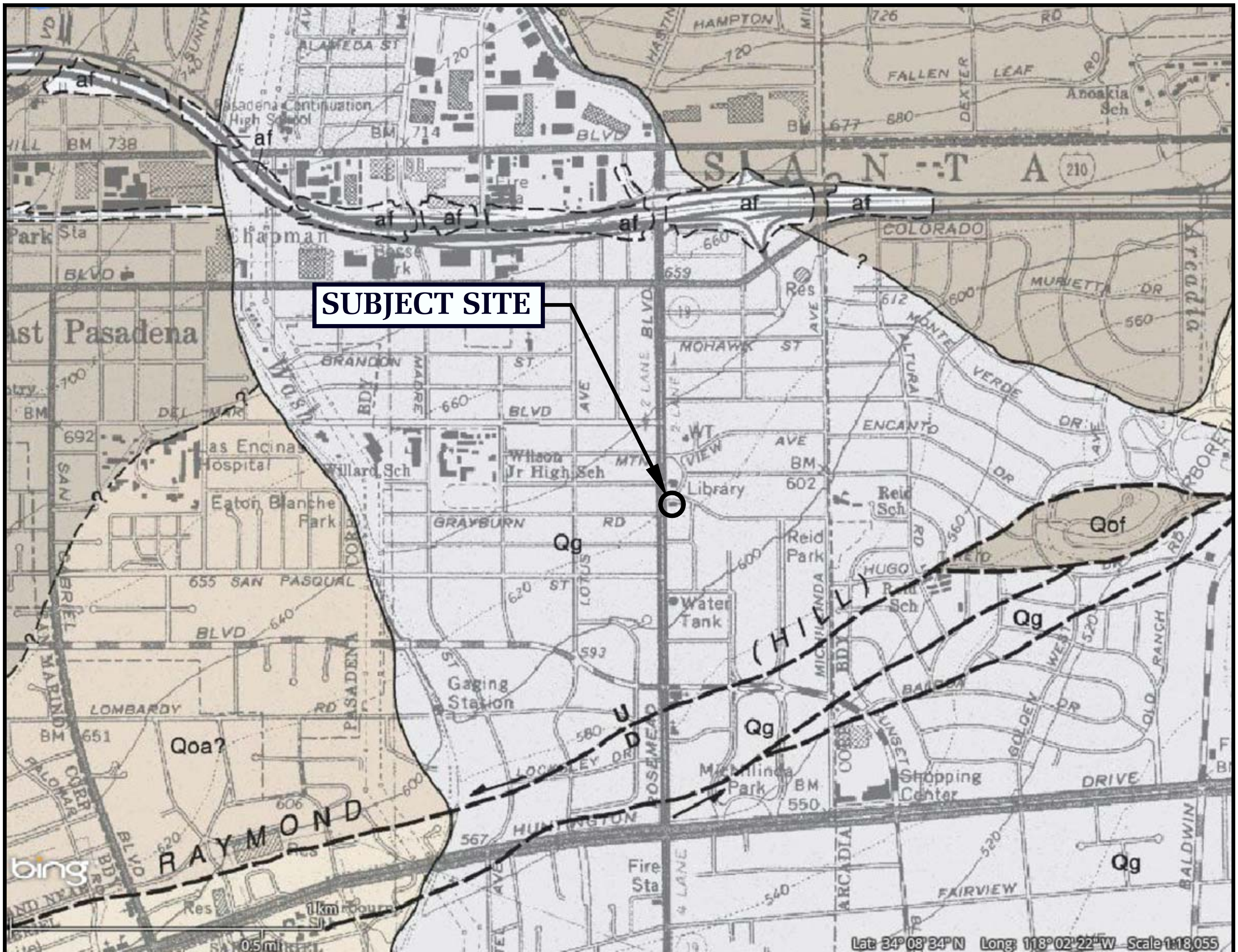
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, MOUNT WILSON, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC
380 S. ROSEMEAD BLVD., PASADENA

FILE NO. 21827



LEGEND

Qg: Surficial Sediments - gravel and sand of major stream channels and alluvial fan outwash from major canyons

Folds - arrow on axial trace of fold indicates direction of plunge

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

REFERENCE: DIBBLEE, T.W., (1998) GEOLOGIC MAP OF THE MOUNT WILSON & AZUSA QUADRANGLES (#DF-67)

N

LOCAL GEOLOGIC MAP - DIBBLEE

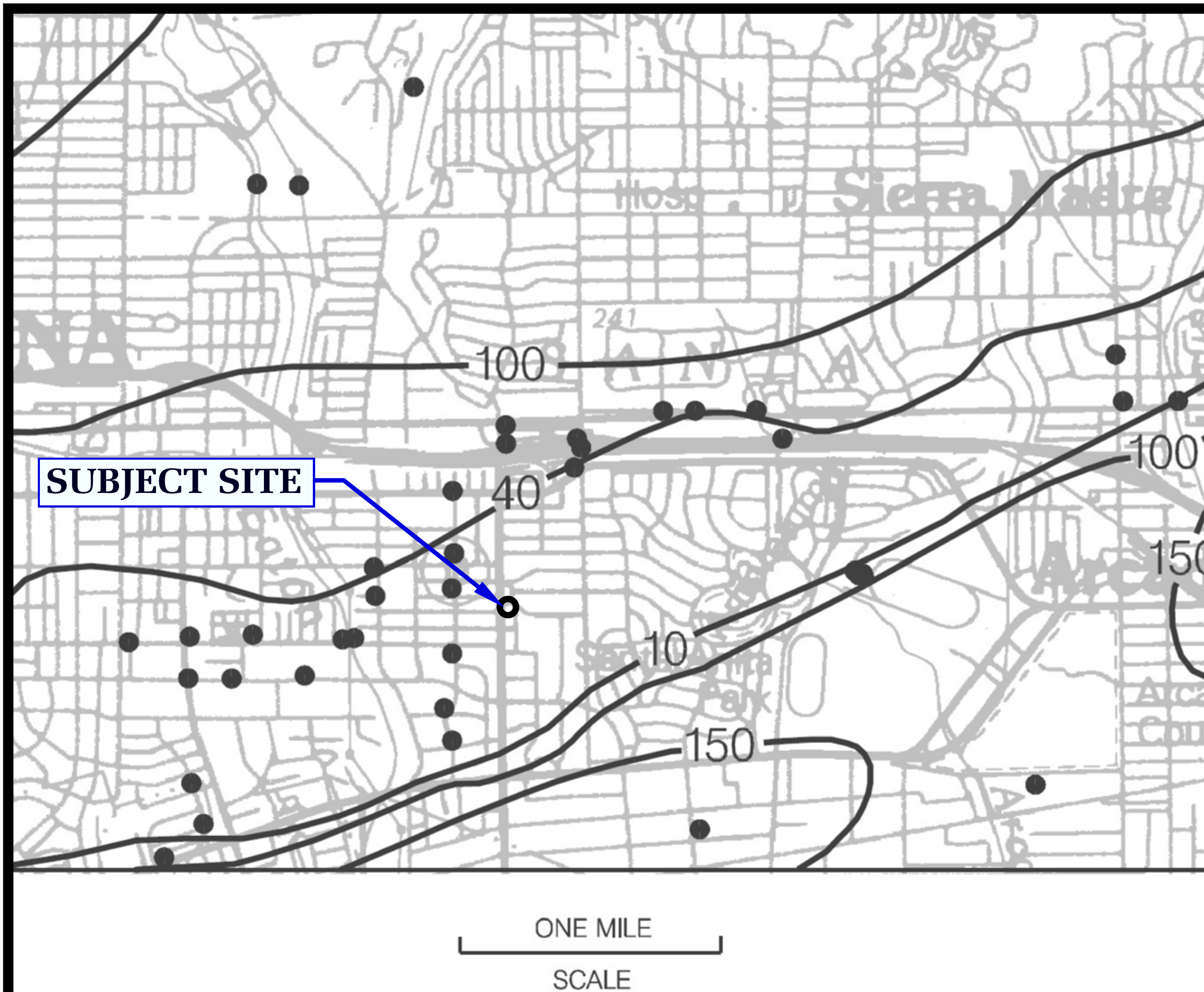
Geotechnologies, Inc.

Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

380 S. ROSEMEAD BLVD., PASADENA

FILE NO. 21827



20 Depth to groundwater in feet

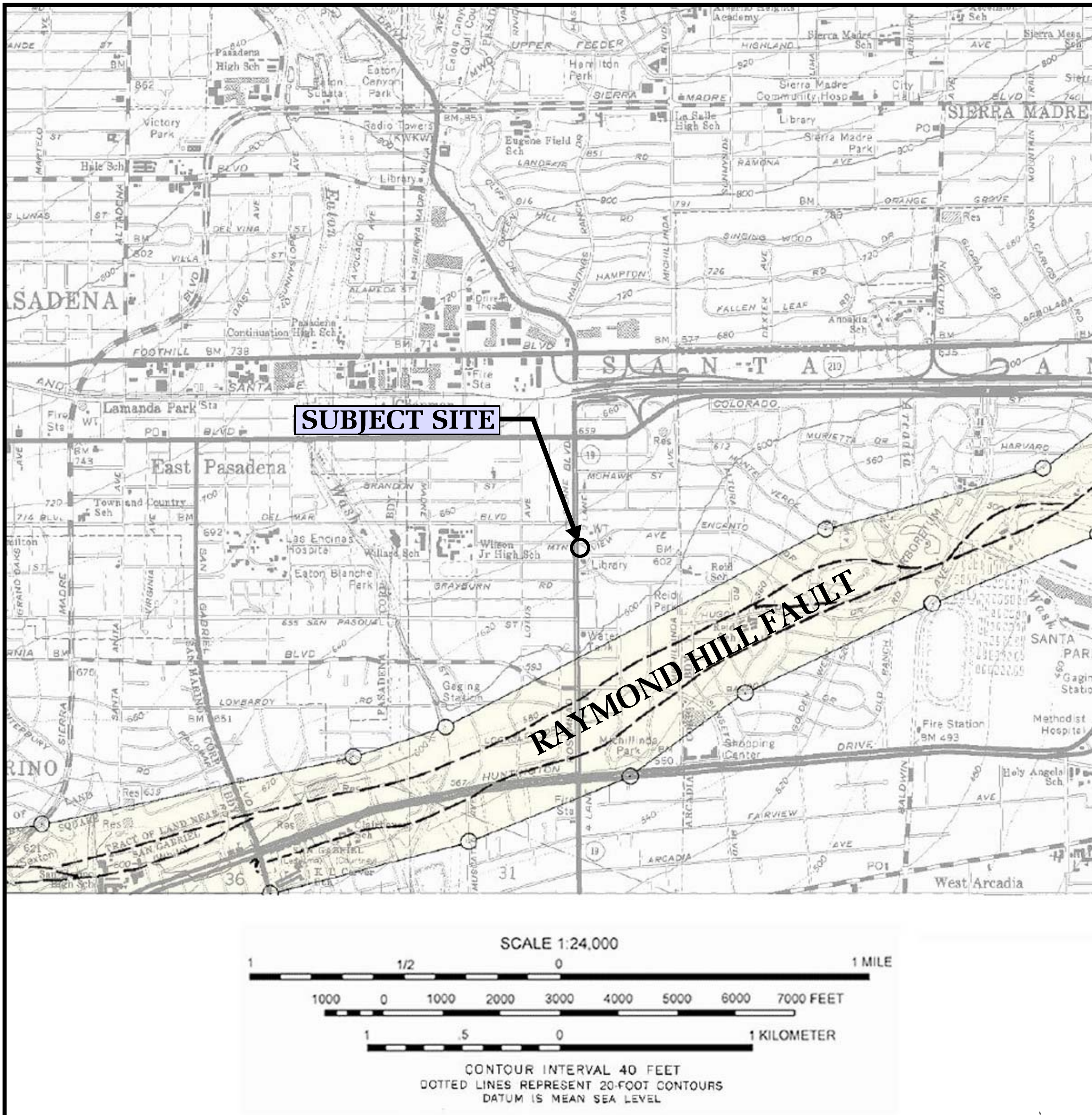
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 030
MOUNT WILSON 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC
380 S. ROSEMEAD BLVD., PASADENA

FILE NO. 21827



Earthquake Fault Zones

Alquist-Priolo Earthquake Fault Zone

REFERENCE: EARTHQUAKE FAULT ZONES, MT. WILSON QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, JANUARY 1977

EARTHQUAKE FAULT ZONE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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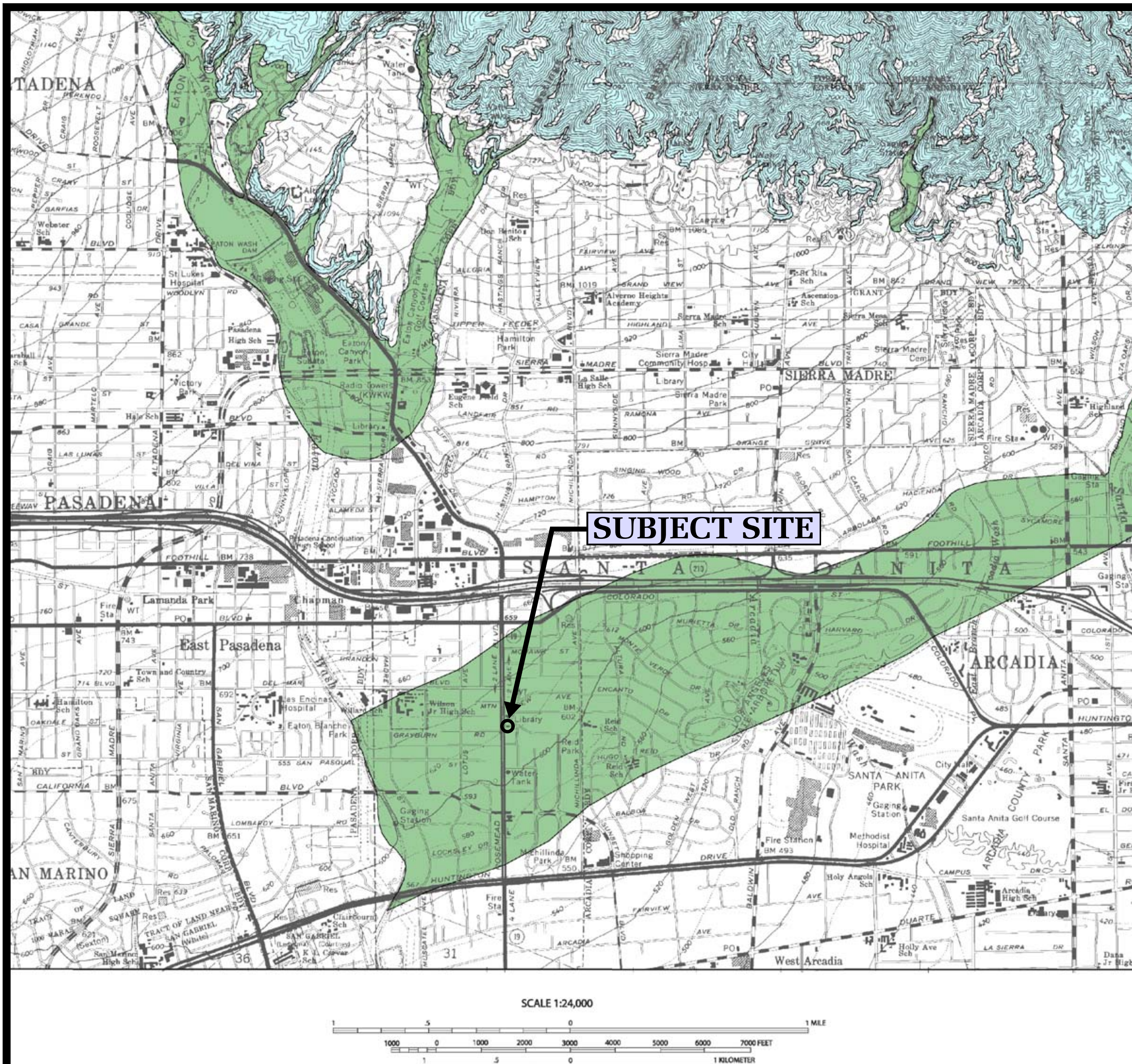
380 S. ROSEMEAD BLVD., PASADENA

FILE NO. 21827



LIQUEFACTION AREA

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

SEISMIC HAZARD ZONE MAP

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Consulting Geotechnical Engineers

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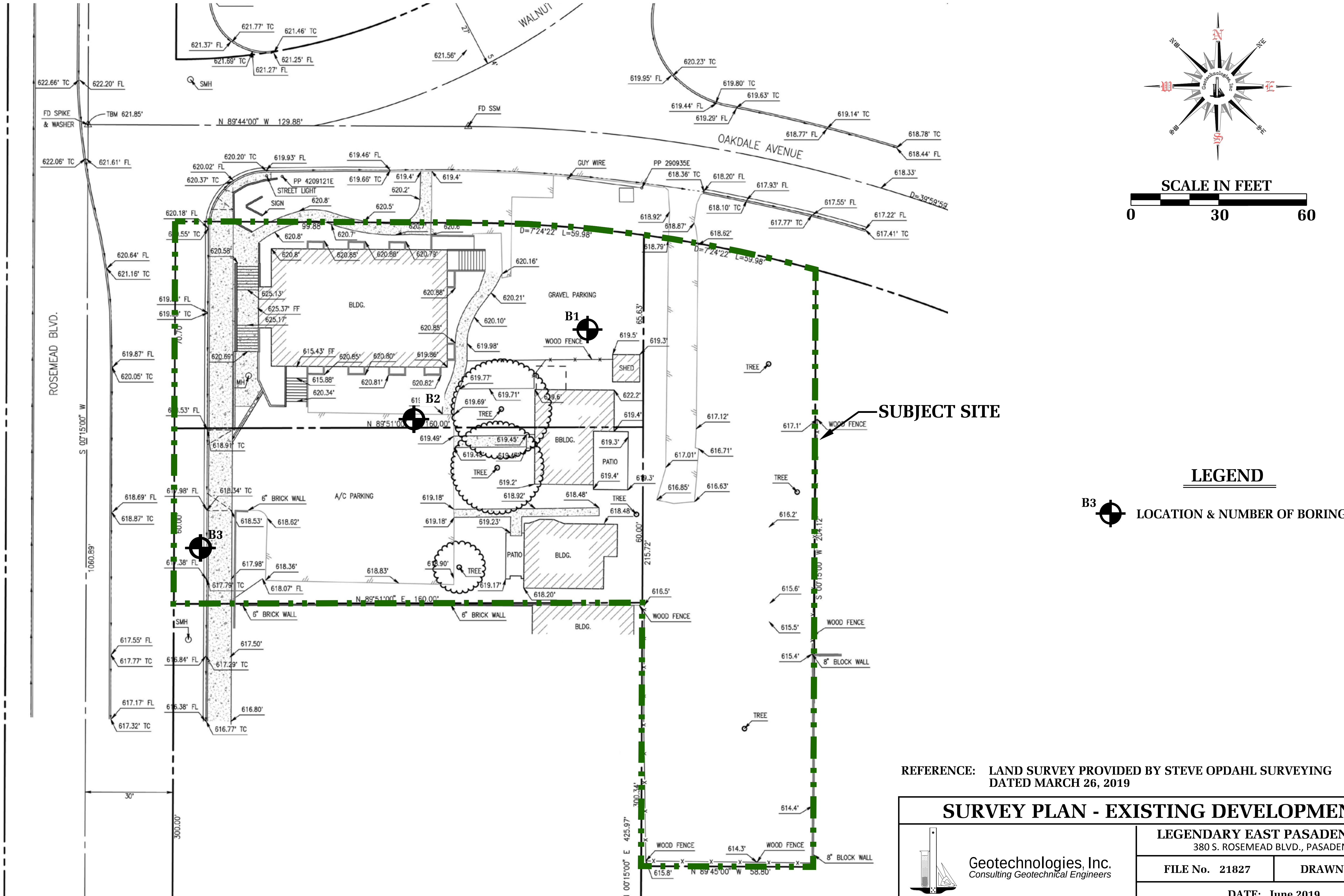
380 S. ROSEMEAD BLVD., PASADENA

FILE NO. 21827



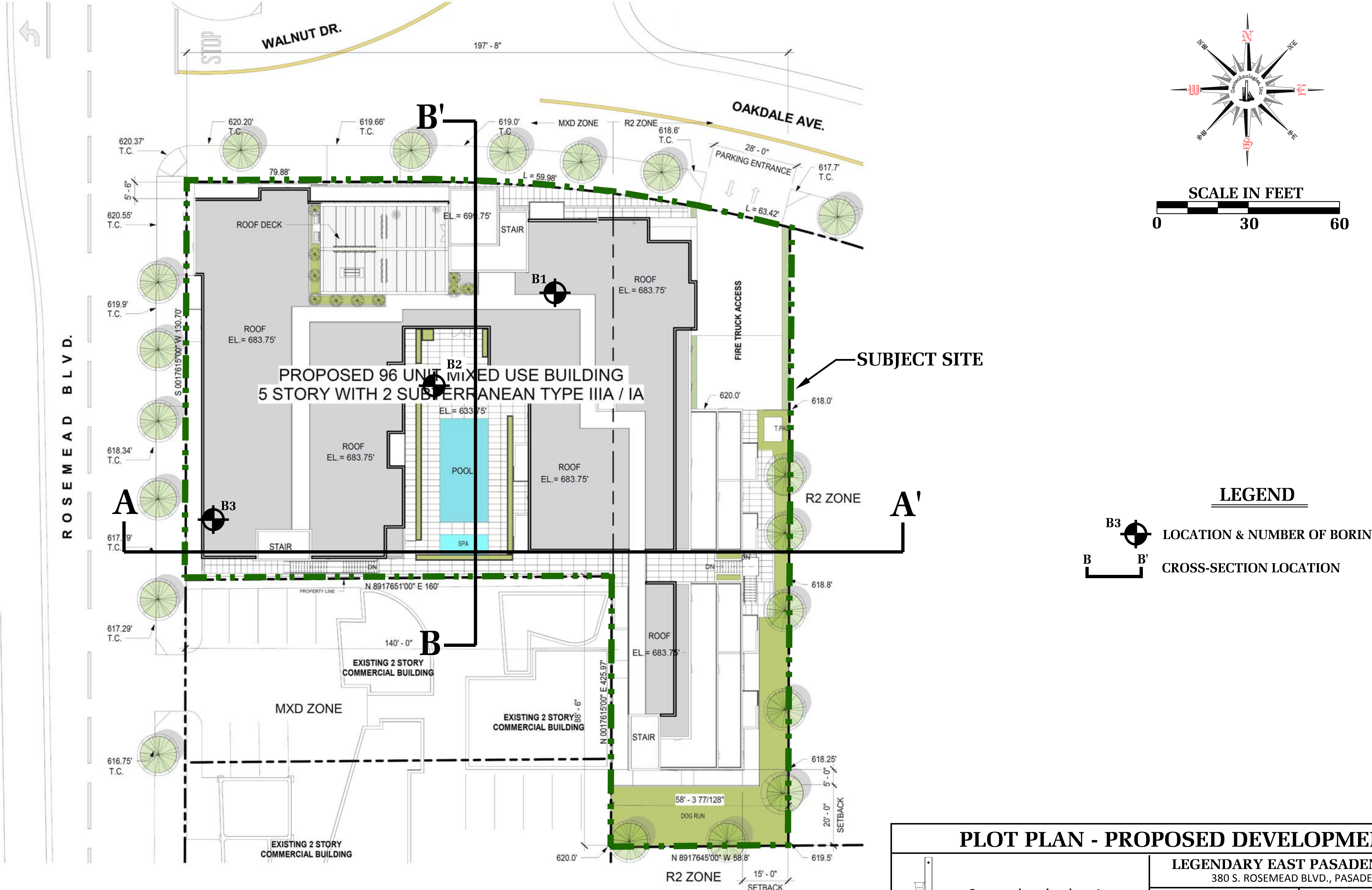
REFERENCE: LAND SURVEY PROVIDED BY STEVE OPDAHL SURVEYING
DATED MARCH 26, 2019

SURVEY PLAN - EXISTING DEVELOPMENT

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC
380 S. ROSEMEAD BLVD., PASADENA

FILE No. 21827	DRAWN BY: TC
DATE: June 2019	



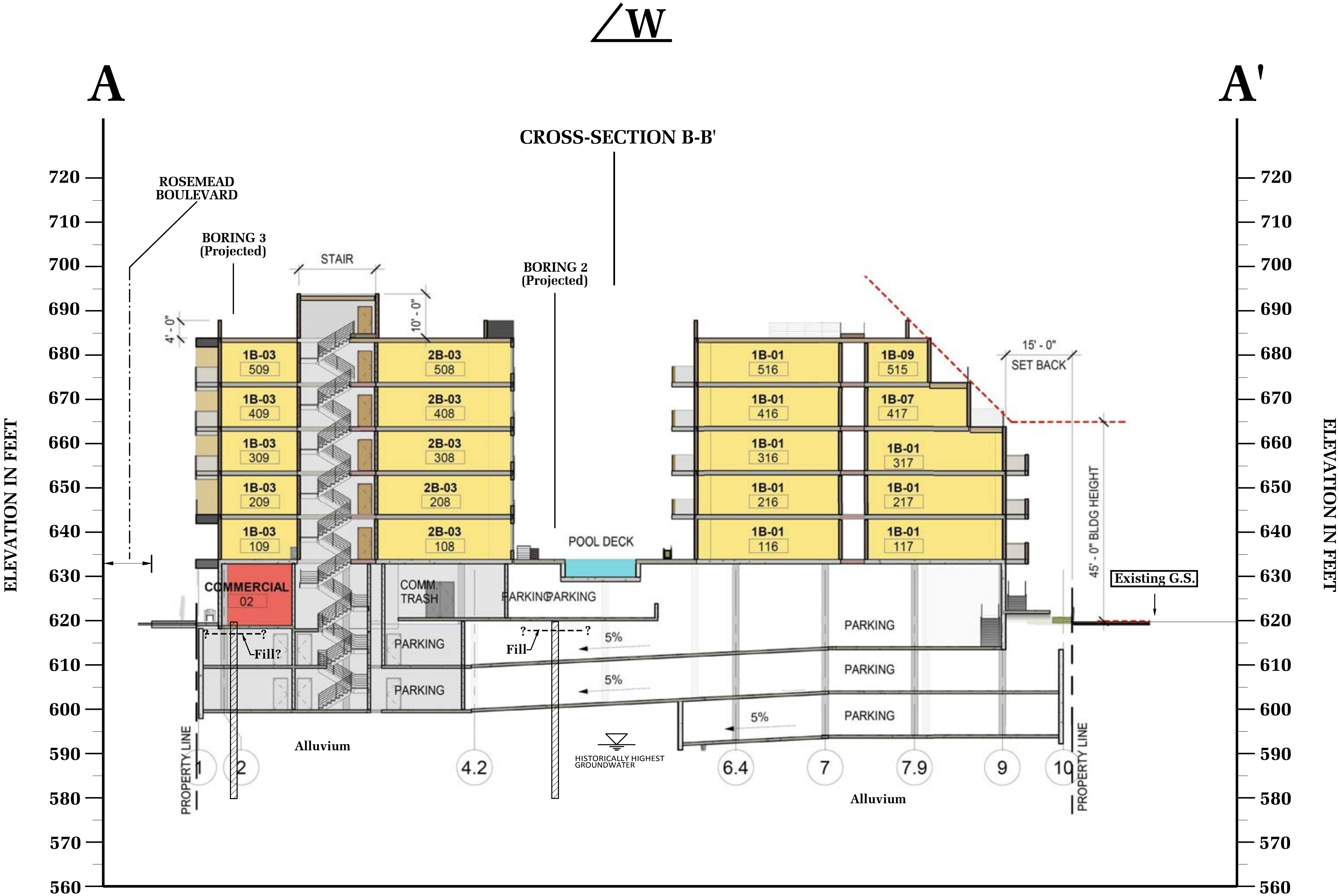
REFERENCE: SITE PLAN PROVIDED BY GMP ARCHITECTS-LA
DATED FEBRUARY 15, 2019

PLOT PLAN - PROPOSED DEVELOPMENT

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC
380 S. ROSEMEAD BLVD., PASADENA

FILE No. 21827	DRAWN BY: TC
DATE: June 2019	



SCALE IN FEET

0 30 60

REFERENCE: SITE PLAN PROVIDED BY GMP ARCHITECTS-LA DATED FEBRUARY 15, 2019

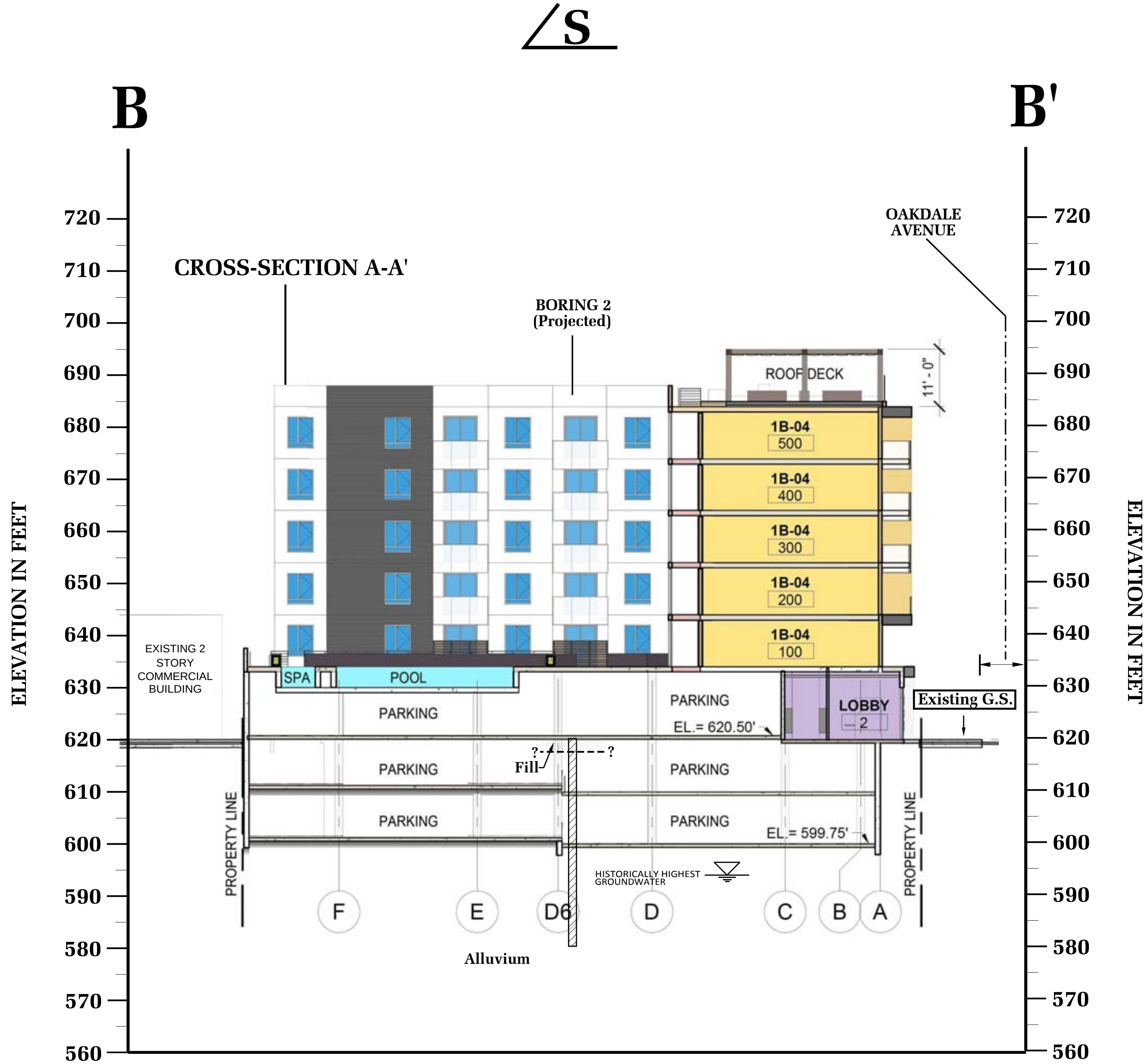
CROSS-SECTION A-A'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC
380 S. ROSEMEAD BLVD., PASADENA

FILE No. 21827 | DRAWN BY: TC

DATE: August 2019



SCALE IN FEET

0 30 60

REFERENCE: SITE PLAN PROVIDED BY GMP ARCHITECTS-LA
DATED FEBRUARY 15, 2019

CROSS-SECTION B-B'		
Geotechnologies, Inc. Consulting Geotechnical Engineers	LEGENDARY EAST PASADENA, LLC 380 S. ROSEMEAD BLVD., PASADENA	
	FILE No. 21827	DRAWN BY: TC
	DATE: August 2019	

BORING LOG NUMBER 1

Legendary East Pasadena, LLC **Date: 06/06/19** **Elevation: 619.0'***

File No. 21827 **Method: 8-inch diameter Hollow Stem Auger**

km/ae *Reference: Site Plan Provided by GMP Architects - LA, dated 02/15/2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				1 --		
				-		
				2 --		
2.5	11	3.9	109.7	-		
				3 --		
				-	SM/SP	ALLUVIUM: Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				4 --		
				-		
5	7	3.3	SPT	5 --		
				-	SP	Sand, dark and yellowish brown, moist, medium dense, fine to medium grained
				6 --		
				-		
				7 --		
7.5	43	2.4	115.8	-		
				8 --	SP/SW	Sand to Gravelly Sand, dark and gray, slightly moist, medium dense, fine to coarse grained
				-		
				9 --		
				-		
10	21	1.7	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	41	7.0	123.3	-		
				13 --	SM/SP	Silty Sand to Sand with cobbles, dark brown, moist, medium dense, fine grained
				-		
				14 --		
				-		
15	20	5.4	SPT	15 --		
				-	SP	Sand, dark brown, moist, medium dense, fine to medium grained
				16 --		
				-		
				17 --		
17.5	38	10.7	117.5	-		
				18 --	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, medium dense, fine to medium grained
				-		
				19 --		
				-		
20	18	8.9	SPT	20 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
22.5	45	9.3	132.6	-		
				23 --		
				-		
				24 --		
				-		
25	27	18.6	SPT	25 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish or grayish brown, moist, medium dense, fine to medium grained

BORING LOG NUMBER 1

Legendary East Pasadena, LLC

File No. 21827

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	46	2.8	127.3	-		
				28 --	SP/SW	Sand to Gravelly Sand, dark and gray, moist, medium dense, fine to coarse grained
				-		
				29 --		
				-		
30	26	14.8	SPT	30 --		
				-	SP/SM	Sand to Silty Sand, dark and yellowish brown, moist, medium dense, fine grained
				31 --		
				-		
				32 --		
32.5	37	2.3	121.3	-		
	50/5"			33 --	SP	Sand, dark brown, moist, very dense, fine to coarse grained
				-		
				34 --		
				-		
35	81	3.8	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	39	3.6	125.5	-		
	50/4"			38 --	SP/SW	Sand to Cobbley Sand, dark brown to gray, moist, very dense, fine to coarse grained
				-		
				39 --		
				-		
40	73	2.9	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42.5	29	7.1	120.4	-		
	50/5"			43 --	SP	Sand, dark and grayish brown, moist, very dense, fine to coarse grained, minor gravel
				-		
				44 --		
				-		
45	48	3.2	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	19	2.9	113.0	-		
	50/5"			48 --		
				-		
				49 --		
				-		
50	36	18.3	SPT	50 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, medium dense, fine to medium grained

BORING LOG NUMBER 1

Legendary East Pasadena, LLC

File No. 21827

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	37 50/5"	3.5	103.1	53	SP	Sand, dark and yellowish brown, moist, very dense, fine to medium grained
				54		
55	33 50/4"	3.6	SPT	55		
				56		
				57		
57.5	37	20.5	102.7	58	SM/ML	Silty Sand to Sandy Silt, dark and olive brown, moist, medium dense, fine grained, stiff
				59		
60	46	2.7	SPT	60		
				61	SP	Sand, dark and yellowish brown, moist, very dense, fine to medium grained
				62		
62.5	33 50/3"	3.5	117.0	63		
				64		
65	48 50/5"	No Recovery		65		
				66		
				67		
67.5	100/3"	2.2	No Recovery	68		
				69		Total Depth 68 feet by refusal No Water Fill to 3 feet
				70		
				71		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				72		
				73		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				74		
				75		SPT=Standard Penetration Test

BORING LOG NUMBER 2

Legendary East Pasadena, LLC | **Date: 06/07/19** | **Elevation: 618.0'***

File No. 21827 | **Method: 8-inch diameter Hollow Stem Auger**

km/ae | *Reference: Site Plan Provided by GMP Architects - LA, dated 02/15/2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, dark brown, moist, medium dense, fine
				-		grained
				1 --		
				-		
				2 --		
2.5	24	5.0	110.5	-		
				3 --		
				-	SM/SP	ALLUVIUM: Silty Sand to Sand, dark brown, moist, medium
				4 --		dense, fine grained
				-		
5	18	4.0	111.4	5 --		
				-	SP	Sand, dark brown, moist, medium dense, fine to medium
				6 --		grained
				-		
				7 --		
7.5	43	3.3	117.6	-		
				8 --		
				-		
				9 --		
				-		
10	40	2.8	118.6	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	43	9.2	120.0	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	45	11.4	118.3	20 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, medium dense, fine
				21 --		grained
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	83	5.8	126.9	25 --		
				-		Silty Sand to Sand, dark brown, moist, dense, fine grained

BORING LOG NUMBER 2

Legendary East Pasadena, LLC

File No. 21827

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	54	8.4	111.2	30 --		
	50/5"			-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	79	4.8	113.9	35 --		
				-	SP	Sand, dark brown, moist, very dense, fine to medium grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	41	4.2	117.8	40 --		
	50/5"			-		Total Depth 40 feet
				41 --		No Water
				-		Fill to 3 feet
				42 --		
				-		
				43 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				44 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				45 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 3

Legendary East Pasadena, LLC **Date: 06/07/19** **Elevation: 618.0'***

File No. 21827 **Method: 8-inch diameter Hollow Stem Auger**

km/ae *Reference: Site Plan Provided by GMP Architects - LA, dated 02/15/2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Planter Area
				0 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				1 --		
				-		
				2 --		
2.5	28	3.3	119.5	-		
				3 --		
				-	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				4 --		
				-		
5	25	11.1	123.7	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	32	11.5	112.0	-		
				8 --	SM/SP	Silty Sand to Sand, dark and gray, moist, medium dense, fine grained
				-		
				9 --		
				-		
10	41	5.0	119.6	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	30	10.2	122.7	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	56	9.3	123.5	20 --		
				-	SP	Sand, dark and gray, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	61 50/4"	12.3	115.9	25 -- -		dark and yellowish brown, very dense

BORING LOG NUMBER 3

Legendary East Pasadena, LLC

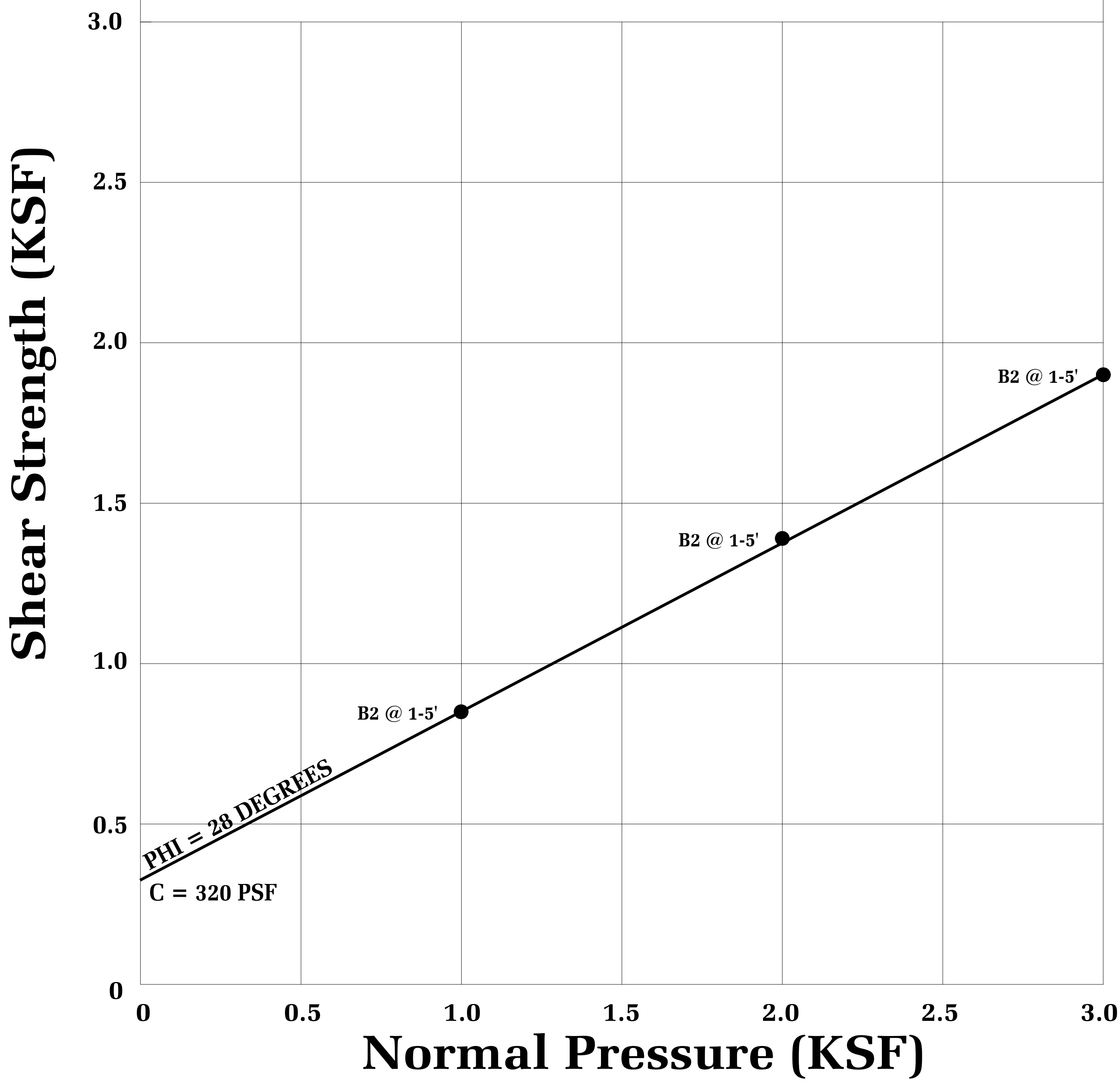
File No. 21827

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	62 50/5"	9.4	111.6	30		
				31		
				32		
				33		
				34		
35	100/9"	4.2	115.3	35		Sand with Cobbles
				36		
				37		
				38		
				39		Sand, medium dense, fine to medium grained
40	30	3.0	112.8	40		Total Depth 40 feet No Water Fill to 3 feet
				41		
				42		
				43		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				44		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				45		
				46		
				47		
				48		
				49		
				50		

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 1-5'	SM/SP	117.0	9.2	15.6



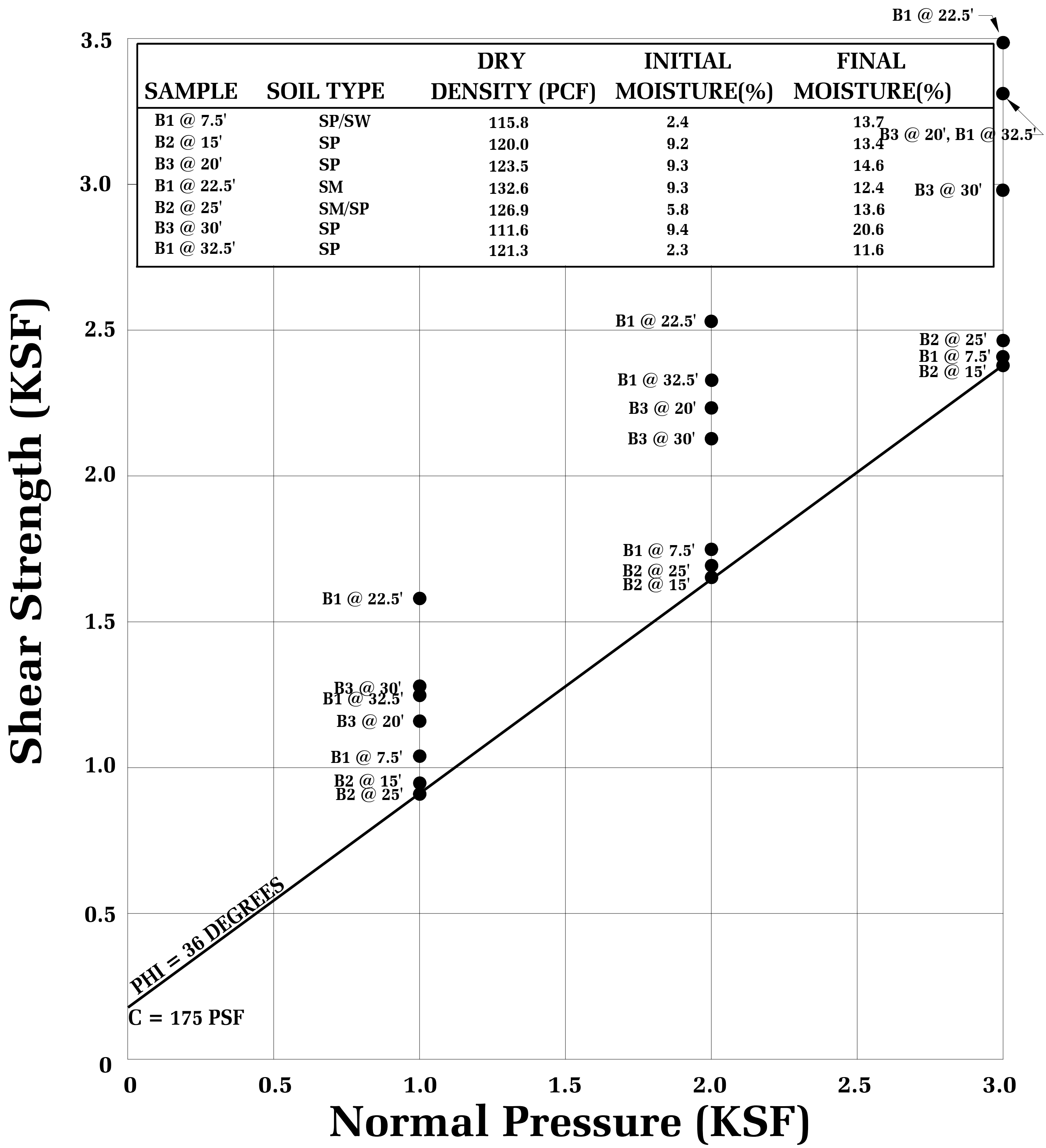
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

FILE NO. 21827	PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 7.5'	SP/SW	115.8	2.4	13.7
B2 @ 15'	SP	120.0	9.2	13.4
B3 @ 20'	SP	123.5	9.3	14.6
B1 @ 22.5'	SM	132.6	9.3	12.4
B2 @ 25'	SM/SP	126.9	5.8	13.6
B3 @ 30'	SP	111.6	9.4	20.6
B1 @ 32.5'	SP	121.3	2.3	11.6

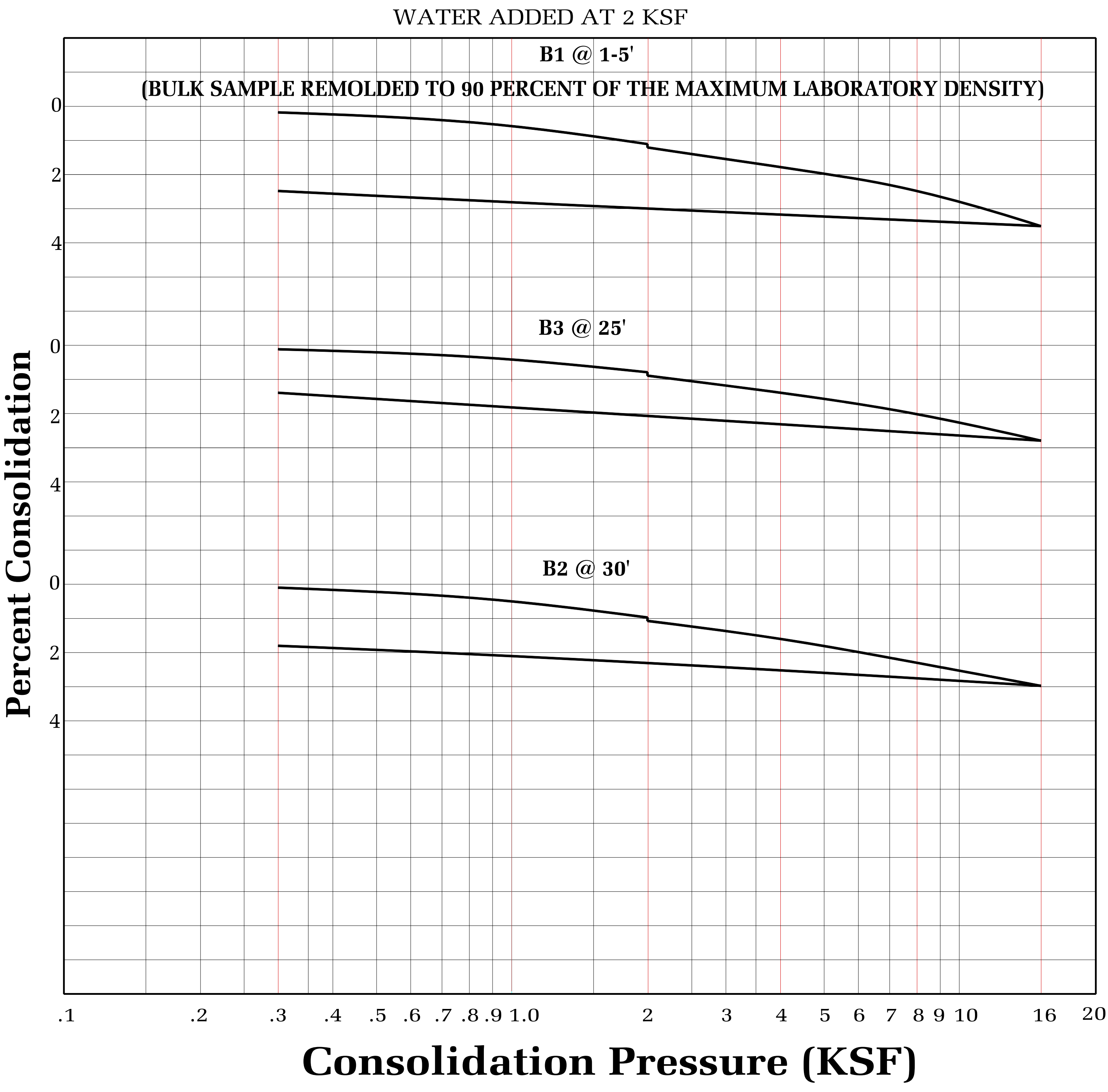
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

FILE NO. 21827 | PLATE: B-2

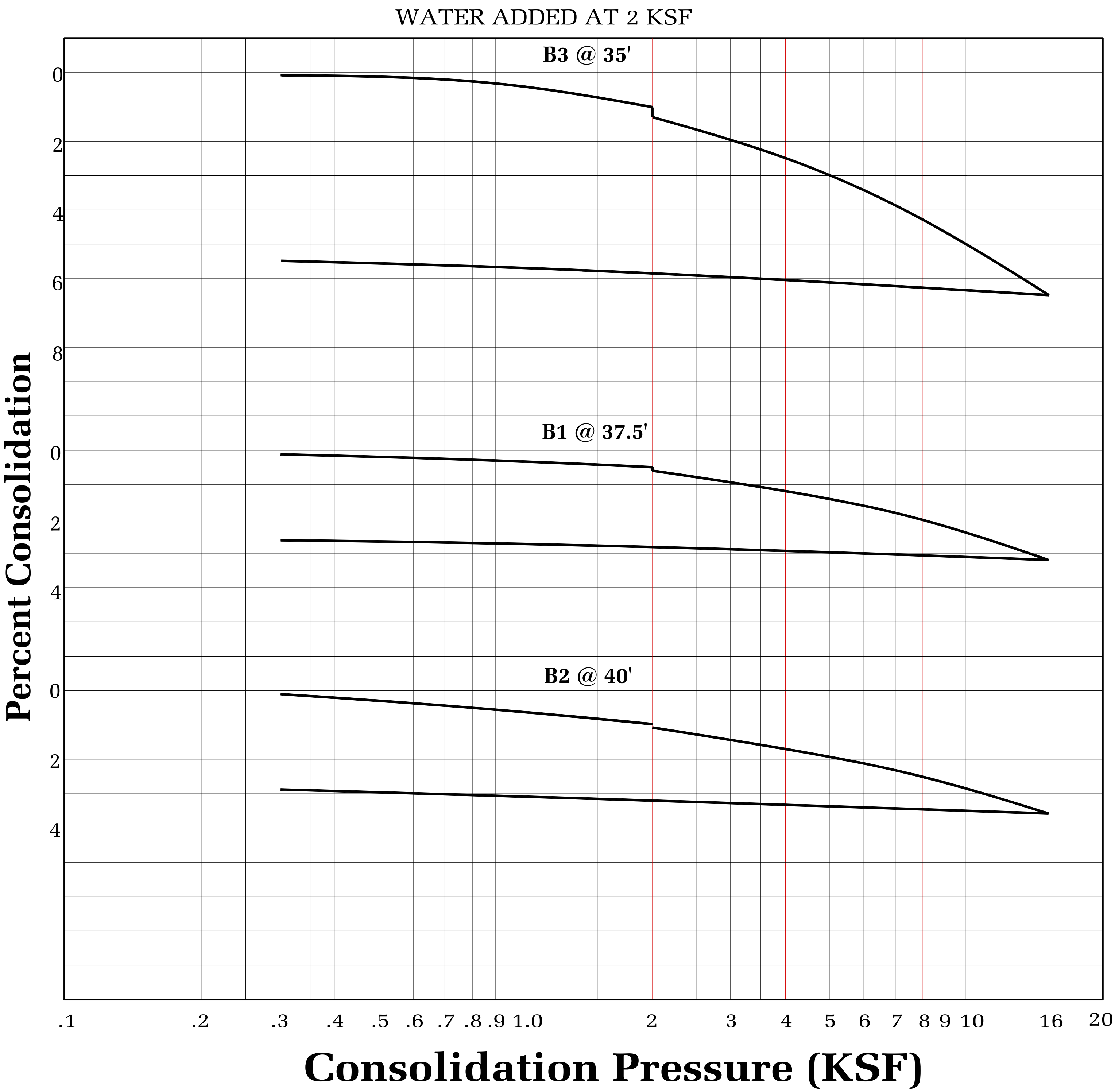


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

FILE NO. 21827 | **PLATE: C-1**

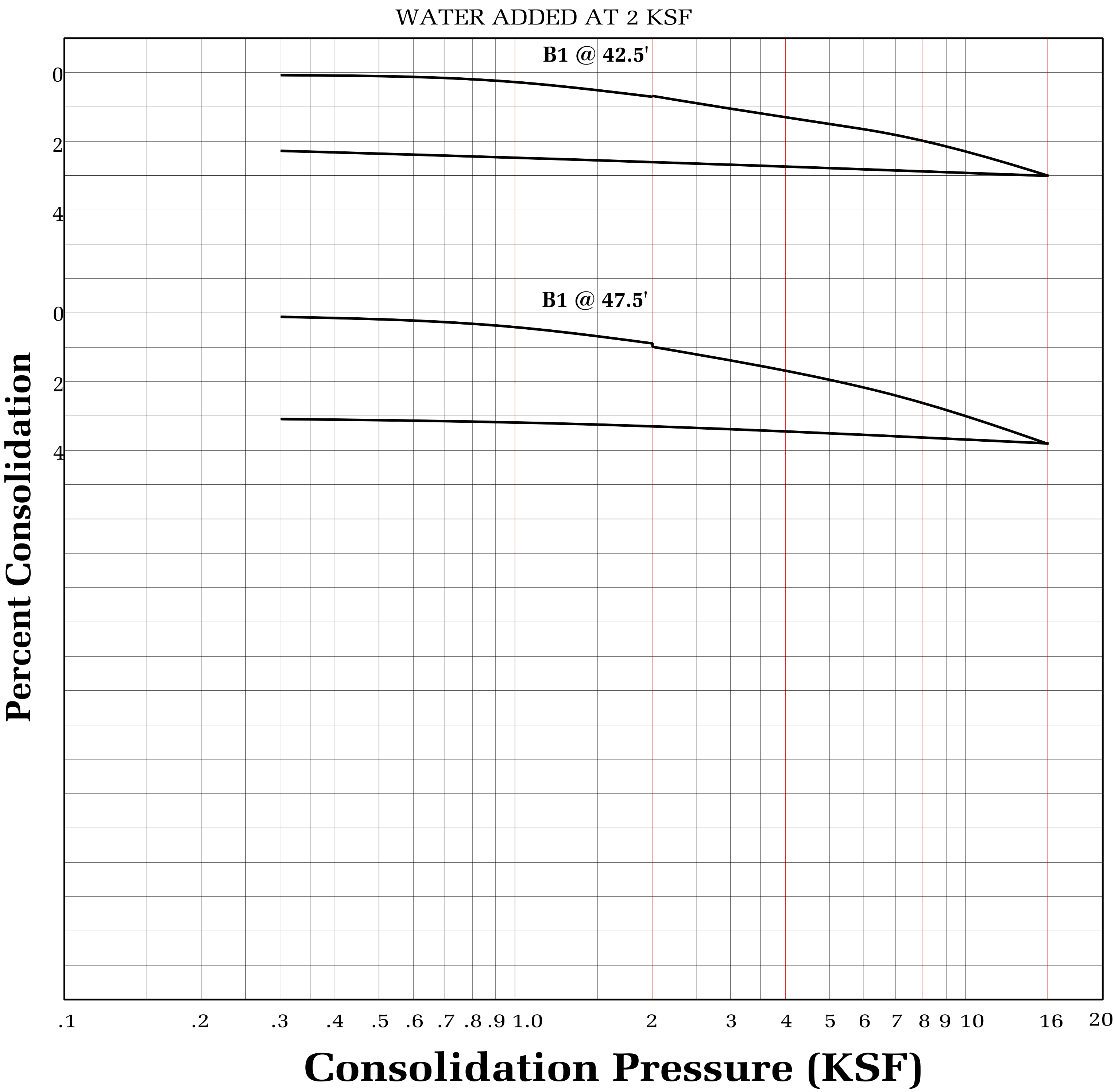


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

FILE NO. 21827	**PLATE: C-2**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

FILE NO. 21827 | **PLATE: C-3**

ASTM D-1557

SAMPLE	B2 @ 1-5'
SOIL TYPE:	SM/SP
MAXIMUM DENSITY pcf.	130.0
OPTIMUM MOISTURE %	9.2

ASTM D 4829

SAMPLE	B2 @ 1-5'
SOIL TYPE:	SM/SP
EXPANSION INDEX UBC STANDARD 18-2	7
EXPANSION CHARACTER	VERY LOW

SULFATE CONTENT

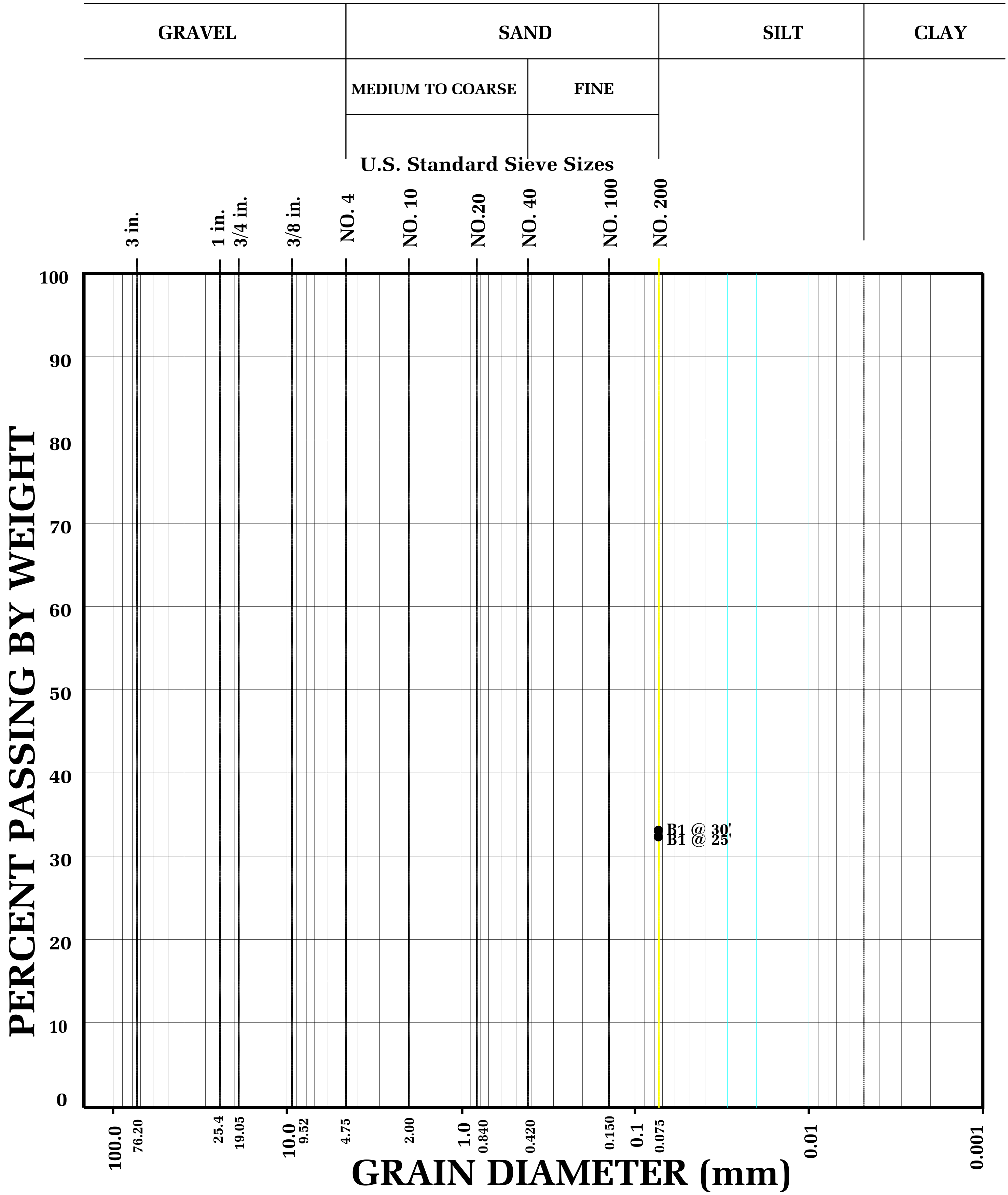
SAMPLE	B2 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.1 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

FILE NO. 21827 | PLATE: D



GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LEGENDARY EAST PASADENA, LLC

FILE NO. 21827

PLATE: E

Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Liquefaction Analysis (2% Exceedance in 50 Years)
Boring No: 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.5
Peak Ground Horizontal Acceleration, PGA (g):	1.06
Calculated Mag.Wtg.Factor:	1.301

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	69.0
Historically Highest Groundwater Level* (ft):	28.0
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):	12
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc} (psf)	Effective Vert. Stress σ_{vc}' (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	114.0	Unsaturated	Unsaturated	7	5	0.0	0	114.0	114.0	14.5	1.00	0.693	0.217	Non-Liq.	0.00
2	114.0	Unsaturated	Unsaturated	7	5	0.0	0	228.0	228.0	14.5	1.00	0.690	0.217	Non-Liq.	0.00
3	114.0	Unsaturated	Unsaturated	7	5	0.0	0	342.0	342.0	14.5	1.00	0.687	0.217	Non-Liq.	0.00
4	114.0	Unsaturated	Unsaturated	7	5	0.0	0	456.0	456.0	14.5	0.99	0.685	0.217	Non-Liq.	0.00
5	114.0	Unsaturated	Unsaturated	7	5	0.0	0	570.0	570.0	15.6	0.99	0.682	0.230	Non-Liq.	0.00
6	114.0	Unsaturated	Unsaturated	7	5	0.0	0	684.0	684.0	15.6	0.98	0.679	0.230	Non-Liq.	0.00
7	114.0	Unsaturated	Unsaturated	7	5	0.0	0	798.0	798.0	14.7	0.98	0.676	0.220	Non-Liq.	0.00
8	118.6	Unsaturated	Unsaturated	21	10	0.0	0	916.6	916.6	40.2	0.98	0.673	2.000	Non-Liq.	0.00
9	118.6	Unsaturated	Unsaturated	21	10	0.0	0	1035.2	1035.2	41.1	0.97	0.670	2.000	Non-Liq.	0.00
10	118.6	Unsaturated	Unsaturated	21	10	0.0	0	1153.8	1153.8	39.9	0.97	0.666	2.000	Non-Liq.	0.00
11	118.6	Unsaturated	Unsaturated	21	10	0.0	0	1272.4	1272.4	38.9	0.96	0.663	2.000	Non-Liq.	0.00
12	118.6	Unsaturated	Unsaturated	21	10	0.0	0	1391.0	1391.0	38.0	0.96	0.659	2.000	Non-Liq.	0.00
13	131.9	Unsaturated	Unsaturated	21	10	0.0	0	1522.9	1522.9	36.9	0.95	0.656	2.000	Non-Liq.	0.00
14	131.9	Unsaturated	Unsaturated	21	10	0.0	0	1654.8	1654.8	35.7	0.95	0.652	1.778	Non-Liq.	0.00
15	131.9	Unsaturated	Unsaturated	20	15	0.0	0	1786.7	1786.7	37.2	0.94	0.648	2.000	Non-Liq.	0.00
16	131.9	Unsaturated	Unsaturated	20	15	0.0	0	1918.6	1918.6	36.2	0.93	0.645	1.917	Non-Liq.	0.00
17	131.9	Unsaturated	Unsaturated	20	15	0.0	0	2050.5	2050.5	35.2	0.93	0.641	1.512	Non-Liq.	0.00
18	130.0	Unsaturated	Unsaturated	20	15	0.0	0	2180.5	2180.5	34.3	0.92	0.637	1.238	Non-Liq.	0.00
19	130.0	Unsaturated	Unsaturated	20	15	0.0	0	2310.5	2310.5	33.4	0.92	0.633	1.041	Non-Liq.	0.00
20	145.0	Unsaturated	Unsaturated	18	20	0.0	0	2455.5	2455.5	28.5	0.91	0.629	0.515	Non-Liq.	0.00
21	145.0	Unsaturated	Unsaturated	18	20	0.0	0	2600.5	2600.5	27.8	0.91	0.624	0.469	Non-Liq.	0.00
22	145.0	Unsaturated	Unsaturated	18	20	0.0	0	2745.5	2745.5	27.1	0.90	0.620	0.432	Non-Liq.	0.00
23	145.0	Unsaturated	Unsaturated	18	20	0.0	0	2890.5	2890.5	26.4	0.89	0.616	0.402	Non-Liq.	0.00
24	145.0	Unsaturated	Unsaturated	18	20	0.0	0	3035.5	3035.5	25.7	0.89	0.612	0.376	Non-Liq.	0.00
25	145.0	Unsaturated	Unsaturated	27	25	32.3	0	3180.5	3180.5	48.1	0.88	0.607	2.000	Non-Liq.	0.00
26	145.0	Unsaturated	Unsaturated	27	25	32.3	0	3325.5	3325.5	47.5	0.87	0.603	2.000	Non-Liq.	0.00
27	145.0	Unsaturated	Unsaturated	27	25	32.3	0	3470.5	3470.5	46.9	0.87	0.599	2.000	Non-Liq.	0.00
28	130.8	Unsaturated	Unsaturated	27	25	32.3	0	3601.3	3601.3	48.9	0.86	0.594	2.000	Non-Liq.	0.00
29	130.8	Unsaturated	Saturated	27	25	32.3	0	3732.1	3669.7	48.4	0.86	0.600	2.000	3.3	0.00
30	130.8	Unsaturated	Saturated	26	30	33.1	0	3862.9	3738.1	45.9	0.85	0.605	2.000	3.3	0.00
31	130.8	Unsaturated	Saturated	26	30	33.1	0	3993.7	3806.5	45.3	0.84	0.609	2.000	3.3	0.00
32	130.8	Unsaturated	Saturated	26	30	33.1	0	4124.5	3874.9	44.7	0.84	0.613	2.000	3.3	0.00
33	124.1	Unsaturated	Saturated	81	35	0.0	0	4248.6	3936.6	125.9	0.83	0.617	2.000	3.2	0.00
34	124.1	Unsaturated	Saturated	81	35	0.0	0	4372.7	3998.3	125.0	0.82	0.620	2.000	3.2	0.00
35	124.1	Unsaturated	Saturated	81	35	0.0	0	4496.8	4060.0	124.0	0.82	0.623	2.000	3.2	0.00
36	124.1	Unsaturated	Saturated	81	35	0.0	0	4620.9	4121.7	123.1	0.81	0.626	1.999	3.2	0.00
37	124.1	Unsaturated	Saturated	81	35	0.0	0	4745.0	4183.4	122.3	0.80	0.628	1.979	3.2	0.00
38	130.0	Unsaturated	Saturated	73	40	0.0	0	4875.0	4251.0	109.4	0.80	0.630	1.958	3.1	0.00
39	130.0	Unsaturated	Saturated	73	40	0.0	0	5005.0	4318.6	108.7	0.79	0.631	1.938	3.1	0.00
40	130.0	Unsaturated	Saturated	73	40	0.0	0	5135.0	4386.2	107.9	0.78	0.632	1.918	3.0	0.00
41	130.0	Unsaturated	Saturated	73	40	0.0	0	5265.0	4453.8	107.2	0.78	0.633	1.899	3.0	0.00
42	130.0	Unsaturated	Saturated	73	40	0.0	0	5395.0	4521.4	106.6	0.77	0.633	1.881	3.0	0.00
43	129.0	Unsaturated	Saturated	48	45	0.0	0	5524.0	4588.0	69.6	0.76	0.634	1.862	2.9	0.00
44	129.0	Unsaturated	Saturated	48	45	0.0	0	5653.0	4654.6	69.2	0.76	0.634	1.845	2.9	0.00
45	129.0	Unsaturated	Saturated	48	45	0.0	0	5782.0	4721.2	68.8	0.75	0.634	1.827	2.9	0.00
46	129.0	Unsaturated	Saturated	48	45	0.0	0	5911.0	4787.8	68.4	0.74	0.633	1.810	2.9	0.00
47	129.0	Unsaturated	Saturated	48	45	0.0	0	6040.0	4854.4	68.0	0.74	0.633	1.794	2.8	0.00
48	116.3	Unsaturated	Saturated	48	45	0.0	0	6156.3	4908.3	67.7	0.73	0.632	1.779	2.8	0.00
49	116.3	Unsaturated	Saturated	48	45	0.0	0	6272.6	4962.2	67.3	0.72	0.632	1.765	2.8	0.00
50	116.3	Unsaturated	Saturated	36	50	0.0	0	6388.9	5016.1	50.3	0.72	0.631	1.751	2.8	0.00
51	116.3	Unsaturated	Saturated	36	50	0.0	0	6505.2	5070.0	50.0	0.71	0.630	1.737	2.8	0.00
52	116.3	Unsaturated	Saturated	36	50	0.0	0	6621.5	5123.9	49.8	0.71	0.629	1.723	2.7	0.00
53	106.7	Unsaturated	Saturated	83	55	0.0	0	6728.2	5168.2	114.3	0.70	0.629	1.711	2.7	0.00
54	106.7	Unsaturated	Saturated	83	55	0.0	0	6834.9	5212.5	113.8	0.69	0.628	1.699	2.7	0.00
55	106.7	Unsaturated	Saturated	83	55	0.0	0	6941.6	5256.8	113.4	0.69	0.627	1.687	2.7	0.00
56	106.7	Unsaturated	Saturated	83	55	0.0	0	7048.3	5301.1	112.9	0.68	0.626	1.675	2.7	0.00
57	106.7	Unsaturated	Saturated	83	55	0.0	0	7155.0	5345.4	112.5	0.68	0.624	1.664	2.7	0.00
58	123.8	Unsaturated	Saturated	83	55	0.0	0	7278.8	5406.8	112.0	0.67	0.623	1.651	2.7	0.00
59	123.8	Unsaturated	Saturated	83	55	0.0	0	7402.6	5468.2	111.5	0.67	0.621	1.638	2.6	0.00
60	121.0	Unsaturated	Saturated	46	60	0.0	0	7523.6	5526.8	61.5	0.66	0.619	1.625	2.6	0.00
61	121.0	Unsaturated	Saturated	46	60	0.0	0	7644.6	5585.4	61.3	0.65	0.617	1.613	2.6	0.00
62	121.0	Unsaturated	Saturated	46	60	0.0	0	7765.6	5644.0	61.0	0.65	0.615	1.601	2.6	0.00
63	121.0	Unsaturated	Saturated	46	60	0.0	0	7886.6	5702.6	60.8	0.64	0.613	1.589	2.6	0.00
64	121.0	Unsaturated	Saturated	46	60	0.0	0	8007.6	5761.2	60.5	0.64	0.611	1.577	2.6	0.00
65	121.0	Unsaturated	Saturated	98	65	0.0	0	8128.6	5819.8	128.4	0.63	0.609	1.566	2.6	0.00
66	121.0	Unsaturated	Saturated	98	65	0.0	0	8249.6	5878.4	127.9	0.63	0.607	1.555	2.6	0.00
67	121.0	Unsaturated	Saturated	98	65	0.0	0	8370.6	5937.0	127.4	0.62	0.605	1.543	2.6	0.00
68	121.0	Unsaturated	Saturated	98	65	0.0	0	8491.6	5995.6	127.0	0.62	0.603	1.532	2.5	0.00

Total Liquefaction Settlement, S = 0.00 inches

GEOTECHNOLOGIES, INC.

FILE NO.: 21827
PROJECT: Legendary East Pasadena, LLC
BORING 1

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS (Existing Water Level Conditions)

INPUT:

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.5
Peak Horiz. Acceleration (g):	1.06

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Soil Type	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	Percent Passing 200 Sieve	ΔN for Fines Content	Fines Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] / [Gmax]	[geff]	[geff]*100%	Volumetric Strain [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
5.0	5.0	SM	2.5	145.0	0.18	0.12	0.125	27	1.3	100.0	1.60	56.2	32.3	5.5	61.7	615.392	1.93E-04	4.00E-03	4.00E-01	2.70E-02	7.3035	0.0195	0.02
10.0	5.0	SM/SP	7.5	130.8	0.53	0.35	0.360	26	1.3	99.0	1.32	44.6	33.1	5.5	50.1	978.355	3.21E-04	1.80E-03	1.80E-01	3.10E-02	7.3035	0.0224	0.03
15.0	5.0	SP	12.5	124.1	0.84	0.57	0.573	81	1.3	100.0	1.08	113.7	0.0	0.0	113.7	1629.149	2.82E-04	8.00E-04	8.00E-02	1.00E-03	7.3035	0.0007	0.00
20.0	5.0	SP/SW	17.5	130.0	1.16	0.78	0.777	73	1.3	100.0	0.96	91.1	0.0	0.0	91.1	1774.900	3.27E-04	8.00E-04	8.00E-02	1.00E-03	7.3035	0.0007	0.00
25.0	5.0	SP	22.5	129.0	1.49	1.00	0.975	48	1.3	100.0	0.84	52.4	0.0	0.0	52.4	1669.202	4.09E-04	1.30E-03	1.30E-01	2.00E-02	7.3035	0.0145	0.02
30.0	5.0	SP	27.5	116.3	1.79	1.20	1.148	36	1.3	90.0	0.79	37.0	0.0	0.0	37.0	1631.977	4.66E-04	1.40E-03	1.40E-01	5.00E-02	7.3035	0.0362	0.04
35.0	5.0	SP	32.5	106.7	2.07	1.39	1.291	83	1.3	100.0	0.73	78.8	0.0	0.0	78.8	2257.299	3.60E-04	8.00E-04	8.00E-02	2.00E-03	7.3035	0.0014	0.00
40.0	5.0	SM/ML	37.5	123.8	2.36	1.58	1.425	46	1.3	93.0	0.69	41.3	0.0	0.0	41.3	1942.100	4.43E-04	1.20E-03	1.20E-01	3.00E-02	7.3035	0.0217	0.03
45.0	5.0	SP	42.5	121.0	2.67	1.79	1.556	98	1.3	100.0	0.64	81.5	0.0	0.0	81.5	2590.304	3.49E-04	7.00E-04	7.00E-02	1.50E-03	7.3035	0.0011	0.00
50.0	5.0	SP	47.5	121.0	2.97	1.99	1.668	98	1.3	100.0	0.60	76.4	0.0	0.0	76.4	2675.164	3.51E-04	6.80E-04	6.80E-02	2.00E-03	7.3035	0.0014	0.00

Total Calculated Dynamic Dry Settlement (inches) 0.14

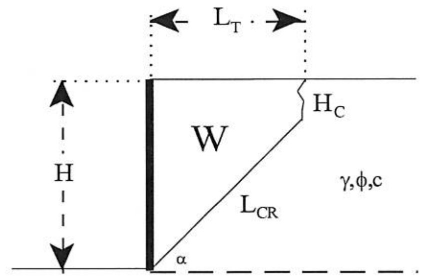
Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Cantilever Retaining Walls (Up to 15 feet)

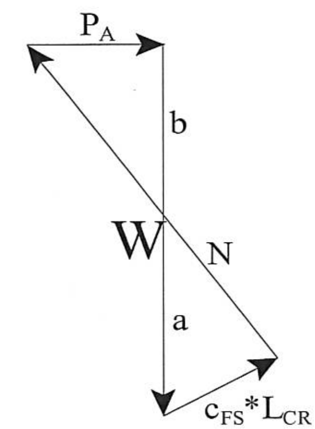
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	25.8 degrees
	(c_{FS})	116.7 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.6	106	13243.4	16.1	5149.6	8093.9	2811.7
46	3.5	103	12836.7	16.0	4867.5	7969.2	2925.2
47	3.4	99	12434.7	15.8	4609.3	7825.4	3028.4
48	3.3	96	12038.4	15.7	4372.6	7665.9	3121.5
49	3.3	93	11648.4	15.6	4154.9	7493.5	3204.9
50	3.2	90	11265.1	15.4	3954.5	7310.5	3278.8
51	3.1	87	10888.6	15.3	3769.5	7119.1	3343.3
52	3.1	84	10519.1	15.1	3598.5	6920.6	3398.8
53	3.1	81	10156.4	15.0	3439.9	6716.5	3445.3
54	3.0	78	9800.5	14.8	3292.6	6507.9	3483.1
55	3.0	76	9451.2	14.6	3155.6	6295.6	3512.2
56	3.0	73	9108.3	14.5	3027.8	6080.5	3532.7
57	3.0	70	8771.6	14.3	2908.4	5863.2	3544.8
58	3.0	68	8440.8	14.2	2796.6	5644.2	3548.3
59	3.0	65	8115.7	14.0	2691.7	5424.0	3543.4
60	3.0	62	7795.9	13.9	2593.0	5202.9	3530.1
61	3.0	60	7481.3	13.7	2500.0	4981.3	3508.2
62	3.0	57	7171.5	13.6	2412.1	4759.4	3477.8
63	3.1	55	6866.4	13.4	2328.9	4537.4	3438.6
64	3.1	53	6565.5	13.2	2249.9	4315.6	3390.7
65	3.1	50	6268.7	13.1	2174.6	4094.1	3333.9
66	3.2	48	5975.6	12.9	2102.7	3873.0	3267.9
67	3.3	45	5686.1	12.7	2033.7	3652.4	3192.5
68	3.3	43	5399.8	12.6	1967.2	3432.5	3107.6
69	3.4	41	5116.4	12.4	1903.0	3213.4	3012.9
70	3.5	39	4835.6	12.2	1840.5	2995.1	2908.1



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 3548.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 31.5 pcf

Design Wall for an Equivalent Fluid Pressure: 32 pcf

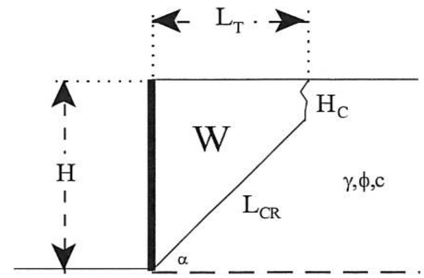
Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Cantilever Retaining Walls (Up to 20 feet)

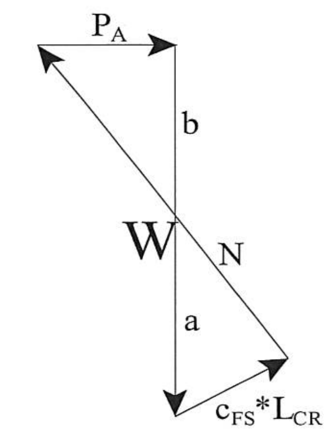
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	25.8 degrees
	(c_{FS})	116.7 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.6	193	24180.9	23.2	7412.1	16768.8	5825.2
46	3.5	187	23399.0	22.9	6985.5	16413.5	6024.7
47	3.4	181	22634.1	22.7	6598.3	16035.8	6205.8
48	3.3	175	21886.6	22.4	6245.8	15640.8	6369.0
49	3.3	169	21156.2	22.2	5923.9	15232.3	6514.8
50	3.2	164	20442.7	21.9	5629.2	14813.5	6643.9
51	3.1	158	19745.6	21.7	5358.7	14386.9	6756.5
52	3.1	153	19064.4	21.5	5109.8	13954.6	6853.3
53	3.1	147	18398.4	21.2	4880.2	13518.3	6934.4
54	3.0	142	17747.1	21.0	4667.8	13079.2	7000.2
55	3.0	137	17109.8	20.7	4471.1	12638.7	7050.8
56	3.0	132	16485.8	20.5	4288.4	12197.4	7086.6
57	3.0	127	15874.5	20.3	4118.3	11756.2	7107.5
58	3.0	122	15275.3	20.1	3959.7	11315.6	7113.7
59	3.0	118	14687.6	19.9	3811.5	10876.1	7105.2
60	3.0	113	14110.7	19.6	3672.7	10438.0	7082.0
61	3.0	108	13544.1	19.4	3542.5	10001.6	7043.9
62	3.0	104	12987.1	19.2	3419.9	9567.2	6990.9
63	3.1	100	12439.3	19.0	3304.5	9134.8	6922.7
64	3.1	95	11900.1	18.8	3195.3	8704.7	6839.2
65	3.1	91	11368.9	18.6	3092.0	8277.0	6740.0
66	3.2	87	10845.3	18.4	2993.8	7851.5	6624.8
67	3.3	83	10328.8	18.2	2900.3	7428.5	6493.1
68	3.3	79	9818.8	18.0	2810.9	7007.9	6344.6
69	3.4	75	9314.9	17.8	2725.1	6589.7	6178.7
70	3.5	71	8816.5	17.5	2642.5	6174.0	5994.8



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

$P_{A,\ max}$ 7113.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$
EFP 35.6 pcf

Design Wall for an Equivalent Fluid Pressure: 36 pcf

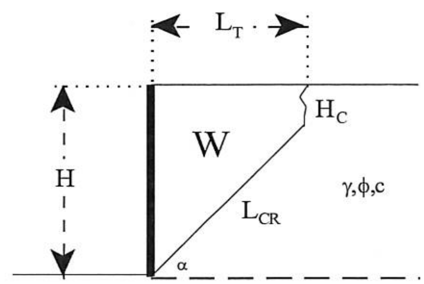
Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Cantilever Retaining Walls (Up to 25 feet)

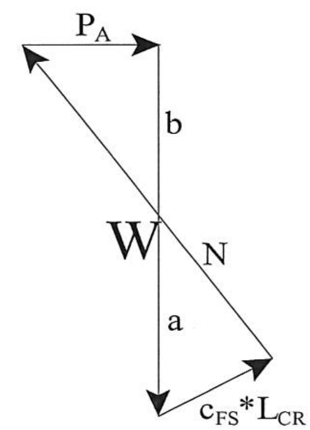
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	25.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	25.8 degrees
	(c_{FS})	116.7 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.6	306	38243.4	30.2	9674.7	28568.7	9924.2
46	3.5	296	36979.0	29.9	9103.5	27875.4	10232.0
47	3.4	286	35747.6	29.5	8587.2	27160.4	10511.0
48	3.3	276	34548.5	29.2	8119.0	26429.6	10762.2
49	3.3	267	33380.6	28.8	7692.8	25687.7	10986.6
50	3.2	258	32242.5	28.5	7303.9	24938.6	11185.0
51	3.1	249	31133.2	28.1	6947.9	24185.3	11358.2
52	3.1	240	30051.2	27.8	6621.1	23430.1	11506.8
53	3.1	232	28995.3	27.5	6320.4	22674.8	11631.4
54	3.0	224	27964.1	27.2	6043.0	21921.0	11732.4
55	3.0	216	26956.4	26.8	5786.6	21169.8	11810.2
56	3.0	208	25971.1	26.5	5548.9	20422.1	11865.1
57	3.0	200	25006.8	26.3	5328.3	19678.6	11897.2
58	3.0	193	24062.5	26.0	5122.9	18939.7	11906.7
59	3.0	185	23137.2	25.7	4931.4	18205.8	11893.7
60	3.0	178	22229.7	25.4	4752.4	17477.2	11858.0
61	3.0	171	21339.0	25.1	4584.9	16754.1	11799.6
62	3.0	164	20464.3	24.9	4427.7	16036.5	11718.2
63	3.1	157	19604.5	24.6	4280.0	15324.5	11613.5
64	3.1	150	18758.8	24.4	4140.8	14618.0	11485.2
65	3.1	143	17926.4	24.1	4009.3	13917.0	11332.8
66	3.2	137	17106.4	23.9	3884.9	13221.4	11155.7
67	3.3	130	16298.0	23.6	3766.9	12531.1	10953.2
68	3.3	124	15500.4	23.4	3654.5	11845.9	10724.7
69	3.4	118	14713.0	23.1	3547.3	11165.7	10469.2
70	3.5	111	13934.8	22.9	3444.5	10490.4	10185.8



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

$P_{A,\,max}$ 11906.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 38.1 pcf

Design Wall for an Equivalent Fluid Pressure: 39 pcf

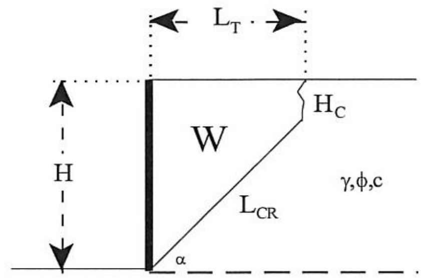
Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Cantilever Retaining Walls (Up to 28 feet)

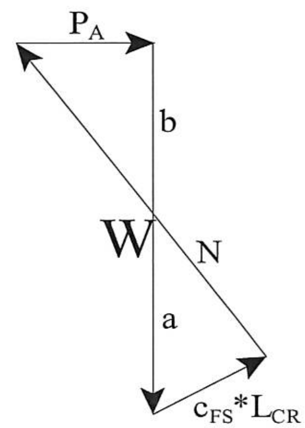
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	28.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	25.8 degrees
	(c_{FS})	116.7 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.6	385	48180.9	34.5	11032.2	37148.7	12904.7
46	3.5	373	46575.5	34.0	10374.3	36201.2	13288.0
47	3.4	360	45014.5	33.6	9780.6	35233.9	13635.4
48	3.3	348	43496.3	33.2	9242.9	34253.4	13948.0
49	3.3	336	42019.1	32.8	8754.2	33264.9	14227.3
50	3.2	325	40581.1	32.4	8308.7	32272.4	14474.2
51	3.1	313	39180.4	32.0	7901.4	31279.0	14689.6
52	3.1	303	37815.2	31.6	7527.9	30287.3	14874.5
53	3.1	292	36483.7	31.2	7184.6	29299.1	15029.4
54	3.0	281	35184.1	30.9	6868.2	28315.9	15155.0
55	3.0	271	33914.7	30.5	6575.9	27338.9	15251.8
56	3.0	261	32674.0	30.2	6305.3	26368.7	15320.0
57	3.0	252	31460.3	29.8	6054.2	25406.1	15360.0
58	3.0	242	30272.2	29.5	5820.8	24451.4	15371.8
59	3.0	233	29108.2	29.2	5603.3	23504.9	15355.6
60	3.0	224	27967.1	28.9	5400.3	22566.8	15311.2
61	3.0	215	26847.5	28.6	5210.4	21637.1	15238.5
62	3.0	206	25748.1	28.3	5032.4	20715.7	15137.3
63	3.1	197	24667.9	28.0	4865.3	19802.6	15007.2
64	3.1	189	23605.7	27.7	4708.1	18897.6	14847.6
65	3.1	180	22560.3	27.4	4559.8	18000.5	14658.0
66	3.2	172	21530.8	27.1	4419.6	17111.2	14437.7
67	3.3	164	20516.2	26.9	4286.9	16229.3	14185.8
68	3.3	156	19515.4	26.6	4160.7	15354.7	13901.4
69	3.4	148	18527.6	26.3	4040.6	14487.0	13583.4
70	3.5	140	17551.8	26.0	3925.7	13626.1	13230.6



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

$P_{A,\ max}$ 15371.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 39.2 pcf

Design Wall for an Equivalent Fluid Pressure: 40 pcf

Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC

File No.: 21827

Soil Weight	γ	125	pcf
Internal Friction Angle	ϕ	36	degrees
Cohesion	c	0	psf
Height of Retaining Wall	H	28	feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.412	
$\sigma'_v = \gamma H$	3500.0	psf

$\sigma'_h =$	1442.8 psf	
EFP =	51.5 pcf	
$P_o =$	20198.5 lbs/ft	(based on a triangular distribution of pressure)

Design wall for an EFP of 52 pcf

Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	28.0 feet
Retained Soil Unit Weight:	(γ)	125.0 pcf
Horizontal Ground Acceleration:	(k_h)	0.35 g

Seismic Increment (ΔP_{AE}):

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.75*k_h)$

$\Delta P_{AE} =$ 12972.8 lbs/ft

Transfer load to 1/3 of the height of the wall

$T*(2/3)*H = \Delta P_{AE}*0.6*H$

$T =$ 11675.5 lbs/ft

$EFP = 2*T/H^2$

EFP = 29.8 pcf

triangular distribution of pressure applied to the proposed retaining wall.

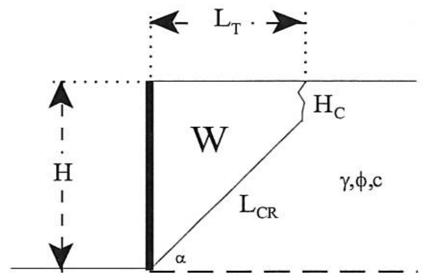
Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Temporary Shoring Walls (Up to 15 feet)

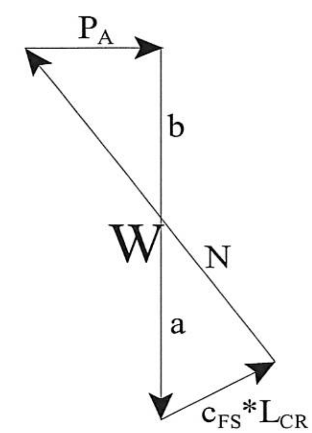
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	30.2 degrees
	(c_{FS})	140.0 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
45	5.3	98	12274.2	13.6	6452.9	5821.3	1541.7
46	5.1	96	12004.6	13.8	6099.9	5904.7	1674.6
47	4.9	94	11712.5	13.8	5770.5	5942.0	1797.8
48	4.7	91	11405.4	13.8	5464.4	5940.9	1911.2
49	4.6	89	11088.6	13.8	5180.7	5907.9	2015.1
50	4.4	86	10766.0	13.8	4917.8	5848.2	2109.3
51	4.3	84	10440.3	13.7	4674.3	5766.0	2194.2
52	4.2	81	10113.5	13.7	4448.5	5665.0	2269.7
53	4.1	78	9787.2	13.6	4239.0	5548.1	2336.0
54	4.1	76	9462.2	13.5	4044.3	5417.9	2393.4
55	4.0	73	9139.5	13.4	3863.2	5276.3	2441.7
56	4.0	71	8819.6	13.3	3694.3	5125.2	2481.3
57	3.9	68	8502.6	13.2	3536.6	4966.1	2512.2
58	3.9	66	8189.0	13.1	3389.0	4800.1	2534.4
59	3.9	63	7878.9	13.0	3250.6	4628.3	2547.9
60	3.9	61	7572.1	12.8	3120.5	4451.7	2552.9
61	3.9	58	7268.9	12.7	2997.9	4270.9	2549.4
62	3.9	56	6969.0	12.6	2882.2	4086.8	2537.2
63	3.9	53	6672.4	12.4	2772.6	3899.8	2516.5
64	4.0	51	6379.0	12.3	2668.5	3710.5	2487.1
65	4.0	49	6088.5	12.1	2569.3	3519.2	2449.0
66	4.1	46	5800.9	12.0	2474.4	3326.4	2402.0
67	4.1	44	5515.8	11.8	2383.4	3132.4	2346.2
68	4.2	42	5233.2	11.6	2295.6	2937.6	2281.4
69	4.3	40	4952.6	11.5	2210.5	2742.1	2207.4
70	4.4	37	4673.9	11.3	2127.5	2546.4	2124.1



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 2552.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 22.7 pcf

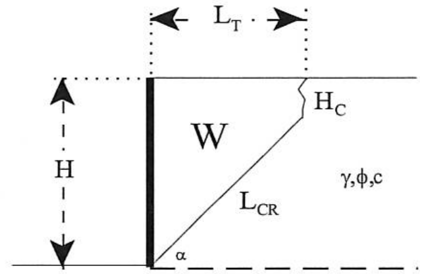
Design Shoring for an Equivalent Fluid Pressure: 25 pcf

Geotechnologies, Inc.

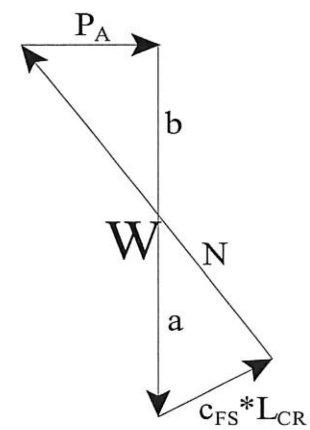
Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Temporary Shoring Walls (Up to 20 feet)

Shoring Design with Level Backfill
(Vector Analysis)

Input:		
Shoring Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	30.2 degrees
	(c_{FS})	140.0 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
45	5.3	186	23211.7	20.7	9796.1	13415.6	3552.9
46	5.1	181	22566.8	20.7	9183.5	13383.3	3795.5
47	4.9	175	21911.9	20.6	8628.0	13283.9	4019.1
48	4.7	170	21253.6	20.6	8123.6	13129.9	4224.0
49	4.6	165	20596.4	20.4	7664.7	12931.7	4410.7
50	4.4	160	19943.6	20.3	7246.3	12697.3	4579.7
51	4.3	154	19297.3	20.2	6863.9	12433.4	4731.3
52	4.2	149	18658.8	20.0	6513.5	12145.3	4866.0
53	4.1	144	18029.2	19.9	6191.8	11837.3	4984.1
54	4.1	139	17408.8	19.7	5895.6	11513.2	5085.9
55	4.0	134	16798.1	19.5	5622.3	11175.7	5171.8
56	4.0	130	16197.0	19.3	5369.6	10827.4	5242.0
57	3.9	125	15605.5	19.2	5135.2	10470.3	5296.6
58	3.9	120	15023.6	19.0	4917.4	10106.1	5335.9
59	3.9	116	14450.8	18.8	4714.6	9736.2	5359.9
60	3.9	111	13886.9	18.6	4525.2	9361.7	5368.7
61	3.9	107	13331.6	18.4	4348.0	8983.7	5362.4
62	3.9	102	12784.6	18.2	4181.7	8602.9	5340.9
63	3.9	98	12245.3	18.0	4025.3	8220.0	5304.2
64	4.0	94	11713.5	17.8	3877.8	7835.7	5252.1
65	4.0	90	11188.7	17.6	3738.4	7450.4	5184.6
66	4.1	85	10670.5	17.4	3606.0	7064.5	5101.3
67	4.1	81	10158.5	17.2	3480.1	6678.4	5002.2
68	4.2	77	9652.2	17.0	3359.7	6292.4	4886.8
69	4.3	73	9151.1	16.8	3244.3	5906.9	4754.9
70	4.4	69	8654.9	16.6	3133.0	5521.9	4606.1



Design Equations (Vector Analysis):

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

Maximum Active Pressure Resultant

$P_{A,\max}$ 5368.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 26.8 pcf

Design Shoring for an Equivalent Fluid Pressure: 27 pcf

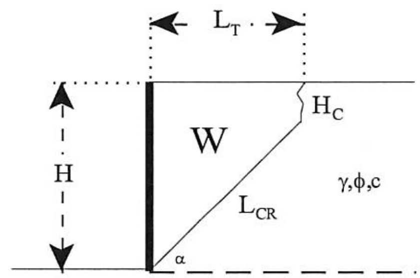
Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Temporary Shoring Walls (Up to 25 feet)

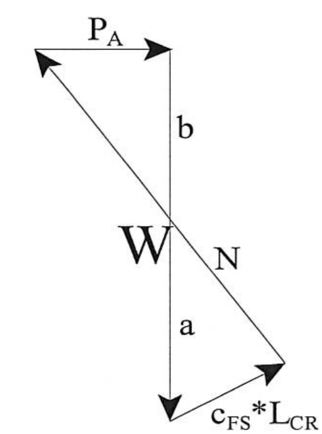
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	25.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	30.2 degrees
	(c_{FS})	140.0 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
45	5.3	298	37274.2	27.8	13139.2	24135.0	6391.8
46	5.1	289	36146.8	27.7	12267.0	23879.8	6772.3
47	4.9	280	35025.4	27.5	11485.5	23539.9	7122.1
48	4.7	271	33915.5	27.3	10782.8	23132.7	7442.0
49	4.6	263	32820.8	27.1	10148.8	22672.0	7732.9
50	4.4	254	31743.5	26.8	9574.8	22168.7	7995.8
51	4.3	245	30684.9	26.6	9053.5	21631.4	8231.4
52	4.2	237	29645.7	26.4	8578.6	21067.1	8440.5
53	4.1	229	28626.0	26.1	8144.6	20481.4	8623.6
54	4.1	221	27625.8	25.9	7746.9	19878.9	8781.5
55	4.0	213	26644.7	25.6	7381.5	19263.2	8914.5
56	4.0	205	25682.3	25.4	7044.8	18637.5	9023.1
57	3.9	198	24737.8	25.1	6733.8	18004.0	9107.7
58	3.9	190	23810.8	24.9	6445.9	17364.9	9168.4
59	3.9	183	22900.4	24.6	6178.6	16721.8	9205.6
60	3.9	176	22005.9	24.4	5929.9	16076.0	9219.2
61	3.9	169	21126.6	24.1	5698.0	15428.6	9209.5
62	3.9	162	20261.7	23.9	5481.2	14780.5	9176.2
63	3.9	155	19410.5	23.6	5278.1	14132.5	9119.4
64	4.0	149	18572.3	23.4	5087.2	13485.1	9038.9
65	4.0	142	17746.2	23.2	4907.4	12838.8	8934.3
66	4.1	135	16931.6	22.9	4737.6	12193.9	8805.3
67	4.1	129	16127.7	22.7	4576.8	11550.9	8651.7
68	4.2	123	15333.8	22.4	4423.9	10909.9	8472.8
69	4.3	116	14549.2	22.2	4278.1	10271.1	8268.1
70	4.4	110	13773.2	21.9	4138.4	9634.8	8036.9



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

$P_{A,\ max}$ 9219.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 29.5 pcf

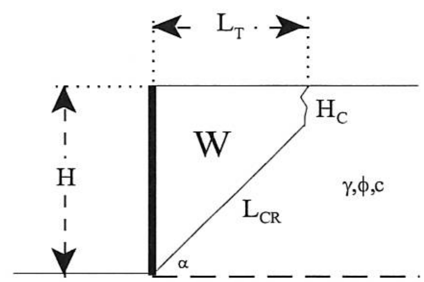
Design Shoring for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

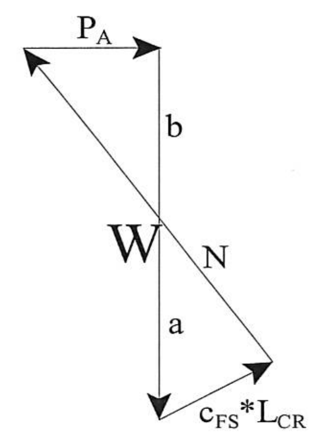
Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Temporary Shoring Walls (Up to 30 feet)

Shoring Design with Level Backfill
(Vector Analysis)

Input:		
Shoring Height	(H)	30.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	30.2 degrees
	(c_{FS})	140.0 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
45	5.3	436	54461.7	34.9	16482.4	37979.3	10058.3
46	5.1	422	52744.6	34.6	15350.6	37394.0	10605.0
47	4.9	408	51053.0	34.3	14343.0	36710.0	11106.7
48	4.7	395	49391.2	34.0	13442.0	35949.2	11565.1
49	4.6	382	47761.6	33.7	12632.8	35128.8	11981.7
50	4.4	369	46165.5	33.4	11903.3	34262.2	12357.7
51	4.3	357	44603.0	33.0	11243.1	33359.9	12694.5
52	4.2	345	43074.0	32.7	10643.6	32430.4	12993.2
53	4.1	333	41577.7	32.4	10097.4	31480.3	13254.7
54	4.1	321	40113.3	32.0	9598.2	30515.1	13480.0
55	4.0	309	38679.5	31.7	9140.6	29538.9	13669.8
56	4.0	298	37275.4	31.4	8720.1	28555.3	13824.7
57	3.9	287	35899.5	31.1	8332.5	27567.1	13945.3
58	3.9	276	34550.7	30.8	7974.3	26576.4	14031.9
59	3.9	266	33227.7	30.5	7642.6	25585.0	14084.9
60	3.9	255	31929.1	30.1	7334.7	24594.4	14104.4
61	3.9	245	30653.8	29.8	7048.1	23605.7	14090.5
62	3.9	235	29400.5	29.5	6780.7	22619.8	14043.1
63	3.9	225	28168.0	29.3	6530.8	21637.2	13962.1
64	4.0	216	26955.2	29.0	6296.6	20658.6	13847.2
65	4.0	206	25760.9	28.7	6076.5	19684.4	13698.0
66	4.1	197	24583.9	28.4	5869.2	18714.7	13514.1
67	4.1	187	23423.3	28.1	5673.5	17749.9	13294.7
68	4.2	178	22278.0	27.8	5488.1	16789.9	13039.3
69	4.3	169	21146.9	27.5	5311.9	15835.0	12746.9
70	4.4	160	20028.9	27.2	5143.8	14885.1	12416.5



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

$P_{A, max}$ 14104.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 31.3 pcf

Design Shoring for an Equivalent Fluid Pressure: 32 pcf

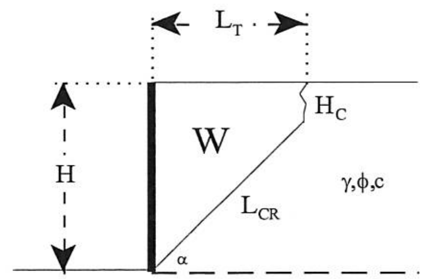
Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Temporary Shoring Walls (Up to 35 feet)

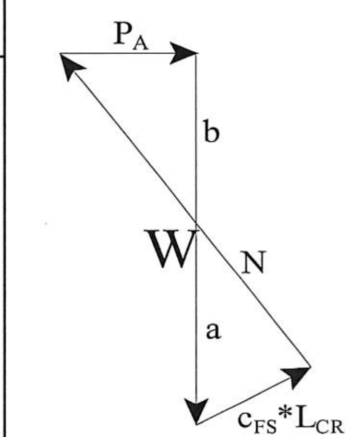
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	35.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	36.0 degrees
Cohesion of Retained Soils	(c)	175.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	30.2 degrees
	(c_{FS})	140.0 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
45	5.3	598	74774.2	41.9	19825.6	54948.7	14552.3
46	5.1	579	72360.2	41.6	18434.2	53926.0	15293.5
47	4.9	560	69994.7	41.2	17200.5	52794.2	15973.1
48	4.7	541	67680.6	40.7	16101.2	51579.5	16593.5
49	4.6	523	65419.0	40.3	15116.9	50302.1	17157.0
50	4.4	506	63209.7	39.9	14231.8	48977.9	17665.4
51	4.3	488	61051.8	39.5	13432.8	47619.0	18120.5
52	4.2	472	58943.9	39.0	12708.7	46235.2	18524.0
53	4.1	455	56884.3	38.6	12050.2	44834.1	18877.3
54	4.1	439	54871.2	38.2	11449.5	43421.7	19181.5
55	4.0	423	52902.5	37.8	10899.8	42002.7	19437.7
56	4.0	408	50976.3	37.4	10395.3	40581.0	19646.8
57	3.9	393	49090.6	37.0	9931.1	39159.5	19809.6
58	3.9	378	47243.4	36.7	9502.8	37740.6	19926.5
59	3.9	363	45432.7	36.3	9106.6	36326.0	19998.0
60	3.9	349	43656.5	35.9	8739.4	34917.2	20024.3
61	3.9	335	41913.2	35.6	8398.1	33515.1	20005.5
62	3.9	322	40200.8	35.2	8080.2	32120.6	19941.5
63	3.9	308	38517.7	34.9	7783.5	30734.2	19832.2
64	4.0	295	36862.2	34.5	7505.9	29356.3	19677.1
65	4.0	282	35232.7	34.2	7245.6	27987.2	19475.8
66	4.1	269	33627.6	33.9	7000.8	26626.8	19227.5
67	4.1	256	32045.5	33.5	6770.2	25275.3	18931.3
68	4.2	244	30484.8	33.2	6552.2	23932.5	18586.3
69	4.3	232	28944.1	32.9	6345.7	22598.4	18191.3
70	4.4	219	27422.1	32.5	6149.3	21272.8	17744.9



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

$P_{A,\,max}$ 20024.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 32.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 33 pcf

Geotechnologies, Inc.

Tiebacks Calculations (Ref: Bowles, 1982)

Project: Legendary East Pasadena, LLC

File No. 21827

Soil Parameters:

Weight of Soil	γ	125.00	*lbs/ft*^{3}
Friction Angle	ϕ	36.00	*degrees*
Cohesion	c	175.00	*lbs/ft*^{2}
Tieback Angle	α	40.00	*degrees*

Design Assumptions:

Diameter of Grout	d	0.50	*feet*
Length of Embeddment	L	20.00	*feet*
Depth to midpoint of Embeddment	h	10.00	*feet*
Earth Pressure Coefficient	K	0.50	
Factor of Safety Applied	F.S.	1.50	

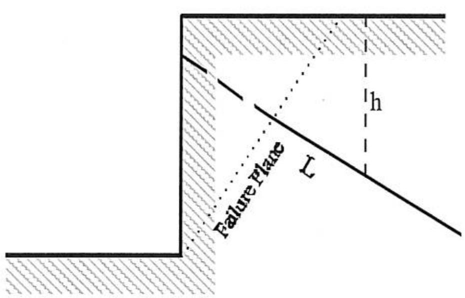
Ultimate Resistance: R_{ult} 25.70 *kips*

Eq: pi*d*γ*L*h*cos(a)*tan(ϕ)+c*pi*d*L

Allowable Resistance: $R_{allow} = R_{ult}/F.S.$ 17.14 *kips*

Allowable Skin Friction: $R_{allow}/2/pi/r/L$ 545.47 *psf*

Allowable Skin Friction Design Value **525** ***psf***



Geotechnologies, Inc.

Tiebacks Calculations (Ref: Bowles, 1982)

Project: Legendary East Pasadena, LLC

File No. 21827

Soil Parameters:

Weight of Soil	γ	125.00	*lbs/ft*^{3}
Friction Angle	ϕ	36.00	*degrees*
Cohesion	c	175.00	*lbs/ft*^{2}
Tieback Angle	α	20.00	*degrees*

Design Assumptions:

Diameter of Grout	d	0.50	*feet*
Length of Embeddment	L	20.00	*feet*
Depth to midpoint of Embeddment	h	10.00	*feet*
Earth Pressure Coefficient	K	0.50	
Factor of Safety Applied	F.S.	1.50	

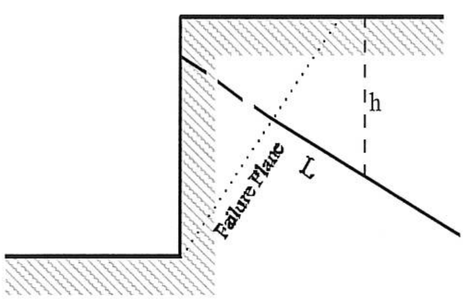
Ultimate Resistance: R_{ult} 30.66 *kips*

Eq: pi*d*γ*L*h*cos(a)*tan(ϕ)+c*pi*d*L

Allowable Resistance: $R_{allow} = R_{ult}/F.S.$ 20.44 *kips*

Allowable Skin Friction: $R_{allow}/2/pi/r/L$ 650.61 *psf*

Allowable Skin Friction Design Value **525 *psf***



Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827
Description: Slot Cut

Slot Cut Calculation

Input:

Height of Slots	(H)	5.0	feet
Unit Weight of Soils	(γ)	125.0	pcf
Friction Angle of Soils	(ϕ)	36.0	degrees
Cohesion of Soils	(c)	175.0	psf
Factor of Safety	(FS)	1.50	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.5	

Surcharge Pressure:

Line Load	(q_L)	900.0	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	2.9	7	902.1	780.3	832.5	1809.1	10.7
61	2.8	7	866.1	748.9	786.6	1736.9	10.3
62	2.7	7	830.8	717.4	742.4	1666.1	10.0
63	2.5	6	796.1	686.1	699.8	1596.6	9.7
64	2.4	6	762.1	654.9	658.8	1528.3	9.4
65	2.3	6	728.6	623.8	619.4	1461.2	9.2
66	2.2	6	695.7	592.9	581.4	1395.1	9.1
67	2.1	5	663.2	562.3	544.8	1330.1	8.9
68	2.0	5	631.3	531.9	509.6	1266.0	8.8
69	1.9	5	599.8	501.8	475.8	1202.8	8.7
70	1.8	5	568.7	472.0	443.3	1140.5	8.6
71	1.7	4	538.0	442.7	412.0	1079.0	8.6
72	1.6	4	507.7	413.7	382.0	1018.1	8.5
73	1.5	4	477.7	385.2	353.1	958.0	8.5
74	1.4	4	448.0	357.2	325.3	898.5	8.5
75	1.3	3	418.7	329.7	298.6	839.6	8.6
76	1.2	3	389.6	302.7	273.0	781.3	8.6
77	1.2	3	360.7	276.3	248.4	723.4	8.7
78	1.1	3	332.1	250.6	224.7	666.1	8.8
79	1.0	2	303.7	225.5	201.9	609.1	8.9
80	0.9	2	275.5	201.0	180.0	552.5	9.1
81	0.8	2	247.5	177.3	159.0	496.3	9.3
82	0.7	2	219.6	154.3	138.7	440.4	9.5
83	0.6	2	191.9	132.1	119.2	384.7	9.8
84	0.5	1	164.2	110.6	100.4	329.3	10.1
85	0.4	1	136.7	90.0	82.3	274.1	10.4

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

d_{allow} 8.5 feet

The proposed excavation may be made using the A-B-C Slot-Cutting Method with a Maximum Allowable Slot Width of 8 Feet, and up to 5 Feet in Height, with a Factor of Safety Equal or Exceeding 1.5.

Geotechnologies, Inc.

Project: Legendary East Pasadena, LLC
File No.: 21827

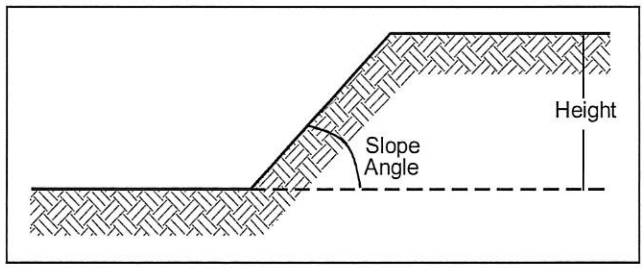
Slope Stability Calculations

Input

Soil Density	(γ)	125 pcf
Friction Angle	(ϕ)	36 degrees
Cohesion	(c)	175 psf
Factor of Safety	(FS)	1.5

Stability Number (N)

(ϕ_d)	25.8 degrees
$N_{(2:1)}$	0.002
$N_{(1.5:1)}$	0.017
$N_{(1:1)}$	0.045
$N_{(3/4:1)}$	0.063
$N_{(1:1.5)}$	0.070
$N_{(1:2)}$	0.085
$N_{(vertical)}$	0.163



Reference: Taylor's Chart (1937)

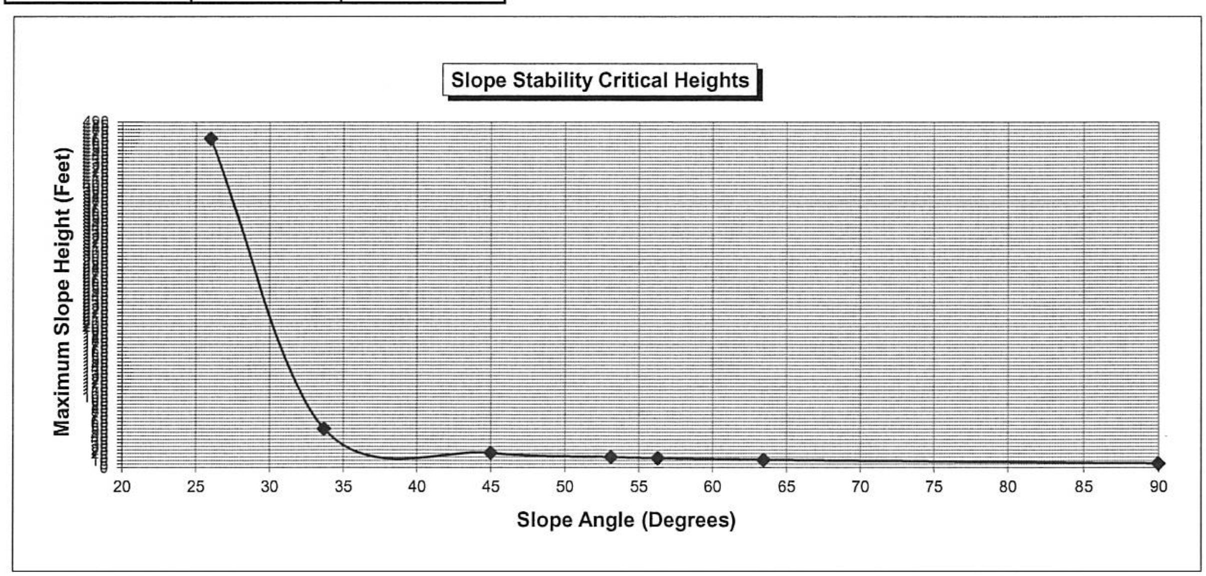
$(\phi_\delta) = \text{ArcTan}[(\tan\phi)/FS]$

$N = \dfrac{c}{(\gamma)(H)(FS)}$

$H = \dfrac{c}{(\gamma)(N)(FS)}$

Assumptions: Slope is uniform, soils are homogeneous, no water seepage, no surcharge loads.

Slope Angle (h:v)	Slope Angle (Degrees)	Maximum Height (Feet)
2 : 1	26.00	467
$1\frac{1}{2} : 1$	33.69	55
1 : 1	45.00	21
$\frac{3}{4} : 1$	53.13	15
$1 : 1\frac{1}{2}$	56.30	13
$\frac{1}{2} : 1$	63.43	11
Vertical	90.00	6

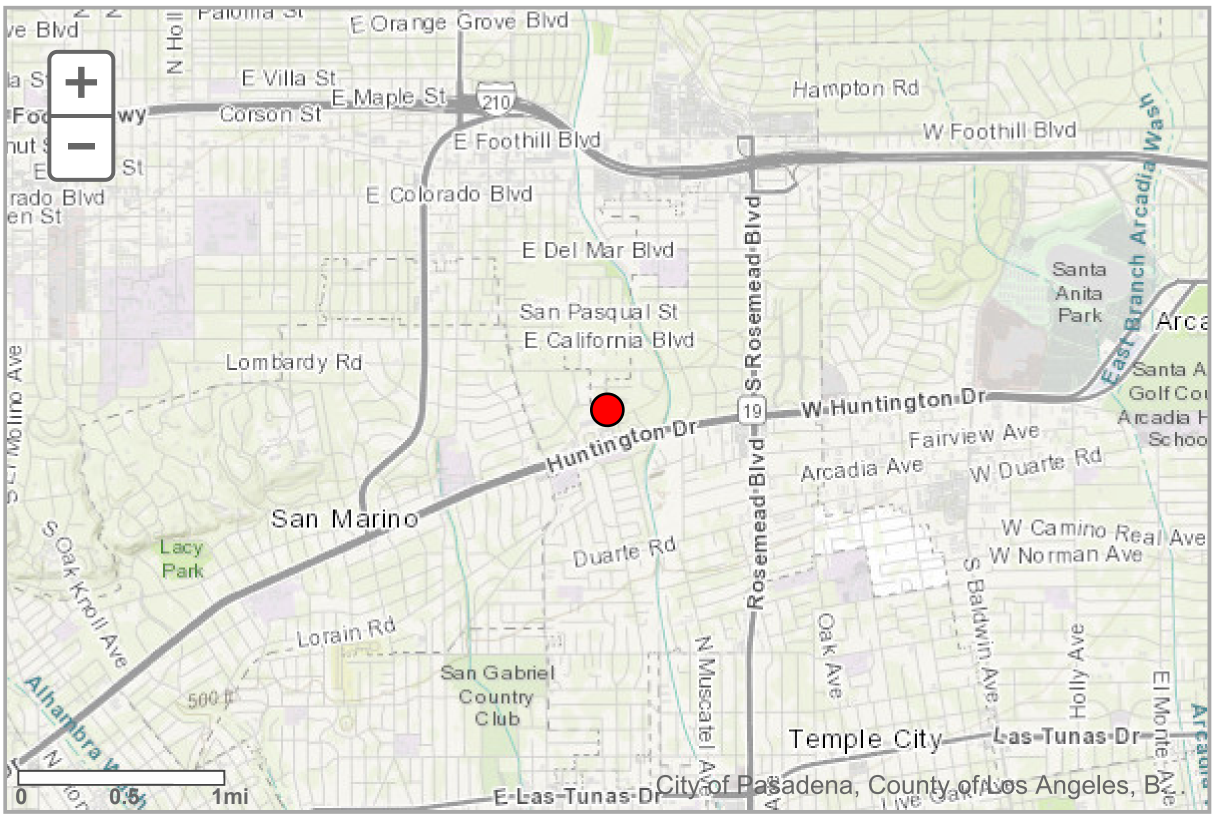


Groundwater Levels for Station 341308N1180857W002

Data for your selected well is shown in the tabbed interface below. To view data managed in the updated WDL tables, including data collected under the CASGEM program, click the "Recent Groundwater Level Data" tab. To view data stored in the former WDL tables, click the "Historical Groundwater Level Data" tab. To download the data in CSV format, click the "Download CSV File" button on the respective tab. Please note that the vertical datum for "recent" measurements is NAVD88, while the vertical datum for "historical" measurements is NGVD29. To change your well selection criteria, click the "Perform a New Well Search" button.

State Well Number:	01N12W36H003S
Local Well ID:	No. 11
Site Code:	341308N1180857W002
Latitude (NAD83):	34.130778
Longitude (NAD83):	-118.085722
Groundwater Basin (code):	Raymond (4-023)

Well Use:	Other
Well Status:	Active
Well Completion Report Number:	
Reference Point Elevation (NAVD88 ft):	603.000
Ground Surface Elevation (NAVD88 ft):	606.000
Total Depth (ft):	800
Perforated Interval Depths (ft):	300.000 780.000



Perform a New Well Search

Groundwater Levels for Station 341308N1180857W002

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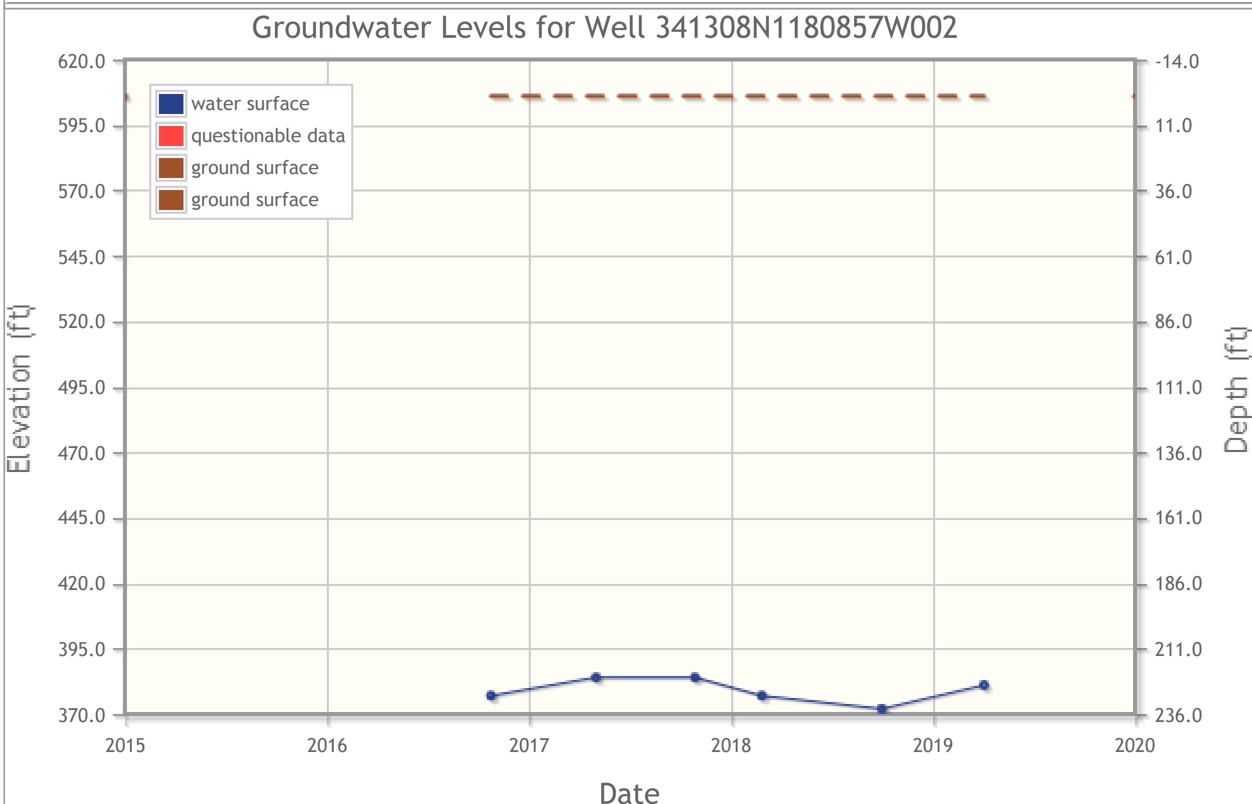
Station Data | **Recent Groundwater Level Data** | Historical Groundwater Level Data



Download CSV File

Date	RPE	GSE	RPWS	WSE	GS to WS	Msmt Code	CASGEM Msmt	Agency	Co
10/23/2016 00:00	603.000	606.000	226	377	229		N	5094	
04/30/2017 00:00	603.000	606.000	219	384	222		N	5094	
10/26/2017 00:00	603.000	606.000	219	384	222		N	5094	
02/25/2018 00:00	603.000	606.000	226	377	229		N	5094	
09/29/2018 00:00	603.000	606.000	231	372	234		N	5094	
03/31/2019 00:00	603.000	606.000	222	381	225		N	5094	

All elevation and depth measurements are in feet. The vertical datum for recent measurements is NAVD88.

Perform a New Well Search

Soil Corrosivity Evaluation Report for Legendary East Pasadena

July 3, 2019

Prepared for:
Scott Prince
Geotechnologies, Inc
439 Western Ave.
Glendale, CA, 91201
sprince@geoteq.com

Project X Job #: S190701F
Client Job or PO #: 21827

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1 Executive Summary

A corrosion evaluation of the soils at Legendary East Pasadena was performed to provide corrosion control recommendations for general construction materials. The site is located at 380 S Rosemead Blvd Pasadena, CA 91107. Three (3) samples were tested to a depth of 25.0 ft. Site ground water and topography information was provided via Geotechnologies and determined to be 28 feet below finished grade.

Every material has its weakness. Aluminums, galvanized/zinc coatings, and coppers 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 over-simplify descriptions as corrosive or non-corrosive, will not detect these other factors 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 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 18,760 ohm-cm and 716,900 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 along infrastructure surfaces which is why minimum or saturated soil resistivity measurements are more important than as-received resistivities.

Saturated soil resistivities ranged between 10,720 ohm-cm to 18,760 ohm-cm. The worst of these values is considered to be mildly corrosive to general metals.

PH levels ranged between 7.7 to 8.7 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 7 mg/kg to 18 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals.

Sulfates ranged between 16 mg/kg to 89 mg/kg. Sulfate levels in these samples are negligible for corrosion of metals and cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 0.6 mg/kg to 2.0 mg/kg. Nitrates ranged between 0.6 mg/kg to 322.5 mg/kg. Concentrations of these elements at shallow depths were high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 156 mV to + 181 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 89 mg/kg or 0.0089 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 18 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.

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.

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

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).[3,4]

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 significant.[5] 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

---

[3] *Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.*
[4] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

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

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.[6] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[7]

Expected Corrosion Rate for Steel = 0.79 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.08 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

- 66.0 Years to Perforation for a 18 gage metal culvert
- 85.8 Years to Perforation for a 16 gage metal culvert
- 105.6 Years to Perforation for a 14 gage metal culvert
- 145.3 Years to Perforation for a 12 gage metal culvert
- 184.9 Years to Perforation for a 10 gage metal culvert
- 224.5 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 = 0.08 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.

[6] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962
[7] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

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. **These are especially important for fire risers.**

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 8 mil thick polyethylene, or
2) Tape coating system, or
3) Wax tape, or
4) Coal tar enamel, or
5) Fusion bonded epoxy, or
6) Apply 3 inch 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.

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 8 mil thick polyethylene, or
2) Tape coating system, or
3) Wax tape, or
4) Coal tar enamel, or
5) Fusion bonded epoxy, or
6) Galvanized steel, or
7) Apply 3 inch 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.

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) 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 4 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials.

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 8 mil thick polyethylene, or

2) Tape coating system, or
3) Wax tape, or
4) Coal tar enamel, or
5) Fusion bonded epoxy, or
6) Apply 3 inch 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.

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 Pipe

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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.

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
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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 nobler than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Copper when cold has one native potential, but when heated develops a more electronegative electro-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 7.7. Copper is greatly affected by pH, ammonia and nitrate concentrations[8]. The highest nitrate concentration was 322.5 mg/kg and the highest ammonia concentration was 7.1 mg/kg at this site.

The shallow soils were determined to be corrosive to copper and copper alloys such as brass.

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

1) Run copper pipes within PVC pipes to prevent soil contact, or
2) Cover piping with a 20 mil epoxy coating free of scratches and defects, or
3) Cover copper pipes with minimum 10 mil polyethylene sleeve over a suitable primer and apply cathodic protection per NACE SP0169

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.

The shallow soils were determined to be corrosive to copper and copper alloys such as brass.

The following are corrosion control options for underground brass.

1) Prevent soil contact by use of impermeable coating system such as wax tape, or
2) Prevent soil contact by use of a 20 mil epoxy coating free of scratches and defects, or
3) Cover brass with minimum 10 mil polyethylene sleeve over a suitable primer and apply cathodic protection per NACE SP0169

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

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

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:[9]

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
14	64.1	5.5
13	72	6.2
12	80.8	7.0
11	90.7	7.8
10	101.9	8.8
9	114.4	9.9
8	128.5	11.1
7	144.3	12.4
6	162	14.0
5	181.9	15.7
4	204.3	17.6
3	229.4	19.8
2	257.6	22.2
1	289.3	24.9

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.

It is recommended that a corrosion inhibiting and water-repelling coating such as Corrosion X Part No. 90102 by Corrosion Technologies (no affiliation to Project X) be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion.

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.

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

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. 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,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
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4 SOIL ANALYSIS LAB RESULTS

Client: Geotechnologies, Inc
Job Name: Legendary East Pasadena
Client Job Number: 21827
Project X Job Number: S190701F
July 3, 2019

	Method	ASTM G187		ASTM D4327		ASTM D4327		ASTM D4327	ASTM D4327	SM 4500-S2-D	ASTM G200	ASTM G51
Bore# / Description	Depth	Resistivity		Sulfates		Chlorides		Nitrate	Ammonia	Sulfide	Redox	pH
		As Rec'd	Minimum									
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(mg/kg)	(mg/kg)	(mg/kg)	(mV)	
B1	22.5	18,760	10,720	17.6	0.0018	6.8	0.0007	3.8	0.6	1.14	156	8.06
B2	25.0	73,700	16,080	15.7	0.0016	8.4	0.0008	0.6	1.7	0.24	179	8.65
B2	1.0-5.0	716,900	18,760	89.0	0.0089	18.3	0.0018	322.5	2.0	0.48	181	7.74

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.

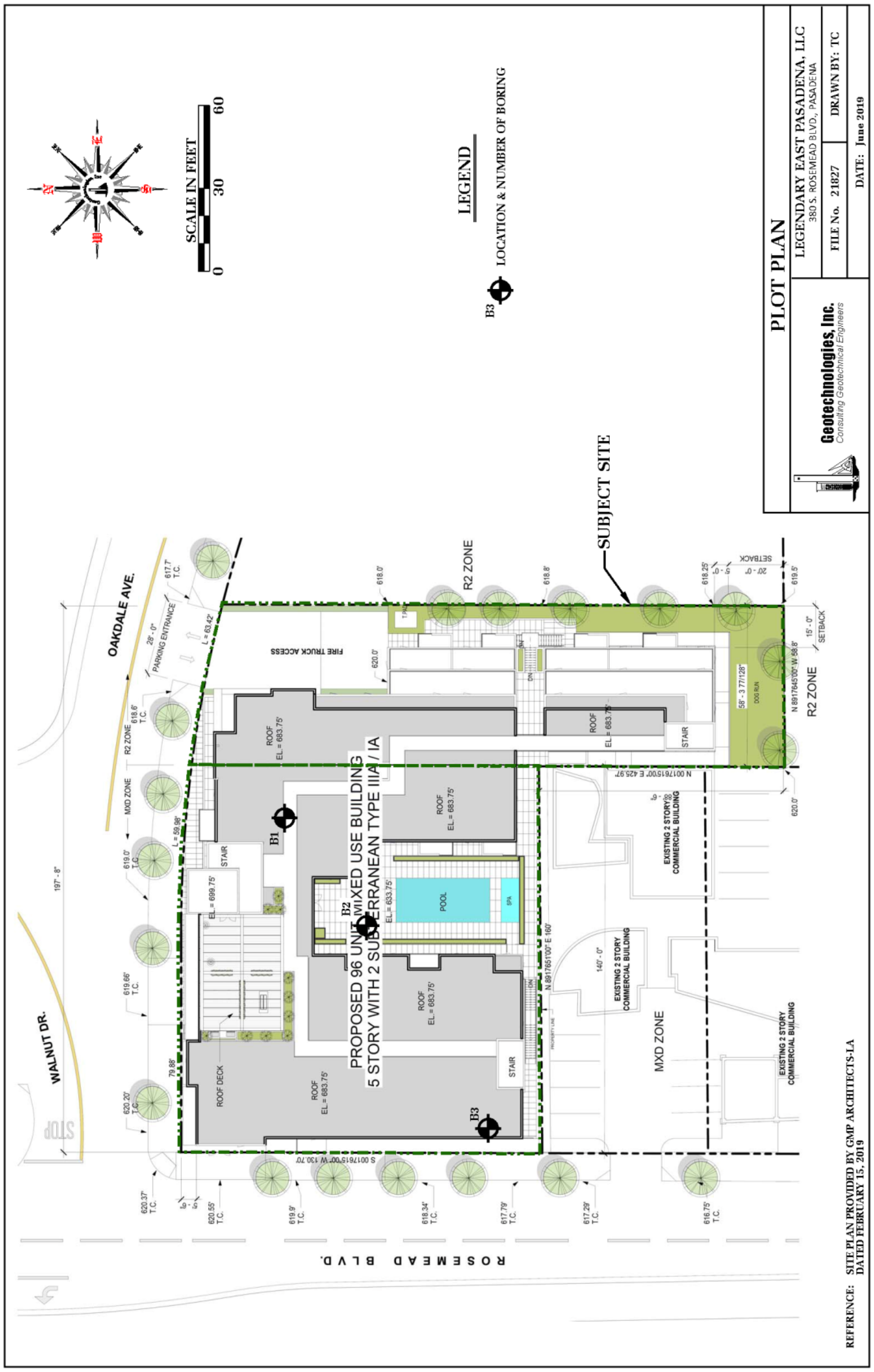


Figure 1 Soil Sample Locations, 380 S Rosemead Blvd Pasadena, CA 91107

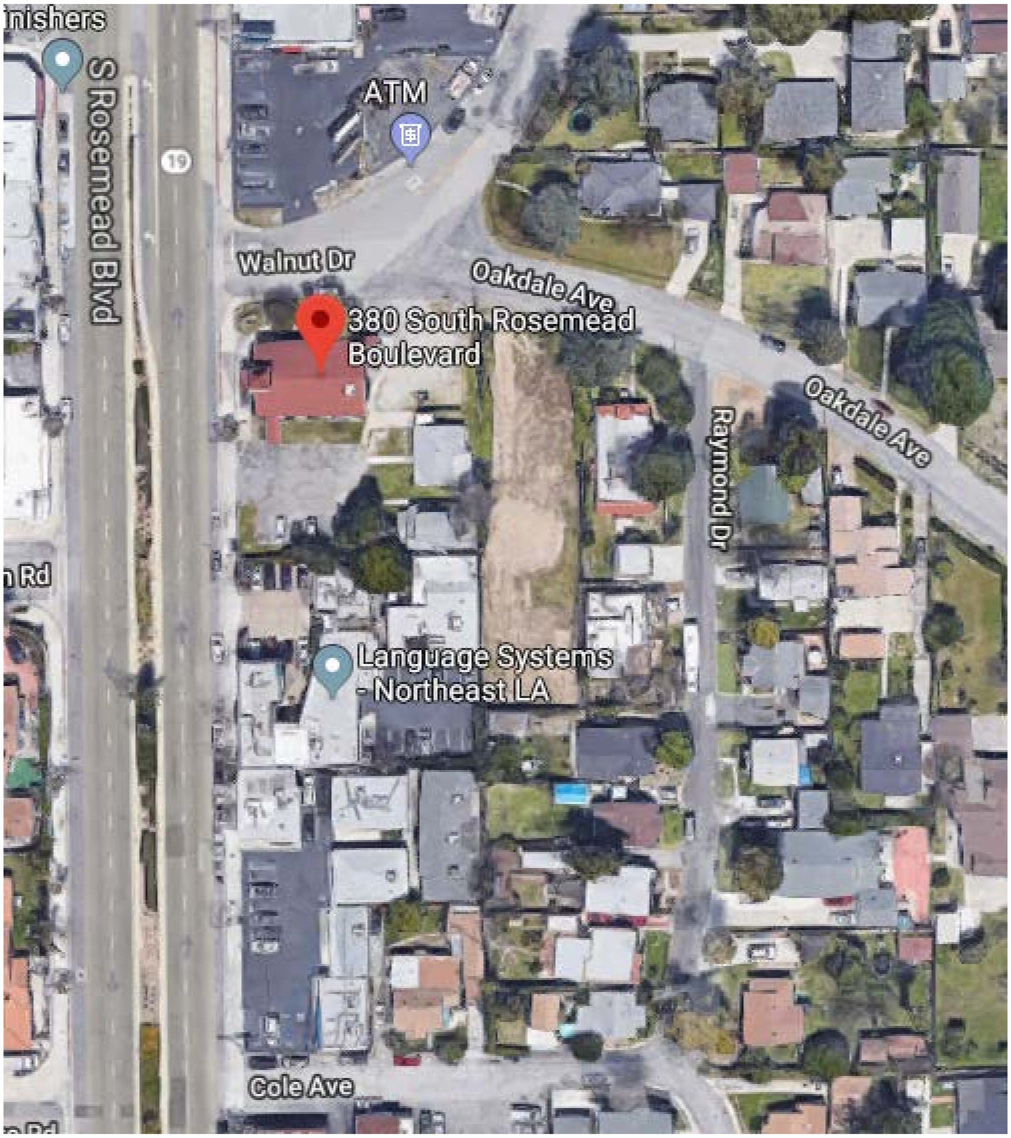


Figure 2 Satellite view of site, 380 S Rosemead Blvd Pasadena, CA 91107

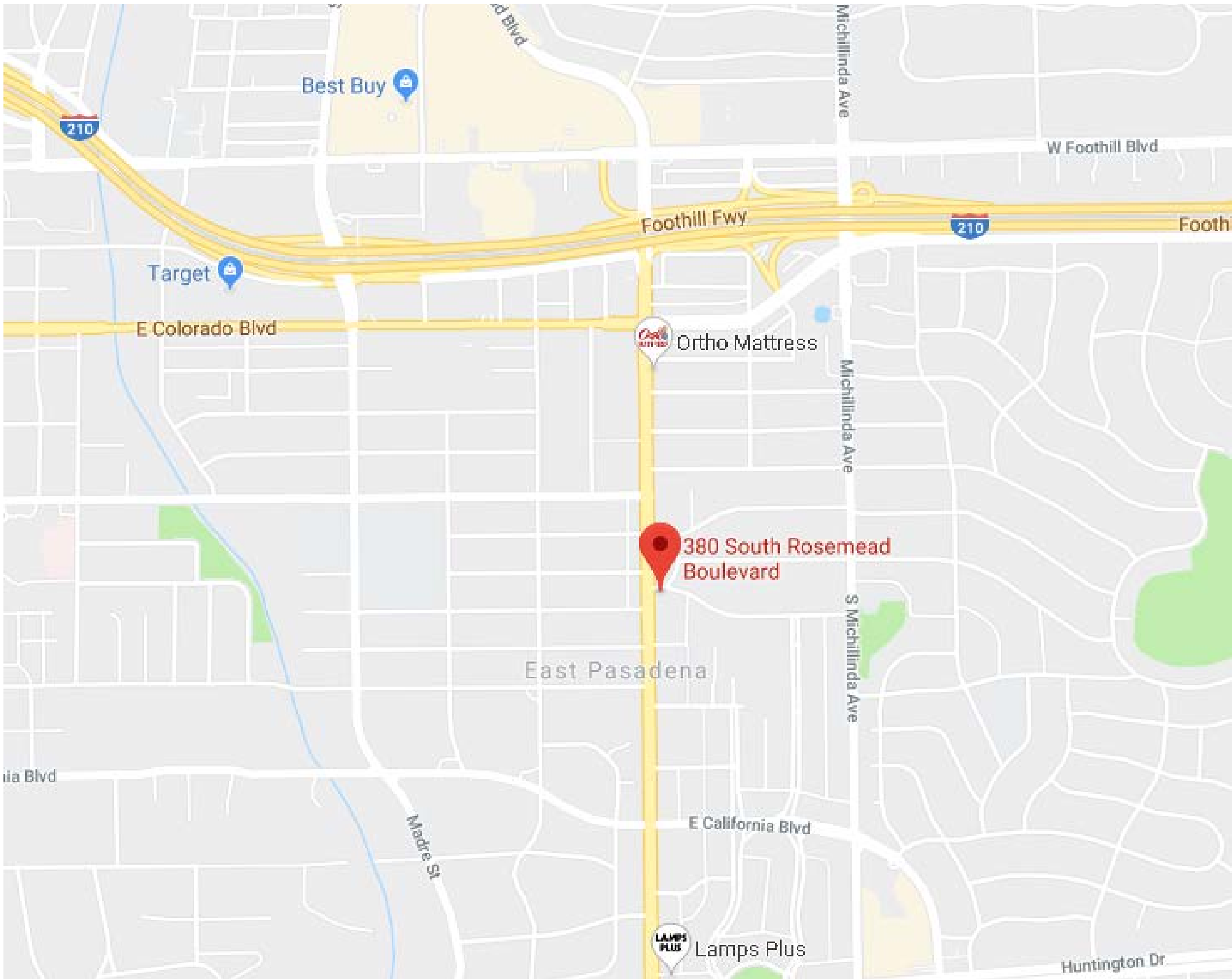


Figure 3 Vicinity Map, 380 S Rosemead Blvd Pasadena, CA 91107

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. Oxygen content in soil can be increased during construction. These soils are considered disturbed soils. When construction equipment at a site is simply driving piles into soil without digging into the soil, the activity can still disturb soil down to 3 feet. Expansive soils will also 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 dependant 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

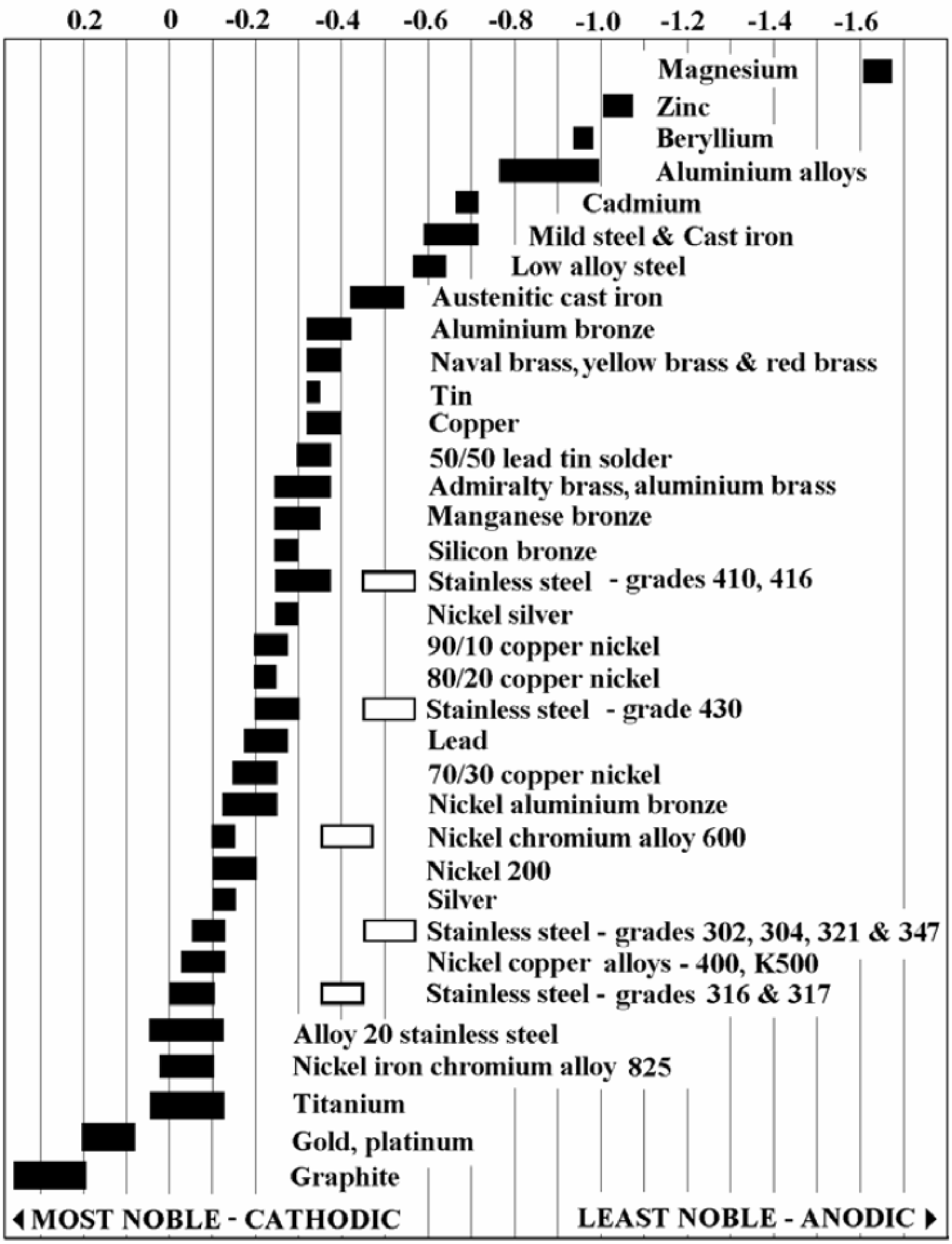
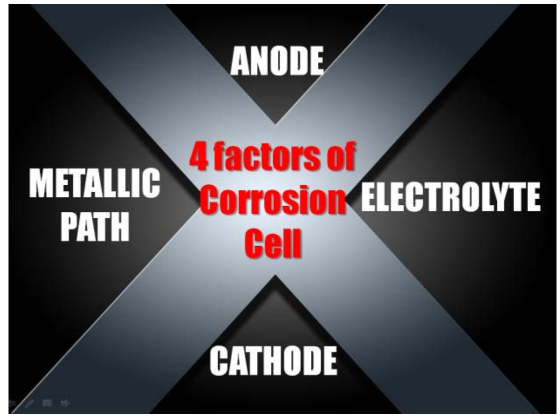
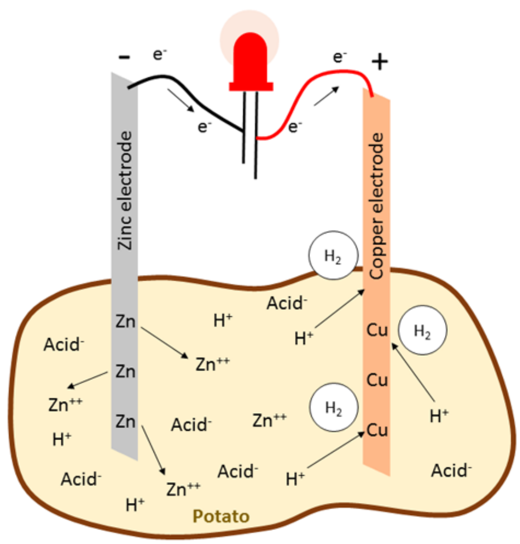


Figure 4 - Galvanic series of metals relative to CSE half cell.

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.

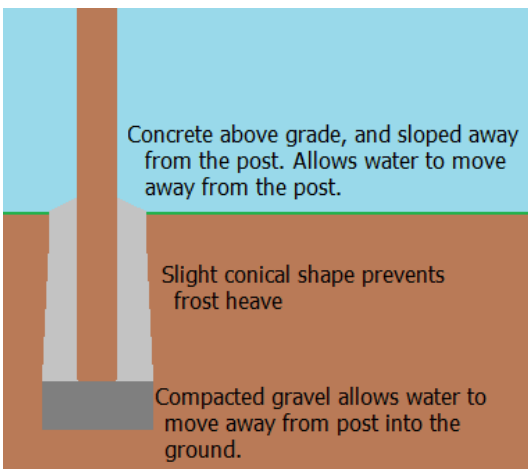
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction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

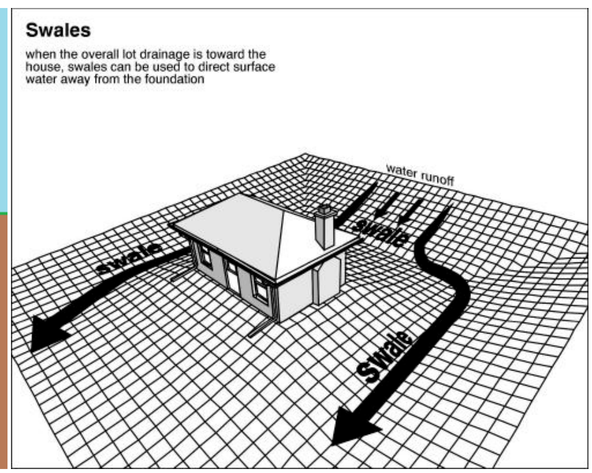
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and 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. [10,11]





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**

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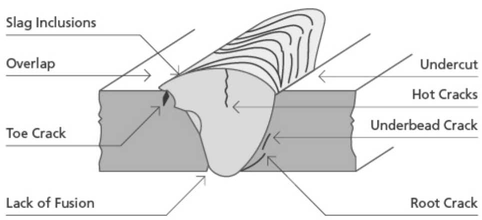
[10] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

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

are like infections, they should not be left to fester or the infection will spread making things worse.



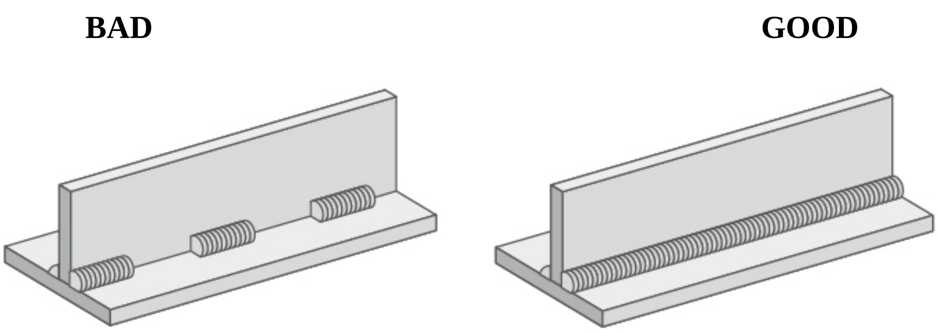


Figure 5 Defects which form weld crevices[12]

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

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

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 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.

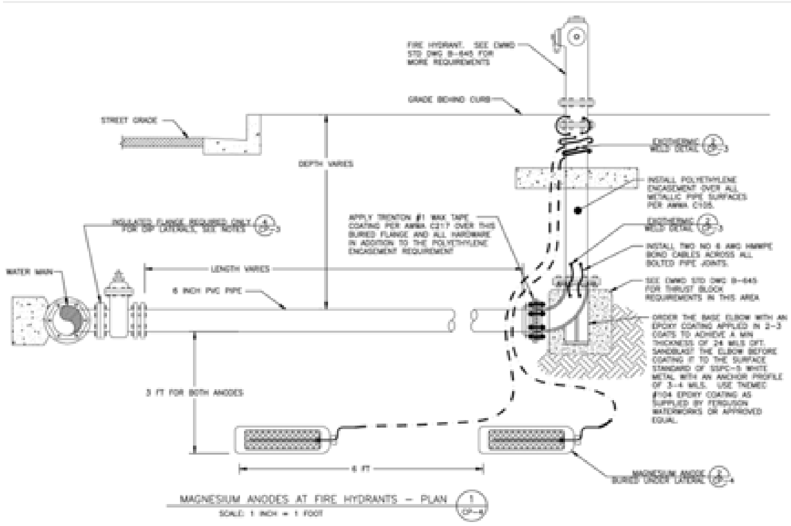


Figure 6 Sample anode design for fire hydrant underground piping

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.

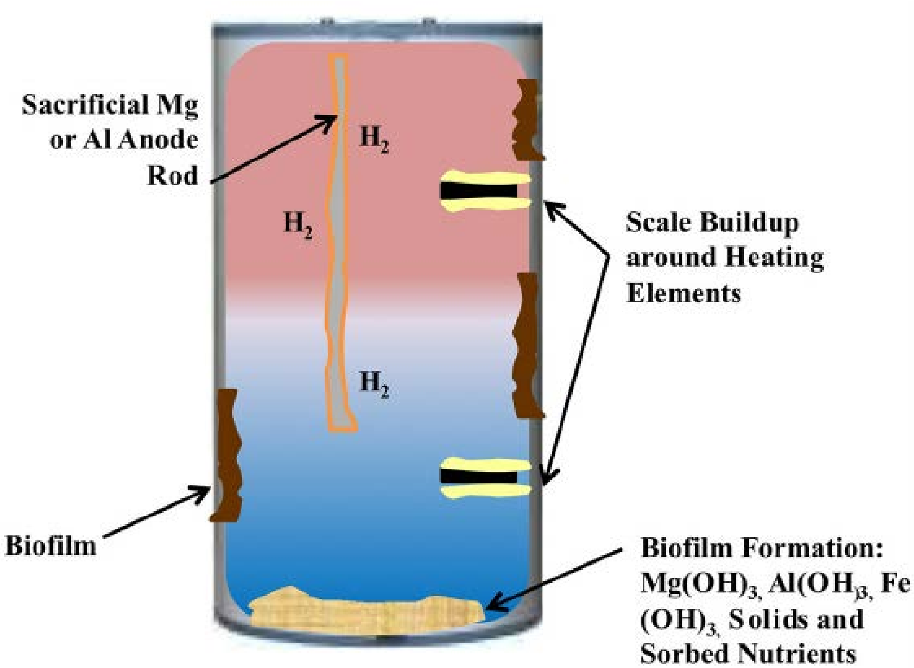


Figure 7 Cross section of boiler with anode

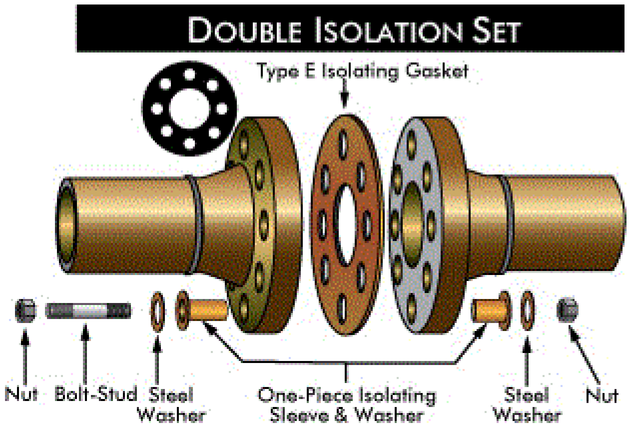
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

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. 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.

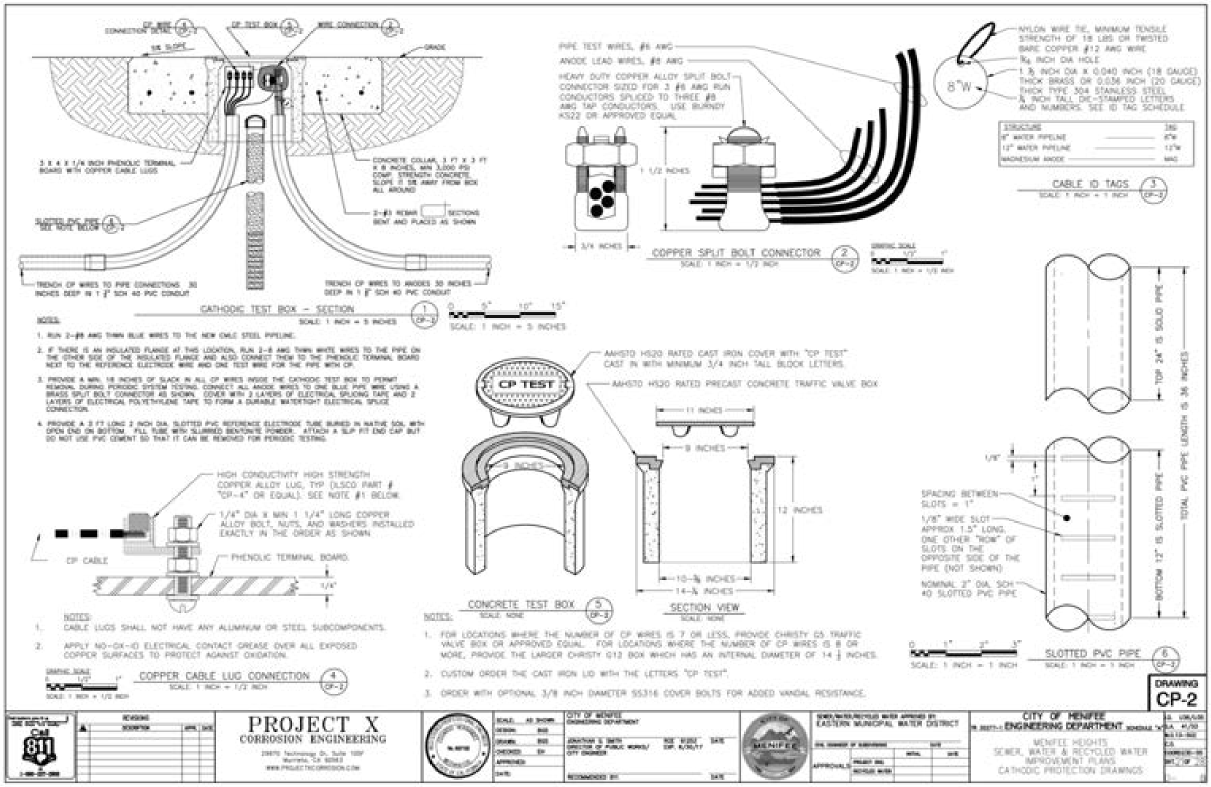


Figure 8 Sample of corrosion test station specification drawing

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.[13] 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.

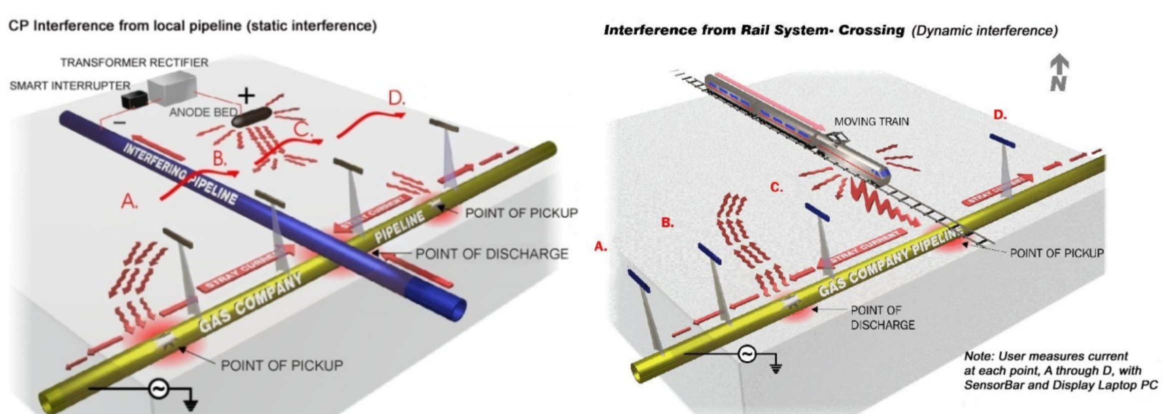


Figure 9 Examples of Stray Current[14]

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

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