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July 8, 2019
File Number 21800

Kilroy Realty Corporation
1220 West Olympic Boulevard
Suite 200
Los Angeles, California 90064

Attention: Alex King

Subject: Geotechnical Engineering Investigation
Proposed Commercial Development
1633 26th Street, Santa Monica, 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.

REGISTERED PROFESSIONAL ENGINEER
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No 2755
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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
1633 26TH STREET
SANTA MONICA, 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 drilling seven borings, collection of representative samples, laboratory testing, engineering analysis, review of available geotechnical engineering information and the preparation of this report. The boring locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report. Corrosion testing was performed by the firm Project X. A copy of their report is attached.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by Alex King of Kilroy Realty Corporation. The site is proposed to be developed with two, four story office buildings constructed over a 3 to 4 level subterranean parking garage Column loads are estimated to be between 950 and 300 kips. Wall loads are estimated to be between 16.5 and 4 kips per lineal foot. These loads reflect the dead plus live load, of which the dead load is approximately 75 percent. Grading will consist of excavations as deep as 45 feet in depth for a subterranean parking level.

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 bordered by an asphalt-paved parking lot to the northeast, Pennsylvania Avenue to the southeast, an asphalt-paved parking lot and a 3-story, an at grade office structure to the southwest, and a 5-story structure to the northwest. The site is located at 1633 26th Street, in Santa Monica, California. The site is rectangular in shape and approximately 1.2 acres in area. The site is shown relative to nearby cultural features on the attached Vicinity Map.

Site elevations range from 158 to 156 feet above mean sea level. The ground surface slopes down to the southwest at a 100 to 1 gradient. The site is currently developed with an asphalt-paved parking lot with planter islands. The planter islands have small trees and shrubs growing in them. The site development is shown on the attached Plot Plan.

The neighborhood is developed with 2- to 5-story offices.

WORK BY OTHERS

Leroy Crandall and Associates, November 17, 1969, Repot of Soil Investigation, Proposed Development, Colorado Avenue and Twenty-Sixth Street, Santa Monica, California for Higgins Brick and Tile Company, File No. A-69161.

This firm was provided with the above-referenced report by Leroy Crandall and Associates. The report describes an area that extends between Colorado and Pennsylvania Avenues, Stewart and 26th Streets. The area was used formerly as a quarry for clay soils that was later filled with soil and debris.

The investigation included drilling 25 borings with a truck-mounted drill rig equipped with an 18-inch diameter bucket auger. The purpose of the investigation was to identify the depth of fill on the site. Fill as much as 35 feet in depth was identified. Pertinent borings from that investigation are included with this report.

LOCAL GEOLOGY

The site is located in the Los Angeles Coastal Plain which is a deep, sediment-filled basin that drains to the southwest. Erosion of the Santa Monica Mountains located to the north of the site has resulted in an accumulation of several hundred feet of alluvium to form a broad southwest-draining alluvial fan. This northwest portion of the Los Angeles basin has been uplifted in the recent geologic time to form the gently rolling topography. The area in turn has been dissected by several south-draining canyons that also begin at the base of the Santa Monica Mountains. The geology of the site is shown on the attached Local Geologic Map-Dibblee.

Faulting in the area is dominated by the east-west trend of the Malibu Coast – Santa Monica Hollywood Fault system. These faults are northwest-dipping, northeast-southwest trending faults that are responsible for the uplift of the Santa Monica Mountains Hollywood Hills and deformation in the Elysian Hills. These faults are considered as the southern boundary of the Transverse Ranges Geomorphic Province. The fault system is shown on the attached Regional Geologic Map.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on April 11, 15, 22 and 23, 2019, by excavating seven borings to depths between 60 and 100 feet. The borings were drilled using a truck-mounted drilling rig equipped with 8-inch diameter hollowstem augers. Soil samples were obtained using a California-modified split spoon sampler lined with 2.5 inch diameter brass rings. In Borings 3 and 5, Standard Penetration Tests (SPTs) were performed at alternating depths with the split spoon sampler. The samplers were advanced using an automatic trip hammer and a 140-pound weight dropped from a height of 30 inches. The soil samples were collected in sealed containers and transported to our office for laboratory testing.

The boring locations are shown on the enclosed Plot Plan, and the geologic materials encountered are logged on Plates A-1 through A-7.

The boring locations were determined by measurement from hardscape features shown on the Plot Plan. The elevation of the borings was determined by interpolating between elevation contours shown on the City of Santa Monica topographic map.

Geologic Materials

The geologic materials underlying the site include fill and alluvium. The subsurface distribution of the geologic materials is shown on the attached Cross Sections A-A' and B-B'.

Fill

The fill consists of sandy silt, clayey silt , and silty sand that is mottled black and brown moist, firm to stiff and moderately dense to very dense. The fill has abundant trash including brick, concrete, metal, wood, and asphalt pieces to 2 inches in dimension.

The fill was identified in all of the borings and test pits and extends to a depth ranging from 3 to 43½ feet. LeRoy Crandall and Associates (LC&A) identified the fill thickness in the borings to be from 3 to 22 feet on site. It should be noted that the boring log for Boring B24 by LC&A indicates the fill is 4 feet deep. However, the site plan included in the report indicates the base of the fill at Boring B24 occurs at an elevation of 115 feet, approximately 40 feet below the ground surface. Borings by this firm indicate the fill depth to be near 43½ feet.

The fill soils were likely placed in the 1940's to 1950's without the benefit of current grading codes.

Alluvium

The alluvium underlies the fill and consists primarily of clayey silt, with layers of silty clay, and sand with few gravel to ½ inch in dimension. The fill is generally dark brown and gray, and firm to stiff and dense to very dense. The top of the alluvium surface is shown on the attached Alluvium Surface Contour Map. The surface descends to the southeast ranging in elevation from 155 feet to 115 feet. Cross Sections A-A' and B-B' show the distribution of fill as well.

Groundwater

Groundwater was encountered at depths of 43 to 53½ feet and corresponds to elevations ranging from 115 to 105 feet above mean sea level. The onsite borings by LC&A were drilled to a maximum depth of 35 feet and did not encounter water. It should be anticipated that the coarse alluvium layers are water bearing.

The historically highest ground water level for the site is indicated as approximately 40 feet below the ground surface which correlates to an elevation of approximately 115 feet (CDMG, 2006). A copy of this map is enclosed as Historically Highest Groundwater Levels.

The Los Angeles Department of Public Works (LADPW) lists three monitoring water wells within approximately 1.8 miles of the site (LADPW, 2010). Two of the wells are located within approximately 0.5 miles of the site. The well locations are shown on the enclosed Vicinity Map. The well logs are enclosed herein. The well readings are summarized in the following table.

GROUNDWATER WELL SUMMARY			
Well No.	**Ground Surface Elevation**	**Highest Recorded Water Surface Elevation**	**Lowest Recorded Water Surface Elevation**
2537	152.5 feet	36.5 feet on 11/1/1971	-4.5 feet on 10/31/1995
2546L	153.0 feet	43.3 feet on 4/1/1998	-20 feet on 10/31/1995
2539L	26.0 feet	18.9 feet on 3/13/1970	-2.0 on 10/31/1986

The highest recorded water surface elevations would be in excess of 100 feet in depth below the subject site. Based on these considerations, it is likely that the onsite groundwater encountered in the borings conducted on the site represents perched zones of groundwater that are trapped within more permeable soils layers or lenses.

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.

Caving

Caving did not occur in the borings after the augers were withdrawn. It should also be noted that the borings by LCA were drilled using an18 inch diameter bucket auger. The boring logs indicate that caving did not occur in those borings either.

City of Santa Monica Clay Pit Areas

Review of the City of Santa Monica Safety Element of the General Plan (Leighton, 1995) and the City of Santa Monica Geologic Hazards Map (City of Santa Monica, 2010), indicates the subject site is located outside by adjacent to a former clay pit area. However, the borings drilled by this firm and by others clearly indicate the site is within the limits of a former clay pit.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

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

The site is underlain by unconsolidated alluvial sediments deposited by river and stream processes.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last

11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

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 and results of site reconnaissance, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

The California Geological Survey (CGS) updated the Earthquake Fault Zone Map for the Beverly Hills 7.5-Minute Quadrangle (CGS, 2018). The nearest active fault indicated on the map is the Santa Monica Fault. These zones were created based on geologic evidence of active fault movement (within the last 11,000 years) along the Santa Monica and Hollywood Faults. The fault trace is not shown although Earthquake Fault Zone is indicated. The zone is shown to be about 2,200 feet north of the site.

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), does not classify the site as part of the potentially "Liquefiable" area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

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

Groundwater was encountered during exploration, between depths of 43 to 53½ feet below the ground surface. According to the Seismic Hazard Zone Report of the Beverly Hills 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically-highest groundwater level for the site was 40 feet below the ground surface. The historically highest groundwater level was conservatively utilized for the enclosed liquefaction analysis.

The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS websites, using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) and the U.S. Seismic Design Maps tool (USGS, 2013). A Site Class "D" (Stiff Soil Profile) and a published shear wave velocity of 230 meters per second were utilized for Vs30 (Tinsley and Fumal, 1985) in the USGS seismic programs. A modal magnitude (M_W) of 6.8 is obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration of 0.80g was obtained using the U.S. Seismic Design Maps tool. These parameters are used in the enclosed liquefaction analyses.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on Borings 2 and 5. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory boring are presented on the enclosed E-Plate and F-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.

The procedure presented in the SP117A guidelines was followed in analyzing the liquefaction potential of the subject site. The SP 117A guidelines were developed based on a paper titled, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", by Bray and Sancio (2006). According to the SP117A, soils having a Plastic Index greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered non-liquefiable, and the analysis of these soil layers was turned off in the liquefaction susceptibility column.

Based on CGS Special Publication 117A (CDMG, 2008), a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk where high-quality, site-specific penetration resistance and geotechnical laboratory data is collected. Based on the enclosed liquefaction analysis, the lowest factor of safety calculated for soil layers considered susceptible to the occurrence of liquefaction is 3.5.

The site-specific liquefaction analysis included in the Appendix, indicates that the site soils would not be capable of liquefaction during the ground motion expected during the design basis earthquake.

Lateral Spreading

Lateral spreading is the most pervasive type of liquefaction-induced ground failure. During lateral spread, blocks of mostly intact, surficial soil displace downslope or towards a free face along a shear zone that has formed within the liquefied sediment. The enclosed liquefaction analysis included in the Appendix, indicates that site soils would not be prone to liquefaction during 475 year return period ground motion. Therefore, lateral spreading is considered to be remote.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the Tsunami Inundation Map for Emergency Planning, for the Beverly Hills Quadrangle (CEMA, 2009), the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990),

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

The purposed structure will extend to the property line limits and will be four stories in height and have three to four levels of subterranean parking. The finish floor elevation of the proposed basement will be either 125 or 115 feet above mean sea level.

The site is underlain by up to 43 ½ feet fill soil. The fill soils contain abundant debris and were not placed under the requirements of current building codes. The fill is in turn, underlain by alluvium consisting of interlayered mixtures of clay silt and sand. The alluvium is generally dense and stiff. Groundwater was encountered between the depths of 43 to 53 ½ feet (equivalent to elevations of 105 to 115 feet). The historically highest groundwater level is at 40 feet below the ground surface (equivalent to elevation 115 feet).

The soils underlying the site are not capable of liquefaction during the design earthquake. The site is not bisected by the trace of any known fault.

If the proposed structure extends 4 levels below grade, nearly all of the fill soil would be removed for the basement for the floor slab and foundation elements. Conventional footings bearing in the natural alluvial soils may be used for support of the proposed structure. A conventional slab may be used if it bears on the on the alluvial soils. A mat foundation may also be used.

If the basement extends to a depth of 3 subterranean levels, the fill soils will occur below the basement slab on the southeast 1/3 of the site. Where the basement is underlain by fill, the slab must be designed as a structural slab and the footings must be either deepened to extend into the alluvial soil or friction piles may be used that derive support from the underlying alluvium.

Excavation of the proposed subterranean levels will require shoring and possibly dewatering measures to provide a stable and dry working area due to the proposed depth, the fine grained consistency of the onsite soils, the presence of groundwater.

Due to the different strength properties of the fill and alluvium, the lateral load on the shoring and retaining walls will be different. The fill soil will be exposed on the northeast, southeast and southwest sides of the excavation yielding larger loads than the northwest side.

Foundations for small outlying structures, such as property line walls, which will not be tied-in to the proposed development, may be supported on conventional foundations bearing in native geologic materials.

SEISMIC DESIGN CONSIDERATIONS

2016 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. This information and the site coordinates were input into the USGS U.S. Seismic Design Maps tool (Version 3.1.0) to calculate the ground motions for the site.

2016 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.09g
Site Coefficient (F_a)	1
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.09g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.393g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.774g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.162g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.774g

FILL SOILS

The maximum depth of fill encountered on the site was 43 ½ feet. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and removed from the site. The existing fill should not be reused for backfilling.

EXPANSIVE SOILS

The onsite geologic materials are in the moderate expansion range. The Expansion Index was found to be 67 for bulk samples remolded to 90 percent of the laboratory maximum density. Additional reinforcing is required as noted in the "Foundation Design" and "Slabs 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 sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, 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. Concrete strength should be a minimum of 2,500psi.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomena wherein soils lose volume when they are saturated. This can result in settlement of structures bearing thereon. The hydroconsolidation potential of the site soils was considered in the provision of 11 consolidation tests. None showed collapse upon saturation of the sample. Based on the laboratory testing, it is the opinion of Geotechnologies, Inc. that the potential for damaging settlement due to hydrocollapse is anticipated to be insignificant.

PERMANENT DEWATERING

Based on review of the California Division of Mines and Geology Seismic Hazard Zone Report for the Beverly Hills, 7.5-Minute Quadrangle, Report No. 023, the Historically Highest Ground Water level is at a depth of 40 feet beneath the site. The proposed 4-level basement will be at this depth, therefore the building will not be required to be designed for potential hydrostatic and buoyancy pressures. If a 3-level basement is planned, the finish floor elevation will be well above the historically highest groundwater level and not require dewatering.

The subterranean portion of this building should be designed with drainage devices to relieve hydrostatic pressure. These devices include drains outside the retaining walls.

The source of water for the drains will be seepage water from plants, and leaking pipes. The system collecting water in such a system should be capable of pumping at least 10 gallons per minute.

GRADING GUIDELINES

Site Preparation

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

Compaction

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

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

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

Acceptable Materials

The natural alluvial soils should be selectively stockpile to retain the sandy soils from later use in backfilling. The fill soils should not be used as compacted fill.

Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 30. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

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

Wet Soils

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

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

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

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

Abandoned Seepage Pits

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

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

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

Geotechnical Observations and Testing During Grading

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

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

LEED Considerations

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

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

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

FOUNDATION DESIGN

Conventional

Conventional foundations may bear in the natural alluvial soil. All conventional foundations for a structure should bear in the same material.

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 and 24 inches in depth below the lowest adjacent grade and 24 inches into the natural alluvial soil.

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 and 24 inches in depth below the lowest adjacent grade and 24 inches into the natural alluvial soil.

The bearing capacity increase for each additional foot of width is 150 pounds per square foot. The bearing capacity increase for each additional foot of depth is 450 pounds per square foot. The maximum recommended bearing capacity is 4,000 pounds per square foot.

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

Deepened Footings

Where the subgrade does not expose the alluvial soil, foundations will require deepening through the fill into the underlying alluvial soil. The deepened portion of the footings may be filled with concrete of the same mix as that specified for the footing or 3-sack slurry. The initial pour would not require reinforcing as it is simply passing the load through to the recommended bearing material. Once the initial pour has hardened, the footing may be reinforced and poured on top of the first pour. Some method of creating a positive bond between the two pours should be employed. Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

Controlled Low Strength Material

Where the deepened footings are needed, 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. This value translates to over 14,000 pounds per square foot.

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.

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed structure may bear in compacted fill. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

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

Foundation Reinforcement

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

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed alluvium may be computed as an equivalent fluid having a density of 400 pounds per cubic foot with a maximum earth pressure of 4,000 pounds per square foot.

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

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1.75 inches and occur below the heaviest loaded columns. Differential settlement is not expected to exceed 0.5 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.

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. All micropiles shall be constructed under the continuous observation by representatives of this firm. Foundations should be deepened to extend into satisfactory geologic materials, if necessary.

FOUNDATION DESIGN - MAT FOUNDATION

Mat Foundation

The mat should be founded exclusively in newly placed compacted fill, subsequent to the recommended grading. The bottom of the mat foundation should be a minimum of 18 inches in depth below the lowest adjacent grade at the perimeter of the structure. Given the size of the proposed mat foundation, the average bearing pressure of 1,200 pounds per square foot is well below the allowable bearing pressures. For design purposes, an allowable bearing pressure of 3,000 pounds per square foot, with locally higher pressures up to 4,000 pounds per square foot may be utilized in the mat foundation design.

Modulus of Subgrade Reaction

A unit modulus of subgrade reaction of 150 pounds per cubic inch may be utilized for design of foundations. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with the larger footings:

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

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

Lateral Design for Mat Foundation

Resistance to lateral loading may be provided by soil friction, and by the passive resistance of the soils. A coefficient of friction of 0.3 may be used with the dead load forces between footings and the underlying supporting soils.

Passive earth pressure for the sides of footings poured against undisturbed soil may be computed as an equivalent fluid having a density of 400 pounds per cubic foot, with a maximum earth pressure of 4,000 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads. A minimum safety factor of 2 has been utilized in determining the allowable passive pressure.

Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. The maximum settlement is not expected to exceed approximately 3 inches, and will occur below the most heavily loaded area of the mat foundation. Differential settlement is not expected to exceed 0.5 inch.

FOUNDATION DESIGN - FRICTION PILES

Vertical Capacities

A deepened foundation system consisting of friction piles should be utilized for support of the proposed structure where the depth to natural soils is too great for a deepened foundation. The capacities of drilled cast-in-place piles are shown on the enclosed “Drilled Cast in Place Pile Capacities” chart. Capacities based on dead plus live load are indicated. A one-third increase may be used for transient loading such as wind or seismic forces. The capacities presented are based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

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

Lateral Design

Lateral loads may be resisted by the piles, and by the passive resistance of the soils against the pile caps. The passive resistance of the existing soils against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 200 pounds per cubic foot. A one-third increase in this value may be used for wind or seismic loads. The resistance of the piles and the passive resistance of the soils against pile caps and grade beams may be combined without reduction in determining the total lateral resistance.

Analyses of the proposed piles using a varying shear loads were performed using the program RSPile (2018) included in the Appendix of this report. The printouts show the calculated shear, moment, and deflection of the proposed piles. The analyses were performed for 24 and 36-inch diameter, drilled, cast-in-place friction piles. Assumed as part of these lateral capacity calculations are:

- A Fixed Head Condition
- A 200 kip vertical load
- A concrete modulus of elasticity of 3,605,000 pounds per square inch (psi)
- Lateral shear load of 20 kips
- Radial Reinforcement consisting of 10 #10 bars (24-in.) or 14 #10 (36-in.)
- The modeled soil condition: Finish floor elevation 125 feet (3 level basement)
  10 feet of uncertified fill over alluvium
  Water at 115 feet.

The output from the program is attached to the Appendix of this report. If any of these assumptions are not valid, please contact this firm and a modified analysis can be performed.

Pile Installation

Due to the nature of the existing geologic materials encountered during exploration, caving is not anticipated during drilling of the proposed piles above the water table. Some caving should be anticipated below the water table but in the colluvium and alluvium. No caving is anticipated below the water table and in the bedrock.

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

Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube, at least 4 inches in diameter, connected to a concrete pump. The tube shall be equipped with a valve at the bottom that will prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Settlement

The maximum settlement of pile-supported foundations is not expected to exceed ½ inch. Differential settlement is expected to be negligible.

RETAINING WALL DESIGN

Due to the wedge-shaped configuration of the fill, the northwest side will expose mostly alluvium and the northeast, southeast and southwest walls will expose mostly fill. The alluvium has higher strength parameters and will therefore exert less of a lateral load on the shoring and retaining walls than the fill. Therefore, the lateral loads are dependent on wall orientation.

Cantilever Retaining Walls

Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls up to 40 feet in height may be designed according to the following table.

RETAINING WALLS – CANTELEVERED (Active Pressure) (Pounds per square foot)		
WALL HEIGHT (feet)	**SUPPORTED MATERIAL FILL (Northeast, Southeast and Southwest Walls)**	**SUPPORTED MATERIAL ALLUVIUM (Northwest Wall)**
Up to 10	44	30
10 to 20	62	34
20 to 30	68	43
30 to 40	71	49

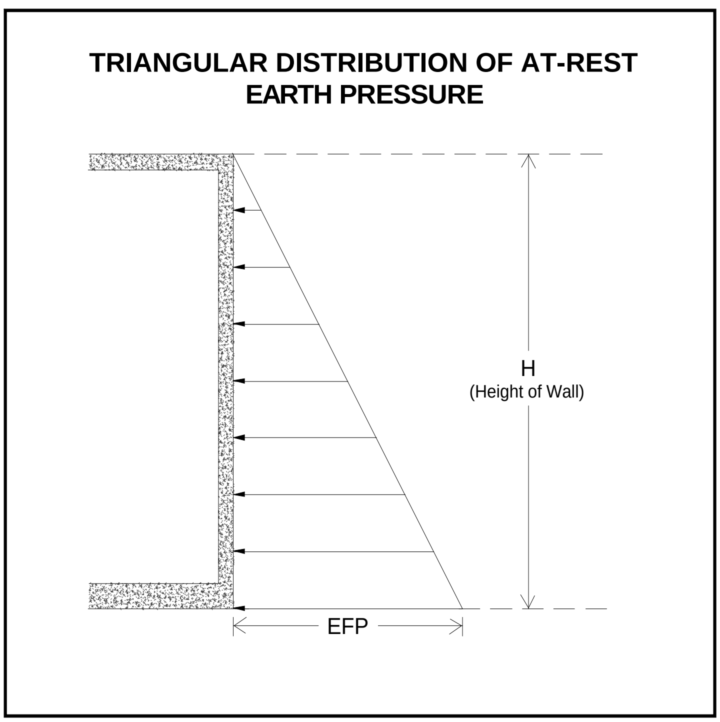
For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained 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 dependent on the material being supported. The loads are in accordance with the following table:

RETAINING WALLS – RESTRAINED (At Rest Pressure) (Pounds per square foot)		
WALL HEIGHT (feet)	**SUPPORTED MATERIAL- FILL (Northeast, Southeast and Southwest Walls)**	**SUPPORTED MATERIAL ALLUVIUM (Northwest Wall)**
Up to 40	87	66

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

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

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

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

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 gravel. The rock pockets should be no more than 8 feet on center.

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 43 ½ (elevation 115 feet) which corresponds to 0 feet below the base of the proposed structure (4 level subterranean) or 10 feet below the base of the proposed structure (3 level subterranean). Therefore the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 10 gallons per minute may be assumed.

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 16.3 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.

Surcharge from Adjacent Structures

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

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

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

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

where:

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

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

Waterproofing

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

Waterproofing is recommended for retaining walls. 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 in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Excavations on the order of 30 to 40 feet in vertical height will be required for the subterranean levels. Assuming the thickness of the concrete slab-on-grade and the foundations, excavations up to 50 feet have been addressed herein. The excavations are expected to expose fill and dense alluvium 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 up to 24 feet in height could be cut at a uniform 1 to 1 slope gradient in either fill or alluvium. A uniform 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 Buildings or Property Lines

Where foundation excavations will leave an adjacent foundation unsupported the foundation excavation should be 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.

Temporary Dewatering

Currently it is proposed that the structure will extend to a depth of up to 45 feet below existing site grades. Continuous groundwater is expected from a depth of approximately 43 feet.

A dewatering contractor should be consulted for temporary dewatering. The dewatering system may consist of wells installed around the perimeter of the site. The pumps should be turned on several weeks in advance of construction to draw down the water level in the site vicinity. The water level should be drawn down sufficiently to permit a stable and dry surface at the bottom of the proposed excavation. The collected water should be pumped to an acceptable disposal area. Appropriate permits and water testing will be necessary.

Where the exposed subgrade is wet, pumping (yielding or vertical deflection of the subgrade) may be encountered. Under these conditions please refer to the "Wet Soils" section of this report.

Excavation Observations

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

SHORING DESIGN

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

Soldier Piles – Drilled and Poured

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.

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 400 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

Groundwater was encountered during exploration at a depth of 43 ½ feet below grade. Proposed piles are to be in excess of 60 feet in depth and will, therefore, encounter water. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 4 inches connected to a concrete pump. The tremie shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Casing may be required should caving be experienced in the granular (saturated) 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 5 feet.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 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 250 pounds per square foot in the alluvial soil. 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.

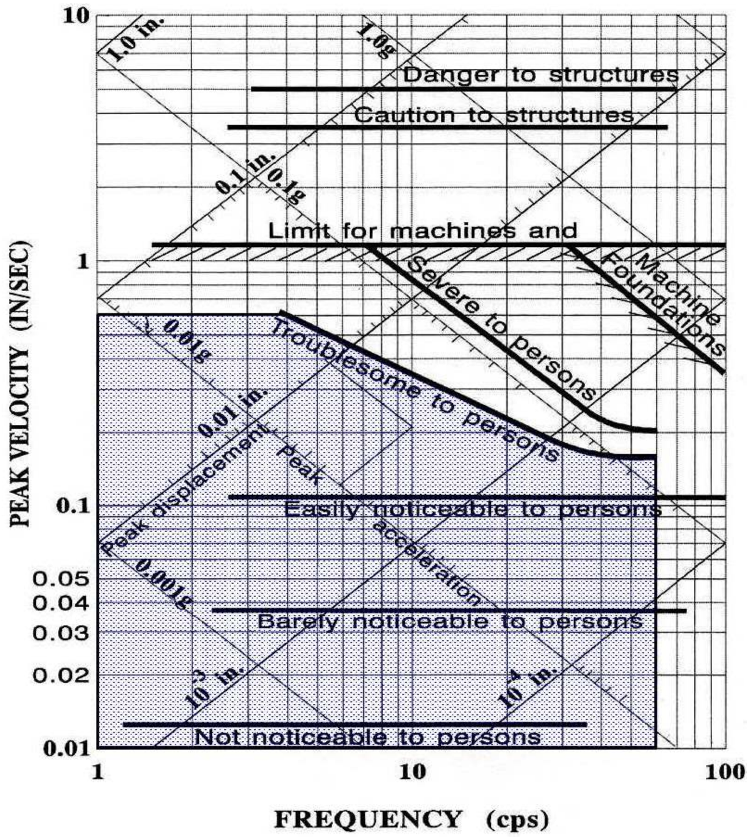
Soldier Piles – Vibrated

The vibration method of shoring pile installation is acceptable to this firm from a geotechnical standpoint provided the recommendations presented herein are implemented. When using the vibration method of installing the soldier beams, the minimum embedment depth shall be 10 feet below the lowest excavated plane.

If predrilling is required, it is recommended that the diameter of the predrilled holes should not exceed 75 percent of the depth of the web of the I-beam. The depth of the predrilled holes should not exceed the planned excavation depth. In addition, when predrilling, the auger shall be backspun out of the pilot holes, leaving the soils in place. All shoring (predrilling, installation of shoring piles, tieback installation and testing, and lagging) shall be performed under the continuous inspections by a deputy grading inspector of this firm.

The allowable level of vibration that results from the installation of the piles should not exceed a threshold where occupants of the nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation. There is a relationship between particle velocity and vibration frequency that will occur due to the installation. A range of tolerable particle peak velocity and frequency of vibration is attached an "Allowable Amplitude of Vertical Vibrations". The shaded area on the graph is considered within acceptable limits to avoid damage to nearby structures. The acceptable limits should be measured at the neighboring structures.

The vibrations should be monitored with a seismograph during pile installation to detect the magnitude of vibration and oscillation experienced by the adjacent structure. The results should be recorded and provided to the owner. If, during installation, the vibrations exceed the range shown on the graph below, the shoring contractor should modify the installation procedure to reduce the values to the acceptable range.



Given Velocity = 0.2 inch/sec.
Frequency = 10 cps
Then from Graph, Displacement = 0.003 inches
Acceleration = 0.03g
Motion is easy noticeable or troublesome to persons

NOTE: Shaded area considered below threshold for structure damage

REFERENCE: Department of Defense, 1997, Soil Dynamics and Special Design Aspects, MIL-HDBK-1007/3

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 may be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

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.

Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. This skin friction is based on 15 foot high shoring, a tied back anchor elevation 6 feet below grade and a minimum twenty foot embedment beyond the potentially active wedge yielding an overburden of 12½ feet below ground surface.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 1,000 pounds per square foot could be utilized for post-grouted anchors in the fill soils and 2,000 psf in the alluvium. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

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

The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been applied. All anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches.

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

Anchor Installation

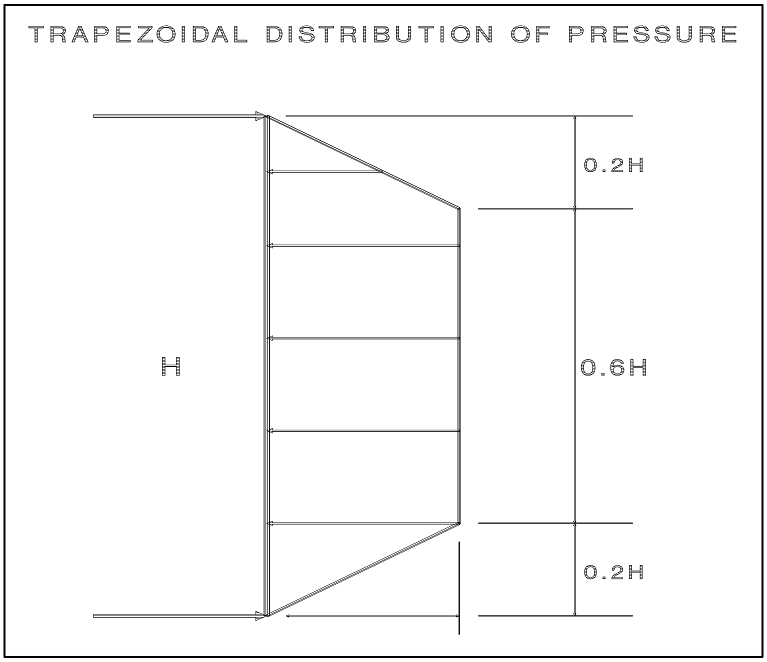
Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete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

Lateral Pressures

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

RETAINING WALLS – CANTELEVERED (Active Pressure) (Pounds per square foot)		
WALL HEIGHT (feet)	**SUPPORTED MATERIAL-FILL (Northeast, Southeast and Southwest Walls)**	**SUPPORTED MATERIAL ALLUVIUM (Northwest Wall)**
Up to 10	34	28
10 to 20	53	28
20 to 30	60	34
30 to 40	63	39
40 to 50	65	43

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:

RETAINING WALLS – RESTRAINED (Active Pressure) (Pounds per square foot)		
WALL HEIGHT (feet)	**SUPPORTED MATERIAL FILL (Northeast, Southeast and Southwest Walls)**	**SUPPORTED MATERIAL ALLUVIUM (Northwest Wall)**
Up to 10	21H	18H
10 to 20	33H	18H
20 to 30	38H	21H
30 to 40	39H	24H
40 to 50	41H	27H

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.

Deflection

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

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.

Pre-Construction Survey

Prior to excavation of the proposed basement levels, it is recommended the surrounding structures and improvements be surveyed to provide a documented record of their condition. It is recommended this include video and/or photographic documentation as well. Such a survey would aid in the resolution of any disputes that may arise concerning damage to adjacent facilities caused by the proposed construction.

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.

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.

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 undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Where the slab on grade is located over deep fill soils that cannot be removed and recompacted, a structural slab must be constructed. Where the tow slab meets, a structural joint must be installed to permit differential movement of the slabs.

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

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

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

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

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 D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

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

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

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

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

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

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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.

Due to the depth of the proposed structure and the relatively shallow depth to groundwater, stormwater infiltration is not feasible. Some other means of stormwater disposal should be considered.

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 soil corrosion potential testing performed by the laboratory Project X. The outside laboratory indicates that the electrical resistivities of the soils were in the severely corrosive categories. Soil pH values of the samples ranged between 7.4 to 8.8, indicating level not detrimental for copper or aluminum alloys, but allow corrosion to steel and iron in moist environments.

Sulfates ranged between 39 mg/kg to 565 mg/kg. These levels are negligible for corrosion of metal and cement. Any type of cement may be used in contact with the alluvium or fill.

Detailed results, discussion of results and recommended mitigating measures are provided within the corrosion report presented herein.

Geotechnologies, Inc. does not practice in the field of corrosion engineering and mitigation. Any questions regarding the results of the soil corrosion report should be addressed to the corrosion engineer.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

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

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

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

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

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

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

EXCLUSIONS

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

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Grain Size Distribution

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

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

The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

Moisture and Density Relationships

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

Direct Shear Testing

Shear tests are performed in general accordance with the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.005 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

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

Consolidation Testing

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

Expansion Index Testing

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

Laboratory Compaction Characteristics

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

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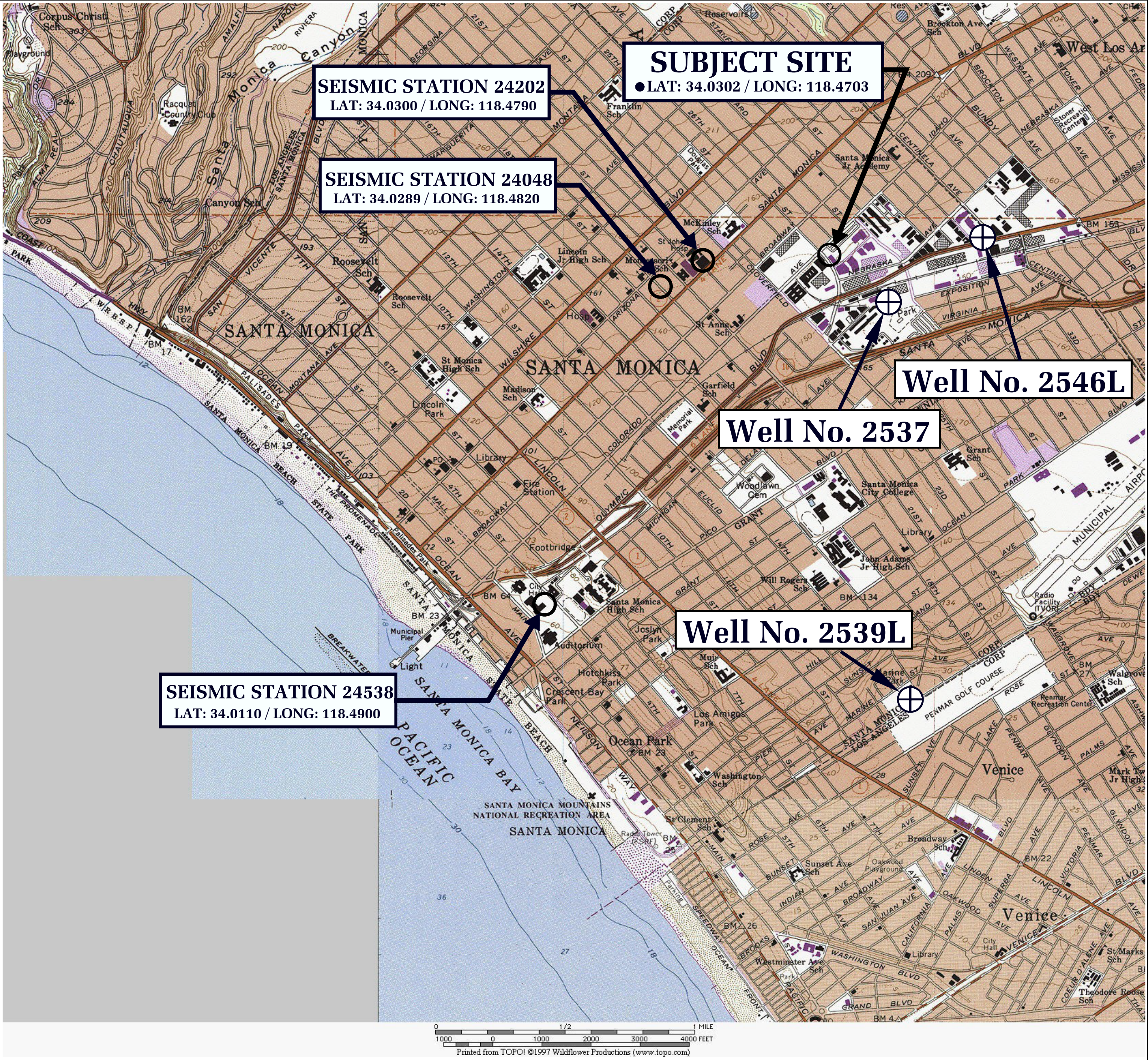

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, BEVERLY HILLS, CA QUADRANGLE

VICINITY MAP

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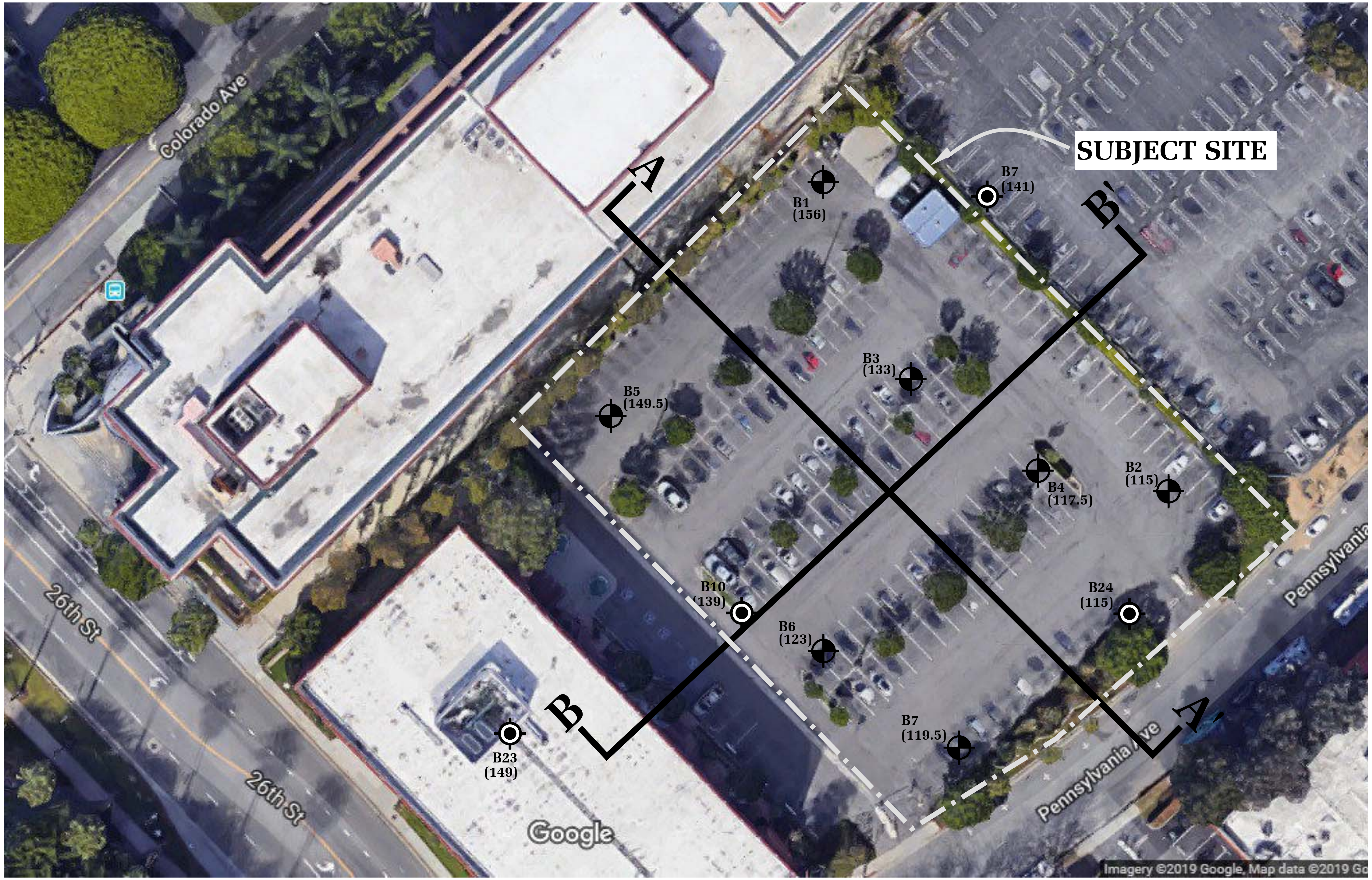

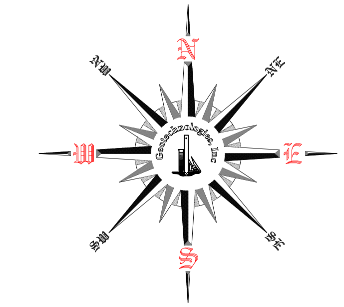

SCALE IN FEET

0 30 50 100

LEGEND

B7 (119.5) — LOCATION & NUMBER OF BORINGS WITH ELEVATION OF ALLUVIUM (THIS INVESTIGATION)

B24 (115) — LOCATION & NUMBER OF BORINGS WITH ELEVATION OF ALLUVIUM LEROY CRANDALL & ASSOCIATES (A-69161)

B B' — CROSS-SECTION LOCATION

REFERENCE: GOOGLE MAPS, 2019

PLOT PLAN

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KILROY REALTY
1633 26TH ST., SANTA MONICA

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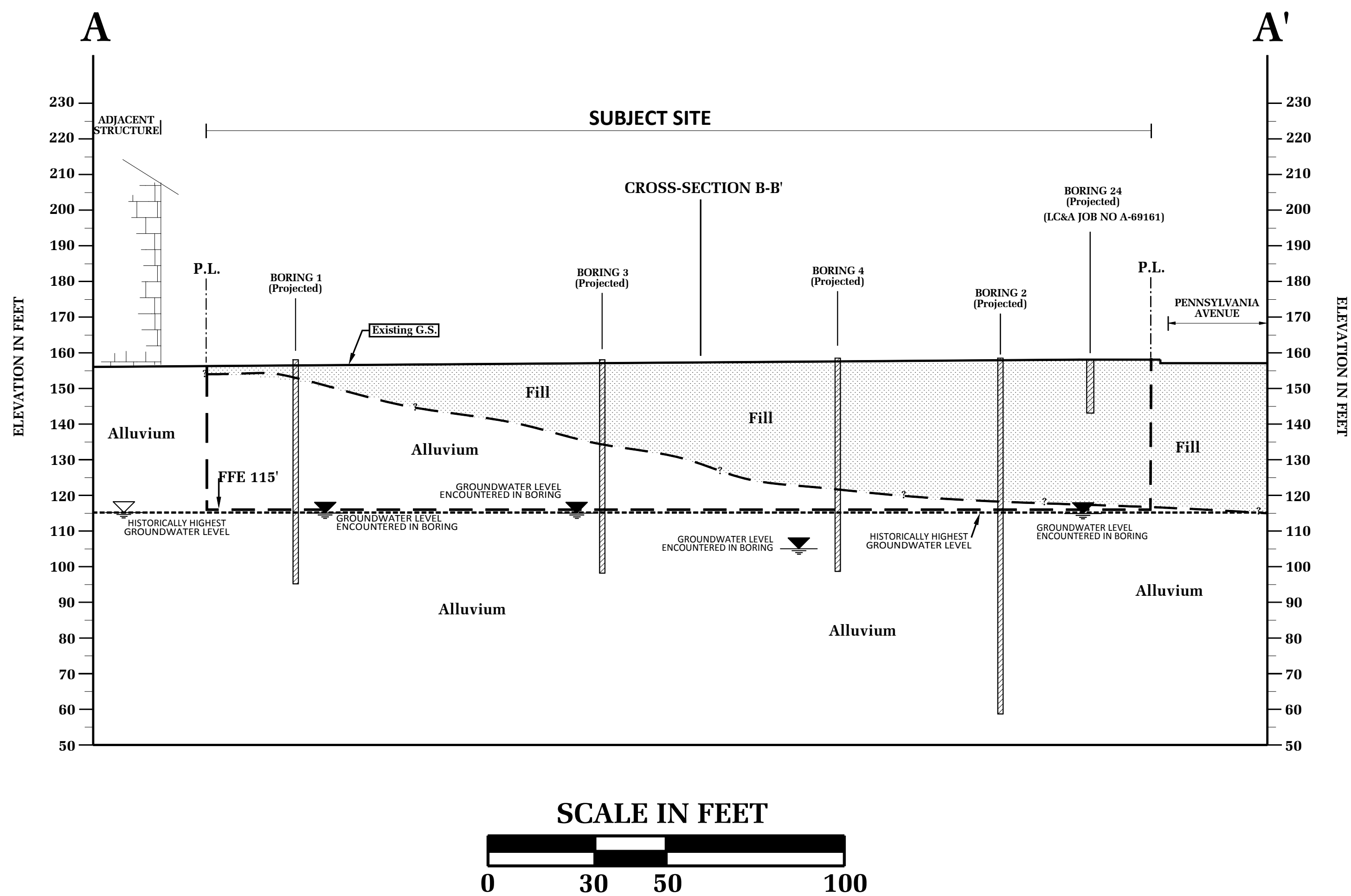

CROSS-SECTION A-A'

Geotechnologies, Inc.
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KILROY REALTY
1633 26TH ST., SANTA MONICA

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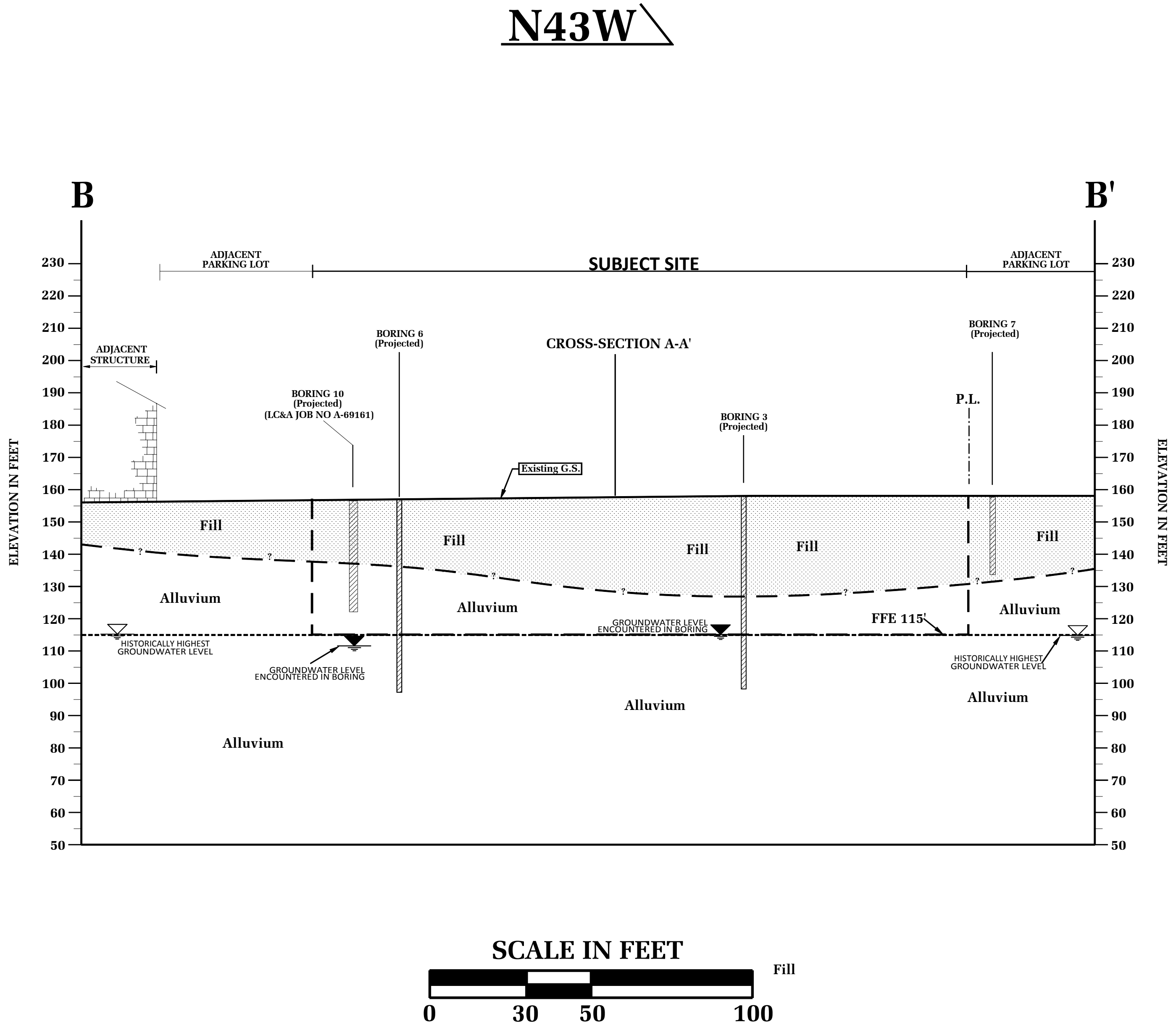
DATE: May 2019	



CROSS-SECTION B-B'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY
1633 26TH ST., SANTA MONICA

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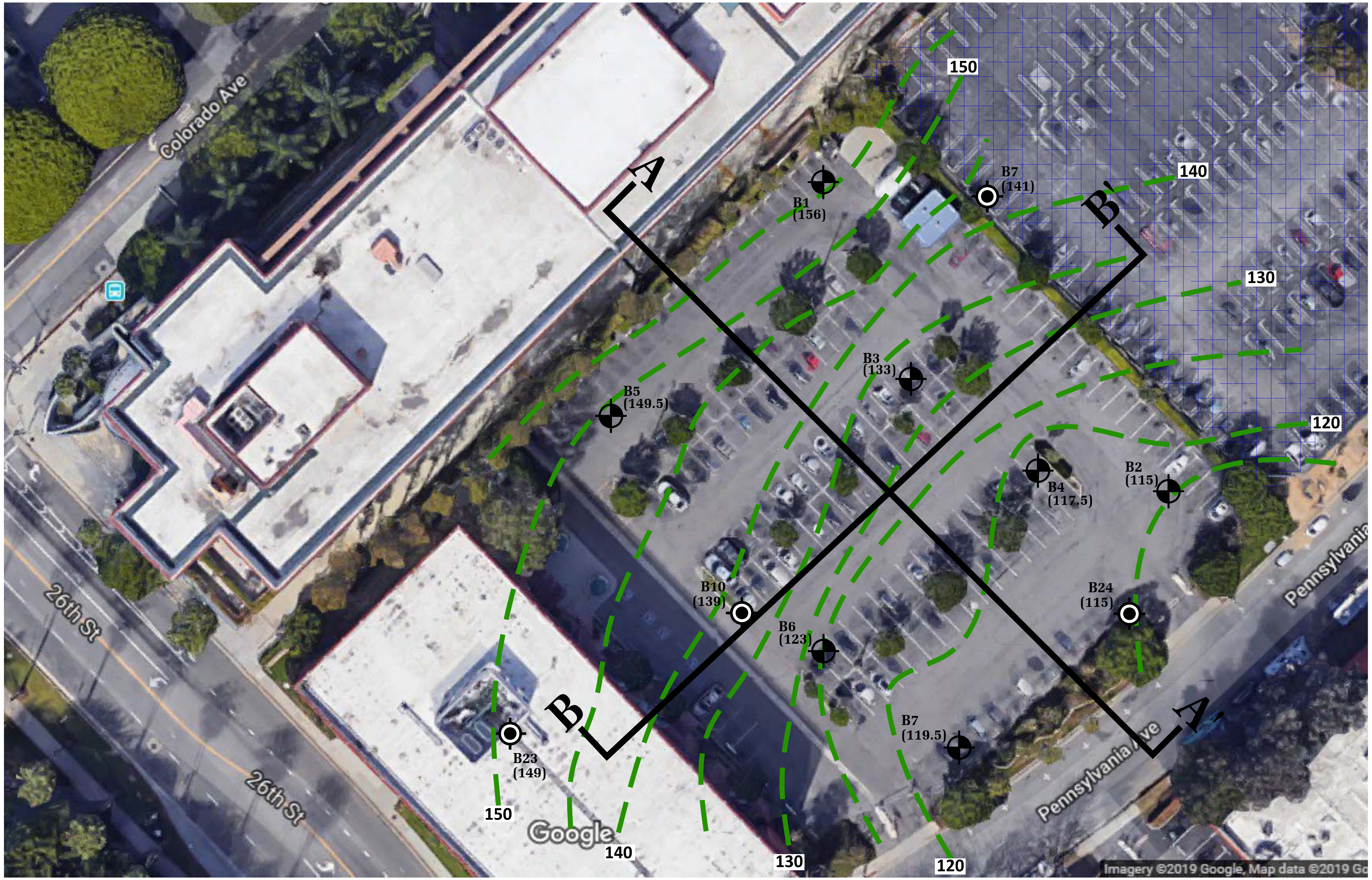
DATE: May 2019	



SCALE IN FEET

0 30 50 100

LEGEND

B7 (119.5) LOCATION & NUMBER OF BORINGS WITH ELEVATION OF ALLUVIUM (THIS INVESTIGATION)

B24 (115) LOCATION & NUMBER OF BORINGS WITH ELEVATION OF ALLUVIUM LEROY CRANDALL & ASSOCIATES (A-69161)

B B' CROSS-SECTION LOCATION

120 ELEVATION OF ALLUVIUM SURFACE (IN FEET)

REFERENCE: GOOGLE MAPS, 2019

ALLUVIUM SURFACE ELEVATION CONTOUR MAP

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KILROY REALTY
1633 26TH ST., SANTA MONICA

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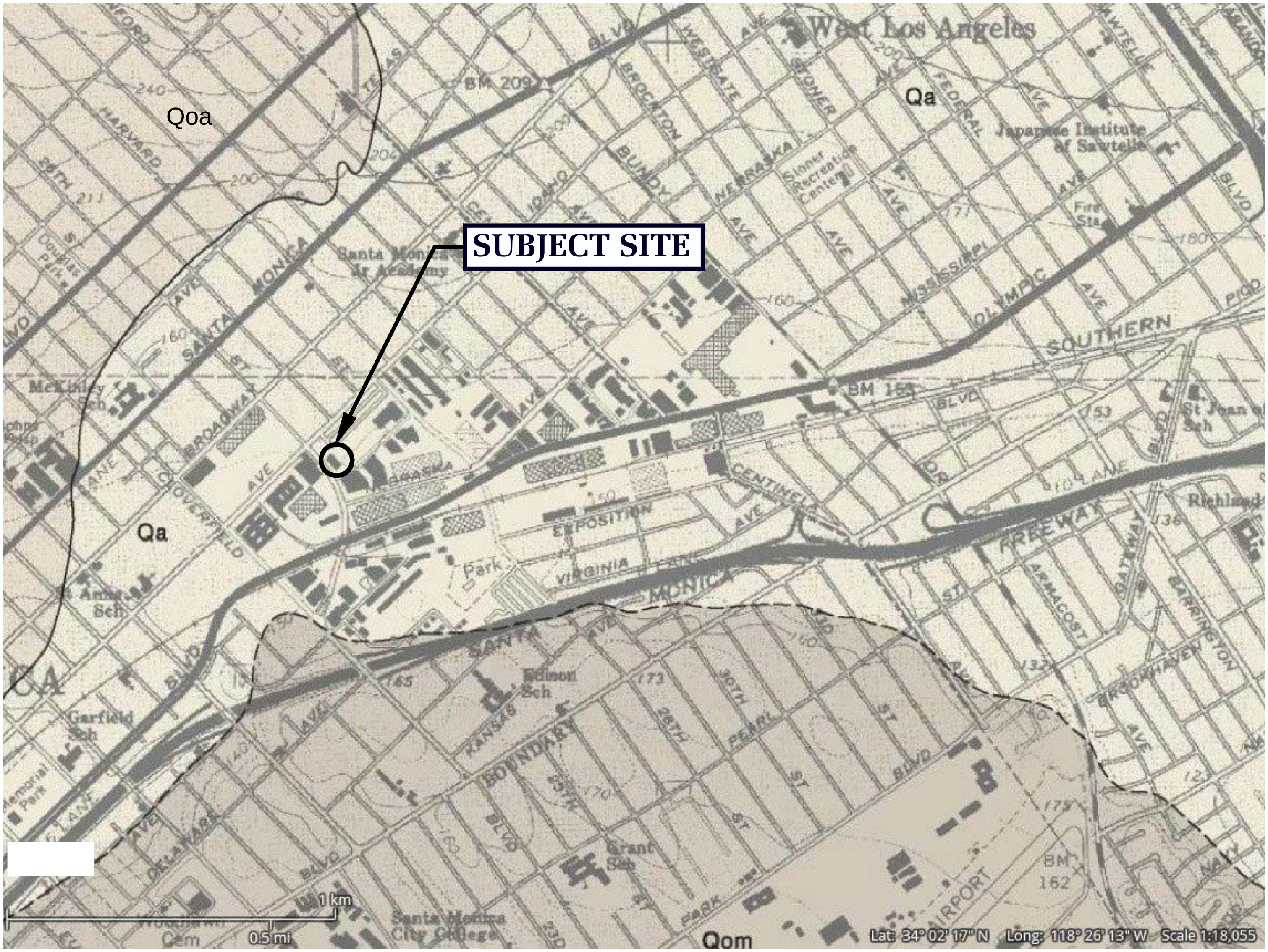
DATE: May 2019	



LEGEND

Qa: Surficial Sediments - alluvial gravel, sand and silt-clay; includes gravel and sand of stream channels

Qoa: Older Surficial Sediments - older alluvium gray to light brown pebble-gravel, sand, silt and clay

Qom: Shallow Marine Sediments - (Marine Deposits of Hoots 1931) light gray to light brown sand, pebbly sand gravel and silt

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., (1991) GEOLOGIC MAP OF THE BEVERLY HILLS AND VAN NUYS (SOUTH HALF) QUADRANGLES (#DF-31)

N

LOCAL GEOLOGIC MAP - DIBBLEE

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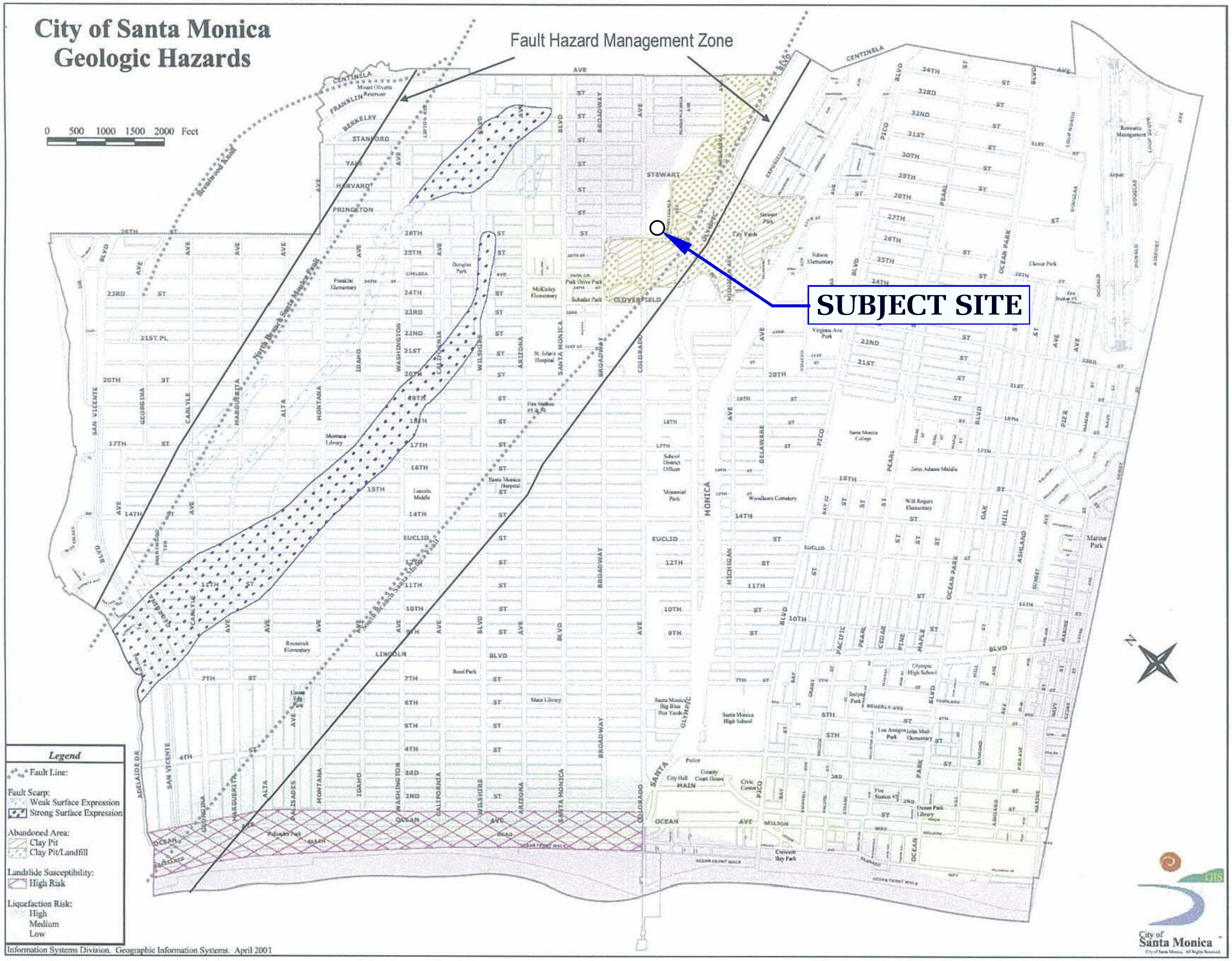
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION

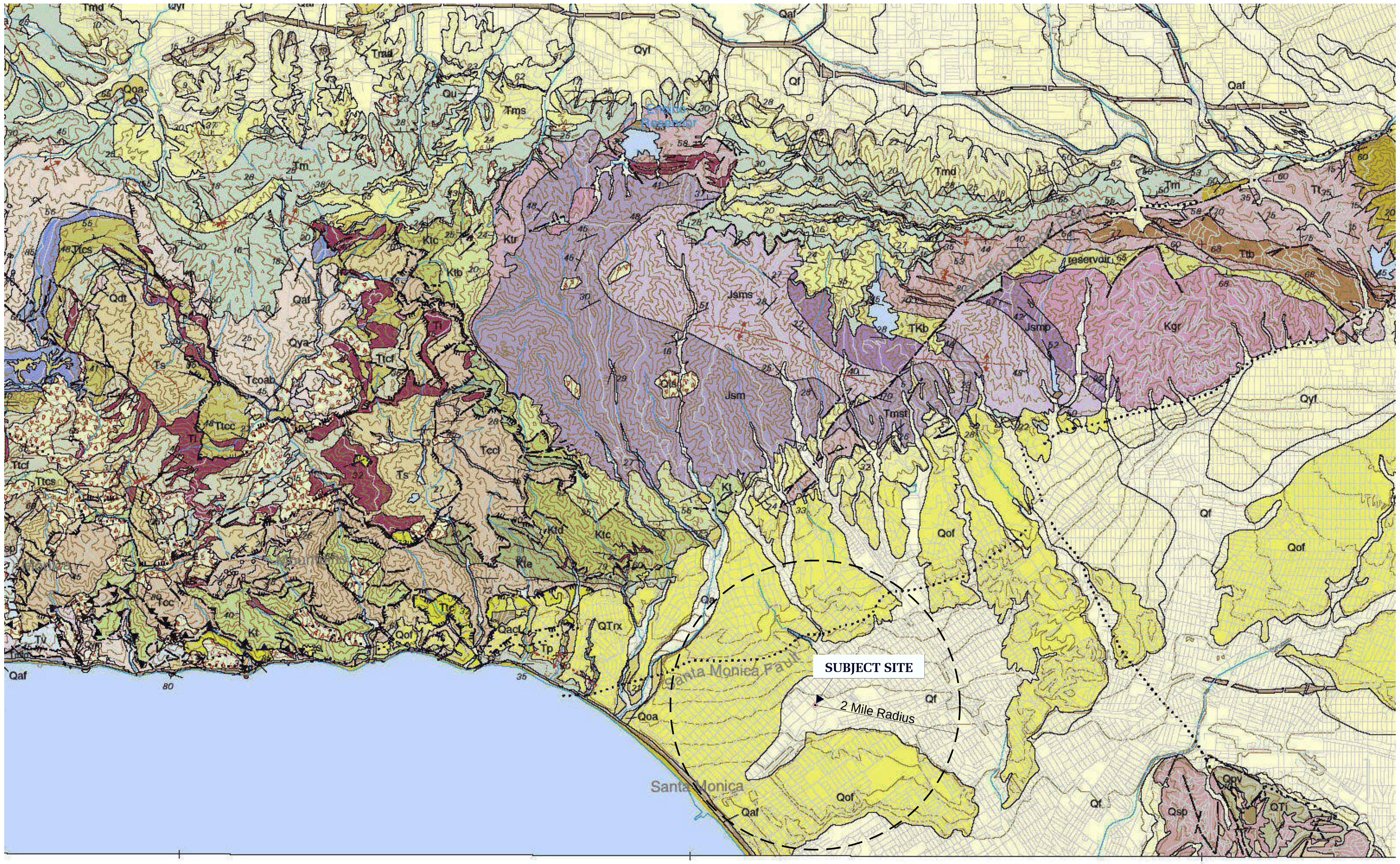
FILE NO. 21800

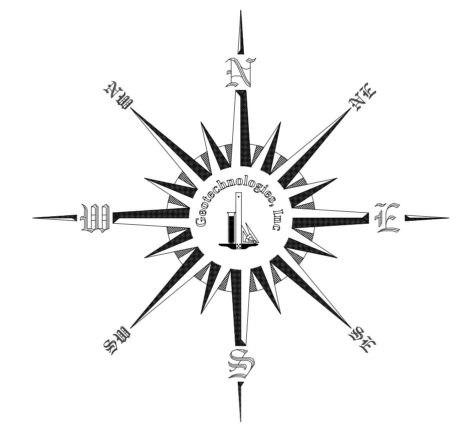


1. A discussion of fault rupture hazards is required for all projects located within the Fault Hazard Management Zone. (Section 3.3.1). The Fault Hazard Management Zone extends 380 to nearly 500 feet north of the north branch and 100 to nearly 600 feet south of the south branch of the Santa Monica fault.
2. The above map is taken from the "*Technical Background Report to the Safety Element of the City of Santa Monica General Plan*", by Leighton & Associates, Inc., March 30, 1994.
3. The more recent state maps depicting the Hazard Zones associated with liquefaction and landslides supercede those shown above and are used in the application of these geotechnical guidelines.

Figure 2

GEOLOGIC HAZARD MAP	
Geotechnologies, Inc. Consulting Geotechnical Engineers	KILROY REALTY CORPORATION
	FILE NO. 21800





LEGEND

- Qaf: Artificial Fill
- Qa: Alluvium
- Qf: Alluvial-Fan Deposits
- Qof: Old Alluvial-Fan Deposits
- Qoa: Old Alluvium
- Tm: Modelo Formation
- Tt: Topanga Group
- TKb: Sedimentary Rock in the Beverly Hills Area
- Kt: Tuna Canyon Formation
- Jsm: Santa Monica Slate

Fault - Solid where accurately located, dashed where approximately located, dotted where concealed, quieried where location or existence uncertain. includes strike slip, normal, reverse, oblique, and unspecified slip.

1 .5 0 1 2 3 4 5 6 7 MILES

1 .5 0 1 2 3 4 5 6 7 KILOMETERS

Contour Interval 40m

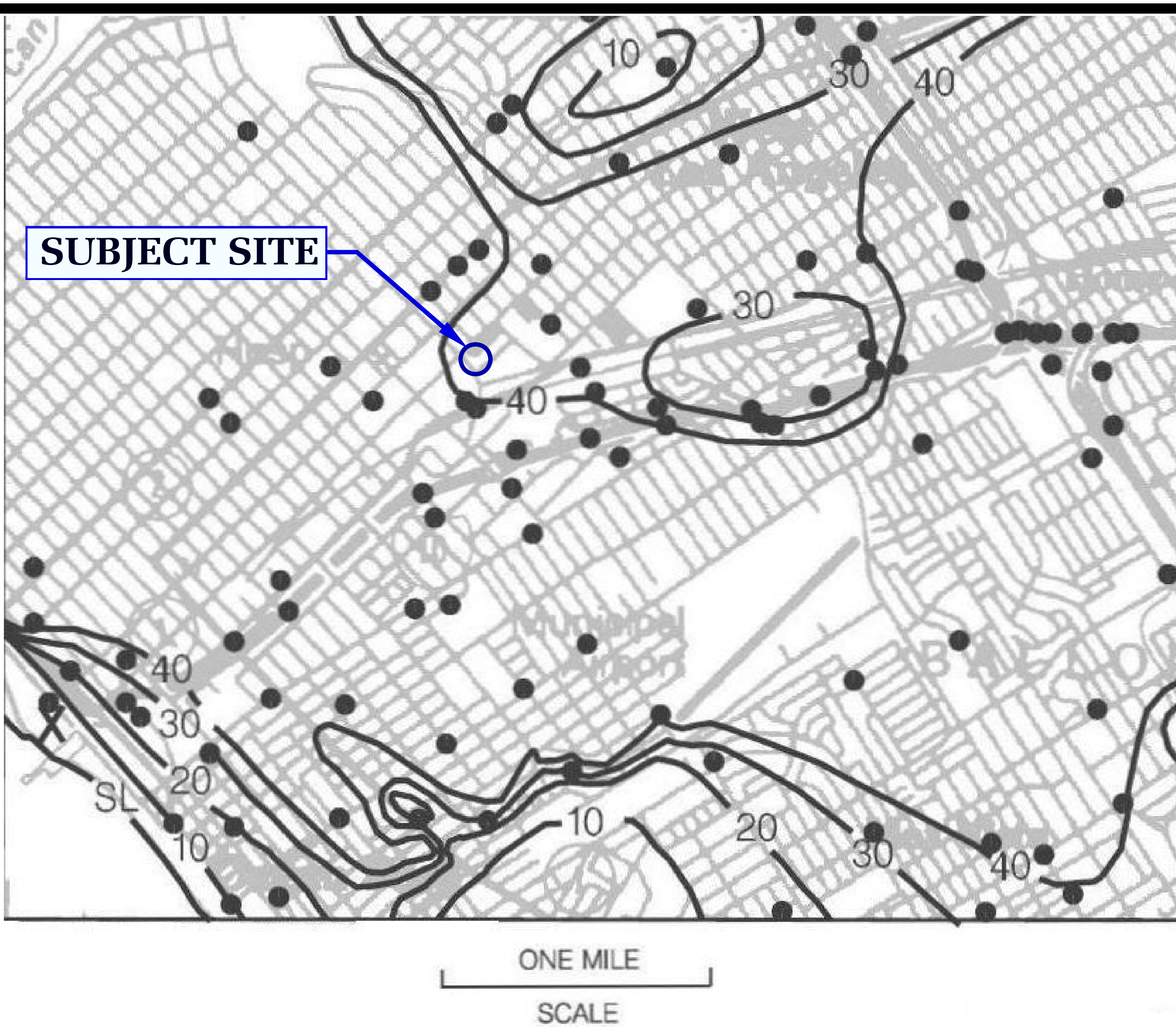
REFERENCE: U.S. DEPARTMENT OF THE INTERIOR, U.S. GEOLOGICAL SURVEY, PRELIMINARY GEOLOGIC MAP OF THE LOS ANGELES 30' X 60' QUADRANGLE, SOUTHERN CALIFORINA, VERSION 1.0, 2005, COMPILED BY ROBERT F. YERKES AND RUSSELL H. CAMPBELL.

REGIONAL GEOLOGIC MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION

FILE No. 21800

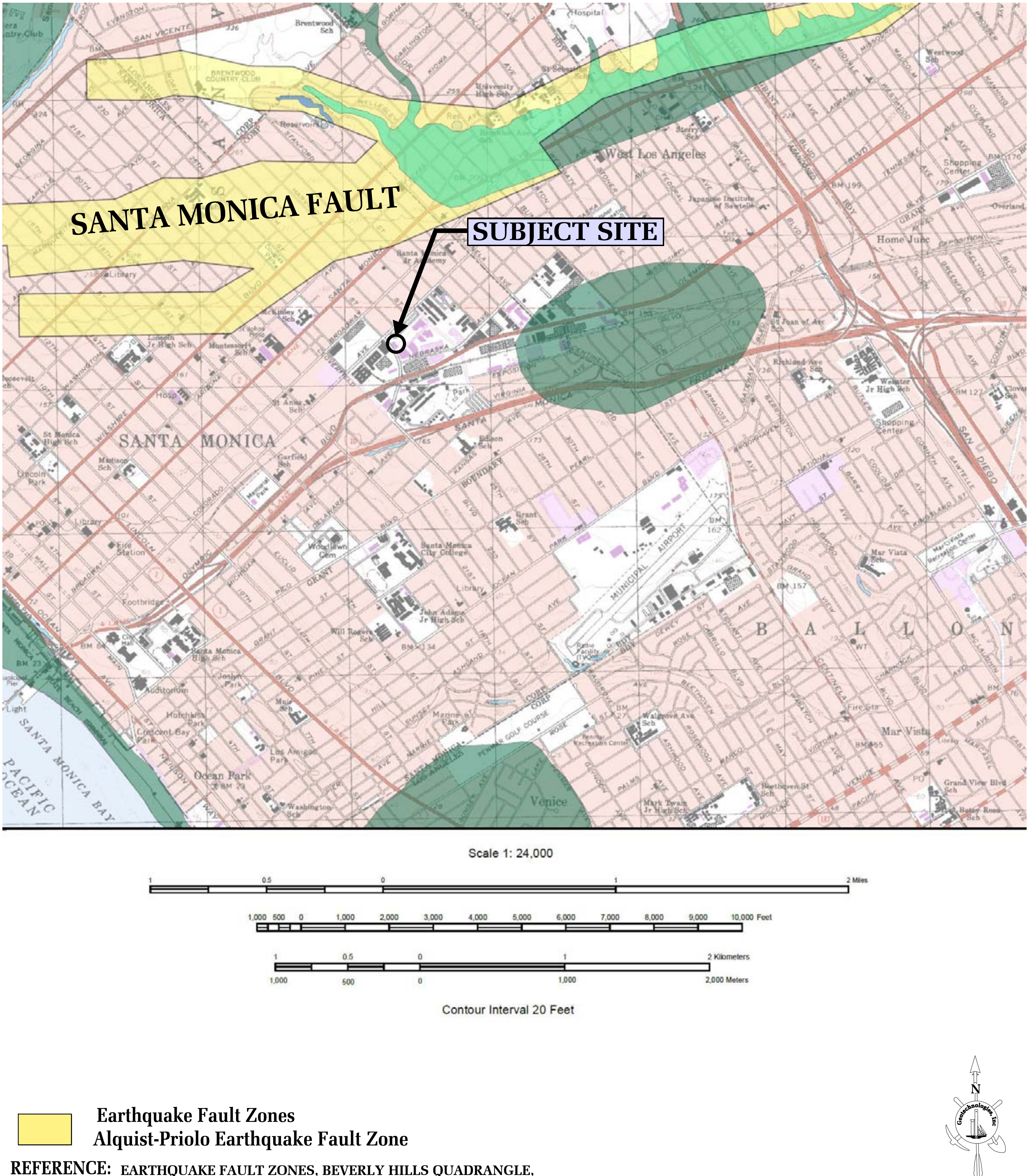


20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 023
BEVERLY HILLS 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2005)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc. Consulting Geotechnical Engineers	KILROY REALTY CORPORATION
	FILE No. 21800



REFERENCE: EARTHQUAKE FAULT ZONES, BEVERLY HILLS QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, JANUARY 2018

EARTHQUAKE FAULT ZONE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION

FILE NO. 21800

BORING LOG NUMBER 1

Kilroy Realty | **Date: 04/11/19** | **Elevation: 158'***

File No. 21800 | **Method: 8-inch diameter Hollow Stem Auger**

km

*Reference: City of Santa Monica Web based Topographic Map, 2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		5-inch Asphalt over 4-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt to Silty Clay, dark brown, moist, stiff
				2 --		
2.5	58	9.6	101.8	-		
				3 --		
				-	ML	ALLUVIUM: Sandy Silt, grayish brown, moist, stiff
				4 --		
				-		
5	44	10.4	96.7	5 --		
				-		dark brown
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	63	9.9	106.4	10 --		
				-		light yellowish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	68	9.4	107.4	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	52	11.4	106.4	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	72	13.4	123.7	25 --		
				-		dark grayish brown, moist, stiff

BORING LOG NUMBER 1

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	28/6" 50/2"	7.7	125.1	30 --	SM/SW	Silty Sand to Gravelly Sand, dark brown and bluish gray, very dense, fine to coarse grained, slate fragments to ½"
				31 --		
				32 --		
				33 --		
				34 --		
35	45/6" 50/5"	5.6	133.0	35 --		
				36 --		
				37 --		
				38 --		
				39 --		
40	67	5.5	113.3	40 --	SP	Sand, dark and gray, moist, dense, fine to coarse grained
				41 --		
				42 --		
				43 --		@ 43' water
				44 --		
45	58	24.2	104.8	45 --	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				46 --		
				47 --		
				48 --		
				49 --		
50	49	No Recovery		50 --		

BORING LOG NUMBER 1

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	100/8"	No Recovery		55 --		
				-		
				56 --		
				-		
				57 --		
				-		
				58 --	CL	Clay
				-		
				59 --		
				-		
60	54	No Recovery		60 --		
				-		
				61 --		
61.5	82	13.6	122.2	-		
				62 --	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, very stiff, few gravel to ¼"
				-		
				63 --		
				-		Total Depth 63 feet
				64 --		Water at 43 feet
				-		Fill to 1½ feet
				65 --		
				-		
				66 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				67 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				68 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 2

Kilroy Realty — **Date: 04/11/19** — **Elevation: 158.5'***

File No. 21800 — **Method: 8-inch diameter Hollow Stem Auger**

km — *Reference: City of Santa Monica Web based Topographic Map, 2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0		5-inch Asphalt over 4-inch Base
				1		
				2		FILL: Clayey Silt to Silty Clay, mottled dark brown and light brown, moist, stiff
2.5	72	20.2	108.0			
				3		
				4		
5	20	14.2	SPT	5		
				6		Sandy Silt, mottled dark and yellowish brown, minor brick and asphalt fragments
				7		
7.5	46	14.2	119.1			
				8		Sandy to Clayey Silt, mottled dark brown and gray, minor brick and glass fragments
				9		
10	34	13.0	SPT	10		
				11		Sandy Silt, dark brown and gray, moist, stiff, some brick, asphalt and glass fragments
				12		
12.5	100/8"	12.5	104.9			
				13		Sandy Silt, mottled dark gray and black, some brick and asphalt fragments
				14		
15	35	13.2	SPT	15		
				16		
				17		
17.5	68	15.1	103.0			
				18		
				19		
20	37	18.4	SPT	20		
				21		
				22		
22.5	48	15.2	112.4			
				23		
				24		
25	34	14.6	SPT	25		

BORING LOG NUMBER 2

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	72	14.7	110.9			
				28		Sandy Silt to Silty Clay, dark gray, moist, stiff, some brick and asphalt fragments
				29		
30	19	18.4	SPT	30		
				31		Silty Sand to Sandy Silt, dark gray, medium dense, fine grained, some brick metal and asphalt fragments to 2"
				32		
32.5	82	13.3	98.2			
				33		
				34		
35	36	13.9	SPT	35		
				36		
				37		
37.5	66	13.1	117.2			
				38		
				39		
40	75	13.9	SPT	40		
				41		
				42		
42.5	81	14.8	116.3			
				43		@ 43½' Water
				44	SM	ALLUVIUM: Silty Sand, dark brown and gray, moist, dense, fine grained, minor slate fragments to ½"
45	26	23.4	SPT	45		
				46	ML	Sandy to Clayey Silt, dark yellowish brown, very moist, stiff
				47		
47.5	49	25.2	104.5			
				48	ML/CL	Clayey Silt to Silty Clay, firm
				49		
50	28	14.2	SPT	50		
					CL	Silty clay, stiff, trace Sand

BORING LOG NUMBER 2

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	72	22.2	104.4	-		
				53 --		Clayey Silt to Silty Clay, moist, firm
				-		
				54 --		
				-		
55	38	18.6	SPT	55 --		
				-	ML	Sandy to Clayey Silt, moist, firm, stiff
				56 --		
				-		
				57 --		
57.5	77	22.2	105.1	-		
				58 --		stiff
				-		
				59 --		
				-		
60	35	19.2	SPT	60 --		
				-	CL	Silty Clay, dark brown, moist, stiff
				61 --		
				-		
				62 --		
62.5	78	9.9	127.7	-		
				63 --	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, very dense, fine grained, very stiff, minor slate gravel to ½"
				-		
				64 --		
				-		
65	45	15.8	SPT	65 --		
				-	CL	Sandy Clay to Silty Clay, dark grayish brown, moist to wet, dense, fine to coarse grained
				66 --		
				-		
				67 --		
67.5	56	17.9	114.5	-		
				68 --	ML	Sandy Silt, dark brown, moist, stiff
				-		
				69 --		
				-		
70	29	20.8	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72.5	81	20.8	109.5	-		
				73 --	CL	Silty Clay
				-		
				74 --		
				-		
75	39	13.8	SPT	75 --		
				-	SM	Silty Sand to Clayey Sand, dark brown, moist to wet, medium dense, fine to medium grained, stiff

BORING LOG NUMBER 2

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
77.5	88	9.5	126.6			
				78		Silty Sand, dark grayish brown, wet, very dense, fine grained, gravel to 3/8", very stiff
				79		
80	46	14.1	SPT	80		
				81		
				82		
82.5	40/6" 50/5"	12.5	125.8			
				83		
				84		
85	72	10.5	SPT	85		
					SP/SW	Sand to Gravelly Sand, dark brown to dark gray, wet, dense, fine to coarse grained
				86		
				87		
87.5	30/6" 50/4"	11.9	125.2			
				88	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, very dense, fine to medium grained, very stiff
				89		
90	85	14.1	SPT	90		
					SM/SP	Silty Sand to Sand, moist to wet, very dense, fine to medium grained
				91		
				92		
92.5	40/6" 50/5"	17.5	119.7			
				93	SM/ML	Silty Sand to Sandy Silt, dark and gray, moist, very dense, fine grained, very stiff, gravel to ½" (slate)
				94		
95	74	10.1	SPT	95		
					SP/SW	Sand to Gravelly Sand, dark brown and gray, wet, dense, fine to coarse grained
				96		
				97		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
97.5	92	8.7	125.5			Used 8-inch diameter Hollow-Stem Auger
				98		140-lb. Automatic Hammer, 30-inch drop
				99		Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
100	36	10.4	SPT	100		
						Total Depth 100 feet Water at 43½ feet Fill to 43½ feet

BORING LOG NUMBER 3

Kilroy Realty | **Date: 04/15/19** | **Elevation: 158'***

File No. 21800 | **Method: 8-inch diameter Hollow Stem Auger**

km | *Reference: City of Santa Monica Web based Topographic Map, 2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		4-inch Asphalt over 3-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt, mottled black and yellowish brown, moist,
				2 --		stiff, with gravel to ¼"
2.5	70	24.6	100.5	-		
				3 --		
				-		
				4 --		
				-		
5	78	11.5	124.0	5 --		
				-		Sandy Silt, mottled black and dark brown, some brick
				6 --		fragments, gravel to 1" (slate)
				-		
				7 --		
7.5	78	10.1	122.1	-		
				8 --		
				-		
				9 --		
				-		
10	81	13.3	119.5	10 --		
				-		dark gray, dense, stiff, minor plastic fragments
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	88	7.3	106.7	15 --		
				-		very stiff, some brick fragments
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	35/6" 50/5"	10.3	114.8	20 --		
				-		mottled dark gray and black, very stiff, with brick and concrete
				21 --		fragments
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	68	16.5	116.4	25 --		
				-	ML	ALLUVIUM: Sandy Silt, dark yellowish brown, moist, stiff

BORING LOG NUMBER 3

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		Sandy Silt, dark yellowish brown, moist, stiff
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	69	9.0	130.3	30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	92	5.0	130.2	35 --		
				-	SW	Gravelly Sand with trace Silt, dark brown and gray, very dense, fine to coarse grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	58	15.6	112.6	40 --		
				-	ML	Sandy Silt trace Clay, dark brown and gray, stiff
				41 --		
				-		
				42 --		
				-		
				43 --		@ 43' Water
				-		
				44 --		
				-		
45	63	15.0	113.8	45 --		
				-	SP/ML	Sand to Sandy Silt, dark yellowish brown, wet, dense, fine grained, stiff
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	76	18.2	113.6	50 --		
				-	ML	Sandy Silt, moist, stiff

BORING LOG NUMBER 3

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
				53 --		
				54 --		
55	63	16.3	116.3	55 --		Sandy Silt, moist, stiff
				56 --		
				57 --		
				58 --		
				59 --		
60	68	18.2	114.4	60 --		Total Depth 60 feet Water at 43 feet Fill to 25 feet
				61 --		
				62 --		
				63 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				64 --		
				65 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				66 --		
				67 --		
				68 --		
				69 --		
				70 --		
				71 --		
				72 --		
				73 --		
				74 --		
				75 --		

BORING LOG NUMBER 4

Kilroy Realty — **Date: 04/22/19** — **Elevation: 158.5'***

File No. 21800 — **Method: 8-inch diameter Hollow Stem Auger**

km — *Reference: City of Santa Monica Web based Topographic Map, 2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		3-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Silty to Sandy Clay, mottled light brownish gray and
				-		yellowish brown, moist, medium dense, fine grained
				2 --		
2.5	13	21.0	100.1	-		
				3 --		
				-		
				4 --		
				-		
5	56	9.7	122.7	5 --		— — — — — — — — — — —
				-		mottled light brownish gray and black
				6 --		
				-		
				7 --		— — — — — — — — — — —
7.5	23	16.0	112.6	-		Sandy Silt, grayish brown, dense, fine to coarse grained
				8 --		
				-		
				9 --		
				-		
10	25	13.8	114.9	10 --		— — — — — — — — — — —
				-		mottled dark gray and reddish brown
				11 --		
				-		
				12 --		— — — — — — — — — — —
12.5	47	15.5	117.6	-		dark gray to black, very dense, glass and brick fragments
				13 --		
				-		
				14 --		
				-		
15	48	12.3	114.7	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	50/4"	14.4	90.5	-		
				18 --		
				-		
				19 --		
				-		
20	40/6" 50/3"	No Recovery		20 --		
				-		
				21 --		
				-		
				22 --		
22.5	58	21.7	98.0	-		
				23 --		
				-		
				24 --		
				-		
25	80	11.3	117.1	25 --		@ 25' pieces of wood to 2"
				-		

BORING LOG NUMBER 4

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	55	No Recovery				
				28		
				29		
30	38/6" 50/4"	12.2	111.8	30		@ 30' piece of concrete to 2"
				31		
				32		
32.5	30	17.5	109.3			
				33		
				34		
35	33	12.4	119.7	35		yellowish brown, piece of concrete to 1"
				36		
				37		
37.5	46	10.2	125.9			
				38		
				39	SM/SP	ALLUVIUM: Silty Sand to Sand, grayish brown, moist, dense
40	26	17.6	110.7	40		
				41		
					CL	Silty to Sandy Clay, grayish brown, moist, firm, fine grained
				42		
42.5	84	8.3	129.5			
				43	SP/SW	Sand to Gravelly Sand, grayish brown, very dense, fine coarse grained, gravel to ¾" (slate)
				44		
					ML	Silt and Silty Clay, reddish brown, wet, firm, fine grained
45	27	23.1	101.8	45		
				46		
				47		
				48		
				49		
50	28/6" 50/5"	21.7	109.5	50		
					SM	Silty Sand, brown, moist, very dense, fine to medium grained

BORING LOG NUMBER 4

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		@ 53½' Water
				54 --		
				-		
55	87	18.9	113.5	55 --		
				-	CL	Silty to Sandy Clay, dark reddish brown, very moist, stiff, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	40	17.5	116.5	60 --		
				-		Total Depth 60 feet
				61 --		Water at 53½ feet
				-		Fill to 40 feet
				62 --		
				-		
				63 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				64 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				65 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 5

Kilroy Realty — **Date: 04/15/19** — **Elevation: 157'***

File No. 21800 — **Method: 8-inch diameter Hollow Stem Auger**

km — *Reference: City of Santa Monica Web based Topographic Map, 2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		4-inch Asphalt over 3-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt with gravel, dark brown and bluish gray,
				2 --		moist, stiff, gravel to ½"
2.5	61	11.0	123.5	-		
				3 --		Sandy Silt, dark brown, moist, stiff, gravel to 1" (slate)
				-		
				4 --		
				-		
5	28	12.9	SPT	5 --		
				-		Sandy Silt, dark brown, moist, stiff, minor brick fragments
				6 --		
				-		
				7 --		
7.5	61	11.1	105.9	-		
				8 --	ML	ALLUVIUM: Silt, light grayish brown, moist, stiff
				-		
				9 --		
				-		
10	21	11.2	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	63	9.9	113.6	-		
				13 --		
				-		
				14 --		
				-		
15	27	9.5	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	72	11.5	112.7	-		
				18 --		dark yellowish brown
				-		
				19 --		
				-		
20	24	11.6	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	84	14.6	116.6	-		
				23 --		Clayey Silt, brown, porous
				-		
				24 --		
				-		
25	30	12.4	SPT	25 --		
				-		

BORING LOG NUMBER 5

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	81	6.9	125.6	-		
				28 --	SM	Silty Sand with gravel, dark brown, moist, dense, fine grained, gravel to ½" (slate)
				-		
				29 --		
				-		
30	54	5.2	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	64	6.8	102.0	-		
				33 --	SP/SM	Sand to Silty Sand, dark and gray, fine grained, minor slate fragments
				-		
				34 --		
				-		
35	18	14.5	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	78	9.2	130.3	-		
				38 --	SM/SC	Clayey and Silty Sand, dark brown, moist, very dense, fine grained, minor gravel fragments to ½" (slate)
				-		
				39 --		
				-		
40	20	14.5	SPT	40 --		
				-	SP/SM	Sand to Silty Sand, dark brown, moist, medium dense, fine grained, stiff
				41 --		
				-		
				42 --		
42.5	44	25.2	98.8	-		
				43 --	CL	Silty Clay, brown, moist, firm
				-		
				44 --		
				-		
45	9	27.3	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	72	16.4	116.4	-		
				48 --	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff, minor gravel fragments up to ½" (slate)
				-		
				49 --		
				-		
50	30	14.0	SPT	50 --		
				-	SM/ML	Silty Sand to Sandy Silt, very moist, medium dense, fine grained, stiff

BORING LOG NUMBER 5

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	41	24.8	103.0	-		
				53 --	CL	Silt to Silty Clay, dark reddish brown, moist, firm
				-		
				54 --		
				-		
55	28	14.4	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	79	12.4	125.8	-		
				58 --	SM	Silty Sand, dark brown, moist, very dense, fine grained, gravel to ½" (slate)
				-		
				59 --		
				-		
60	24	17.6	SPT	60 --		
				-	ML	Sandy Silt, stiff
				61 --		
				-		
				62 --		
62.5	72	20.8	107.5	-		
				63 --	ML/CL	Clayey Silt to Silty Clay
				-		
				64 --		
				-		
65	44	11.9	SPT	65 --		
				-	SM/ML	Silty Sand to Sandy Silt, wet, medium dense, fine grained, stiff
				66 --		
				-		
				67 --		
67.5	85	12.6	119.1	-		
				68 --	ML	Sandy Silt, dark brown, moist, very stiff
				-		
				69 --		
				-		
70	36	12.1	SPT	70 --		
				-	SM/ML	Silty Sand to Sandy Silt, medium dense, fine grained, stiff
				71 --		
				-		
				72 --		
72.5	78	14.9	118.1	-		
				73 --	ML/CL	Sandy Silt to Silty Clay, stiff
				-		
				74 --		
				-		
75	34	16.4	SPT	75 --		
				-	SM/ML	Silty Sand to Sandy Silt, medium dense, fine grained, stiff

BORING LOG NUMBER 5

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
77.5	88	12.6	117.8			
				78	SP/CL	Sand to Silty Clay, very dense, fine grained, very stiff
				79		
80	66	14.1	SPT	80		
					SM/ML	Silty Sand to Clayey Silt, dense, fine grained, stiff
				81		
				82		
82.5	88	14.7	118.9			
				83		dark brown and gray, wet, very dense, fine grained, very stiff, with gravel fragments
				84		
85	68	20.4	SPT	85		
					ML/CL	Clayey Silt to Silty Clay, dark brown, stiff
				86		
				87		
87.5	45/6" 50/4"	No Recovery				
				88		
				89		
90	69	15.6	SPT	90		
					SM	Silty Sand with gravel, gray, vey dense, fine grained, gravel to ¼" (slate)
				91		
				92		
92.5	40/6" 50/3"	11.7	122.3			
				93		
				94		
95	48	11.3	SPT	95		
				96		
				97		
97.5	45/6" 50/2"	11.4	127.9			
				98		
				99		Sandy to Clayey Silt to Silty Sand, very dense, fine grained, very stiff
					ML/SM	
100	79	15.0	SPT	100		
						Total Depth 100 feet Water at 43½ feet Fill to 7 feet

NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.

Used 8-inch diameter Hollow-Stem Auger
140-lb. Automatic Hammer, 30-inch drop
Modified California Sampler used unless otherwise noted
SPT=Standard Penetration Test

BORING LOG NUMBER 6

Kilroy Realty | **Date: 04/23/19** | **Elevation: 157'***

File No. 21800 | **Method: 8-inch diameter Hollow Stem Auger**

km

*Reference: City of Santa Monica Web based Topographic Map, 2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		3-inch Asphalt over 4-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt to Silty Sand, grayish reddish brown, slightly moist, dense, stiff, fine to medium grained, few gravel to ½"
2	48	17.0	112.0	2 --		
				-		
				3 --		
				-		
4	77	11.7	118.2	4 --		
				-		Silty Sand, mottled yellowish and brownish gray, moist, few gravel to ½"
				5 --		
				-		
6	46	15.4	115.7	6 --		
				-		Sandy Silt, mottled dark reddish brown to very dark gray, moist, stiff, gravel to ½"
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	30/6" 50/5"	10.5	117.4	10 --		
				-		brick fragments up to ¾", metal fragments, torn pieces of rubber
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	86	13.0	114.4	15 --		
				-		very large brick fragments up to 3"
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	48/6" 50/2"	12.2	110.7	20 --		
				-		brick fragments, considerable amount of rock fragments up to 1"
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	32/6" 50/5"	14.1	111.4	25 --		
				-		black, very stiff, brick fragments, few slate fragments

BORING LOG NUMBER 6

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	72	13.2	119.8	30 --		
				31 --		Sandy to Silty Clay, very dark gray to medium brown, moist, stiff, wire
				32 --		
				33 --		
				34 --		
35	41	13.7	115.9	35 --	ML/SM	ALLUVIUM: Sandy Silt, medium to dark grayish brown, moist, stiff, dense, very fine to medium grained, few gravel fragments to ¼" (slate)
				36 --		
				37 --		
				38 --		
				39 --		
40	41	23.3	99.6	40 --	CL	Silty Clay, dark grayish brown to olive gray, moist, firm
				41 --		
				42 --		
				43 --		
				44 --		
45	72	24.0	102.7	45 --	SM/SP	Silty Sand to Sand, medium to reddish brown, moist to very moist, stiff, dense, very fine to medium grained
				46 --		
				47 --		@ 45½' Water
				48 --		
				49 --		
50	75	14.2	114.2	50 --	SM/SP	Silty Sand to Sand, very dark grayish brown to reddish brown, wet, dense, stiff, fine to medium grained

BORING LOG NUMBER 6

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
				53 --		
				54 --		
55	43	14.3	120.2	55 --		
				-	CL	Sandy Clay, medium to reddish brown, wet, stiff, gravel to ½" (slate)
				56 --		
				57 --		
				58 --		
				59 --		Silty Sand, dark brown, dense, some clay, fine to medium grained
				-	SM	
60	69	18.3	108.7	60 --		
				-		Total Depth 60 feet Water at 45½ feet Fill to 34 feet
				61 --		
				62 --		
				63 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				64 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				65 --		
				66 --		
				67 --		
				68 --		
				69 --		
				70 --		
				71 --		
				72 --		
				73 --		
				74 --		
				75 --		

BORING LOG NUMBER 7

Kilroy Realty **Date: 04/23/19** **Elevation: 157.5'***

File No. 21800 **Method: 8-inch diameter Hollow Stem Auger**

km *Reference: City of Santa Monica Web based Topographic Map, 2019

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		3-inch Asphalt over 3-inch Base
1	24	12.7	106.4	1 --		FILL: Clayey Sand to Sandy Clay, mottled dark olive brown to very dark grayish brown, stiff, dense, fine to medium grained, some asphalt fragments to ½"
				2 --		
3	36	11.9	121.2	3 --		Silty Sand, brown, fine to medium grained, trace fragments of clay, gravel up to 2"
				4 --		
5	28	17.3	112.5	5 --		Clayey Sand, mottled medium brown, fine to medium grained, gravel to 1"
				6 --		
7	22	16.9	105.6	7 --		Sandy Clay to Clayey Sand, mottled medium to dark brown, medium stiff, medium dense
				8 --		
				9 --		
10	44	9.8	117.9	10 --		Silty Sand, very dark brown to black, dense, fine to medium grained, large amount of slate, brick fragments
				11 --		
				12 --		
12.5	23	11.0	104.8	13 --		Sandy Clay, very dark gray to black, stiff, brick fragments, gravel
				14 --		
15	29	17.5	111.3	15 --		@ 15' brick fragments up to 1"
				16 --		
				17 --		@ 17½' metal wires
17.5	34	14.7	102.8	18 --		
				19 --		
20	45	12.8	117.4	20 --		
				21 --		
				22 --		
				23 --		
				24 --		
25	32/6" 50/5"	12.2	118.0	25 --		@ 25' mixture of debris (brick and porcelain fragments), slate fragments

BORING LOG NUMBER 7

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	35/6" 50/5"	10.2	125.0	30		Silty to Clayey Sand, mottled black, moist, dense, brick fragments, concrete pieces up to ¾", gravel to 1"
				31		
				32		
				33		
				34		
35	38/6" 50/5"	9.1	115.0	35		
				36		
				37		
				38		
				39		@ 39½' considerable amount of rock fragments up to 1"
40	50/6"	No Recovery		40		
				41		
				42		
				43	ML/CL	ALLUVIUM: Silty Clay to Clayey Silt, brown, moist, stiff, some gravel to ½"
				44		
45	21/6" 50/6"	16.9	116.6	45		
				46		@ 46' water
				47		
				48		
				49		
50	54	20.3	110.0	50		

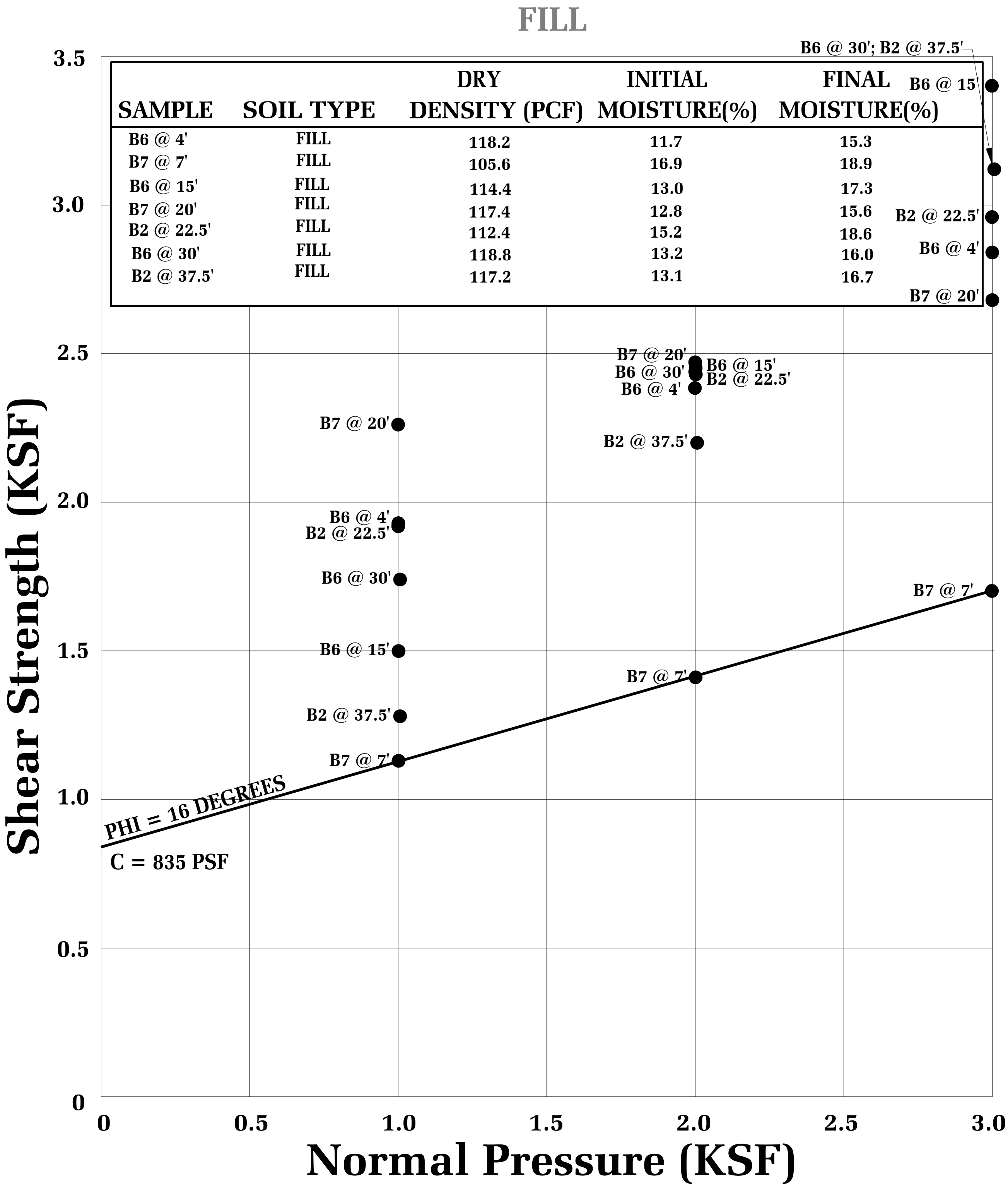
BORING LOG NUMBER 7

Kilroy Realty

File No. 21800

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-	ML/CL	ALLUVIUM: Silty Clay to Clayey Silt, brown, moist, stiff, some
				51 --		gravel to ½"
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	63	15.2	116.5	55 --		
				-		
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	58	20.2	107.9	60 --		
				-		Total Depth 60 feet
				61 --		Water at 46 feet
				-		Fill to 42 feet
				62 --		
				-		
				63 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				64 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				65 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B6 @ 4'	FILL	118.2	11.7	15.3
B7 @ 7'	FILL	105.6	16.9	18.9
B6 @ 15'	FILL	114.4	13.0	17.3
B7 @ 20'	FILL	117.4	12.8	15.6
B2 @ 22.5'	FILL	112.4	15.2	18.6
B6 @ 30'	FILL	118.8	13.2	16.0
B2 @ 37.5'	FILL	117.2	13.1	16.7

● Direct Shear, Saturated

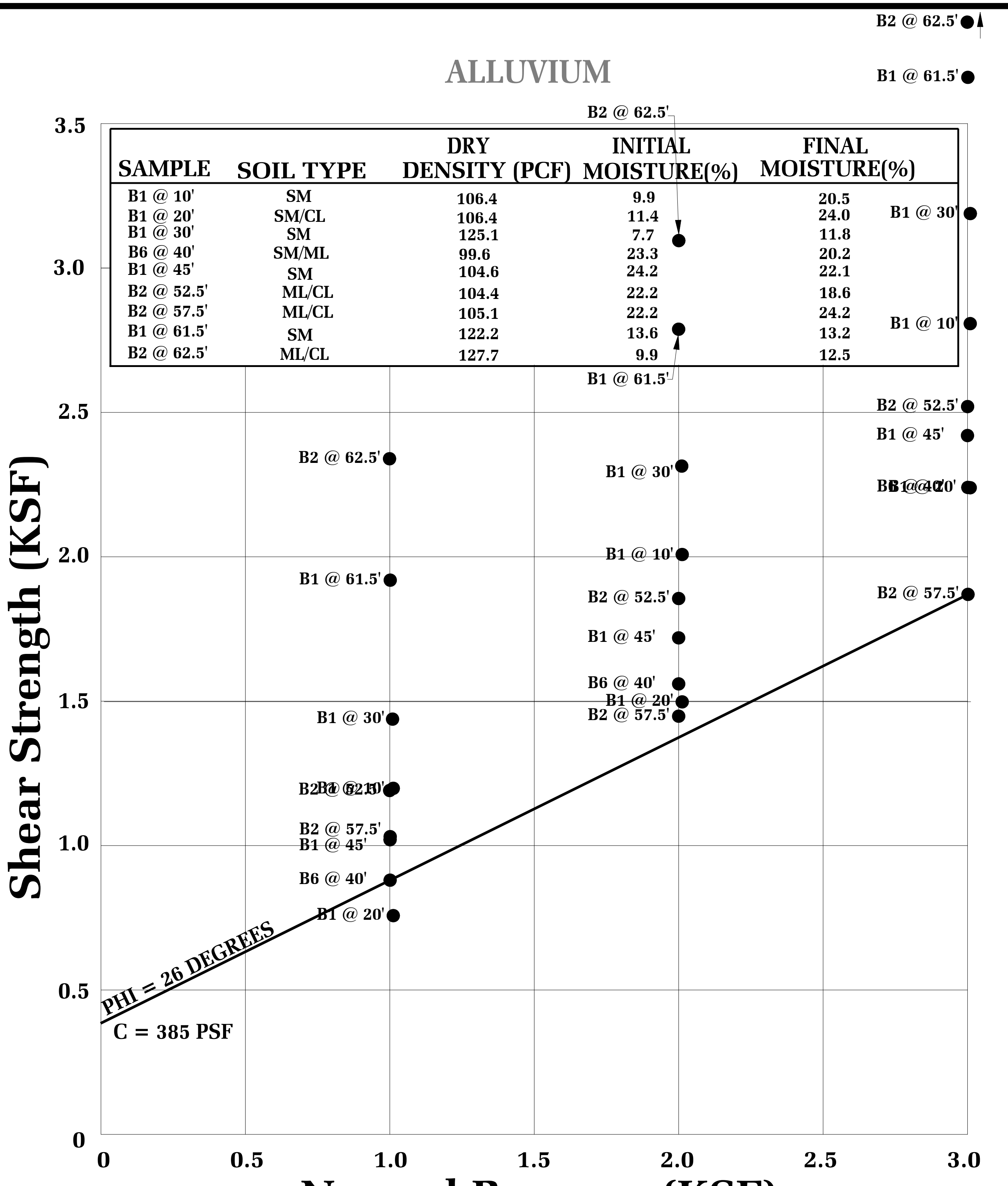
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 | **PLATE: B-1**

ALLUVIUM



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 10'	SM	106.4	9.9	20.5
B1 @ 20'	SM/CL	106.4	11.4	24.0
B1 @ 30'	SM	125.1	7.7	11.8
B6 @ 40'	SM/ML	99.6	23.3	20.2
B1 @ 45'	SM	104.6	24.2	22.1
B2 @ 52.5'	ML/CL	104.4	22.2	18.6
B2 @ 57.5'	ML/CL	105.1	22.2	24.2
B1 @ 61.5'	SM	122.2	13.6	13.2
B2 @ 62.5'	ML/CL	127.7	9.9	12.5

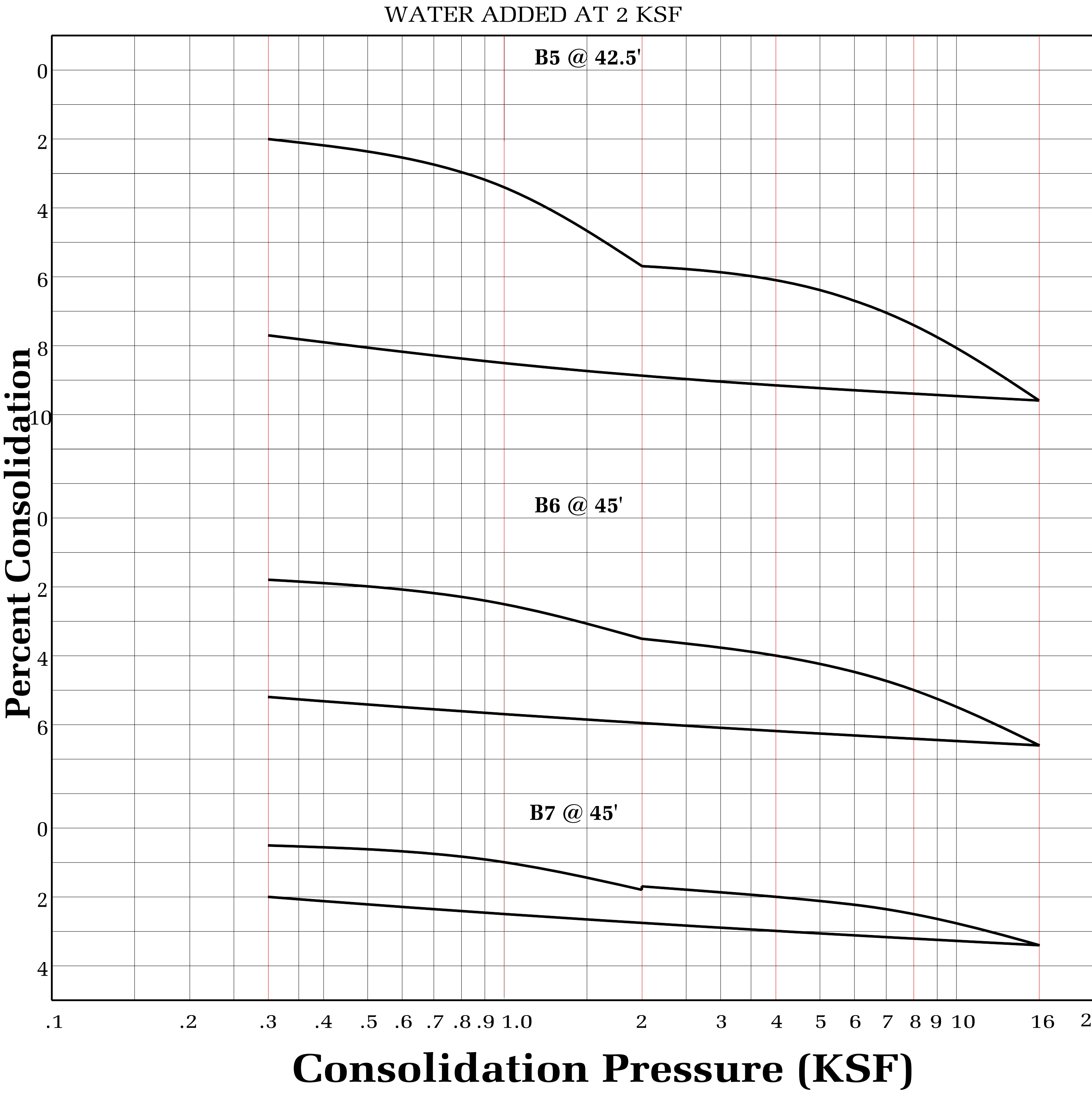
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 | **PLATE: B-2**

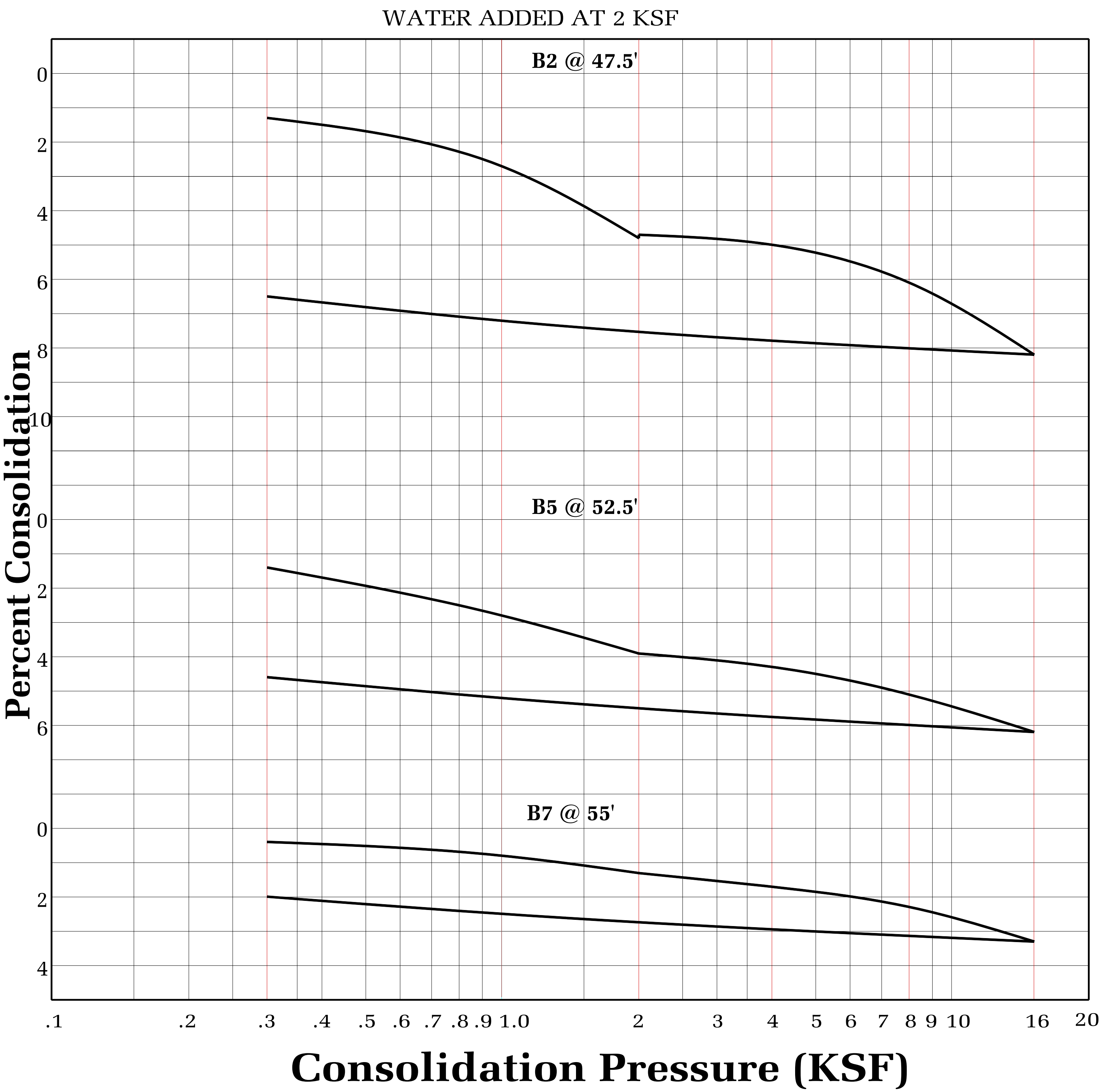


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 **PLATE: C-1**



CONSOLIDATION TEST

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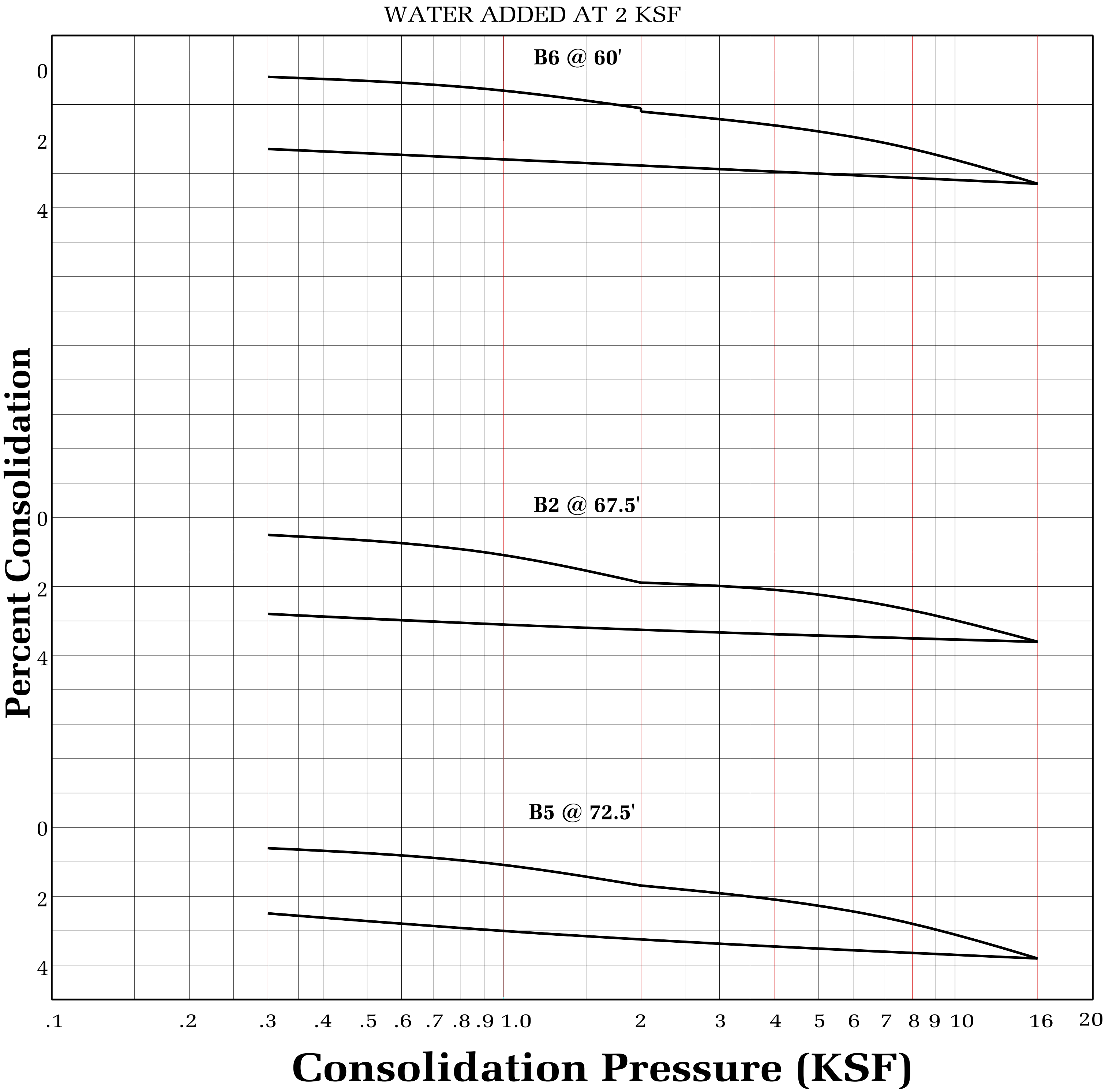
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800	**PLATE: C-2**

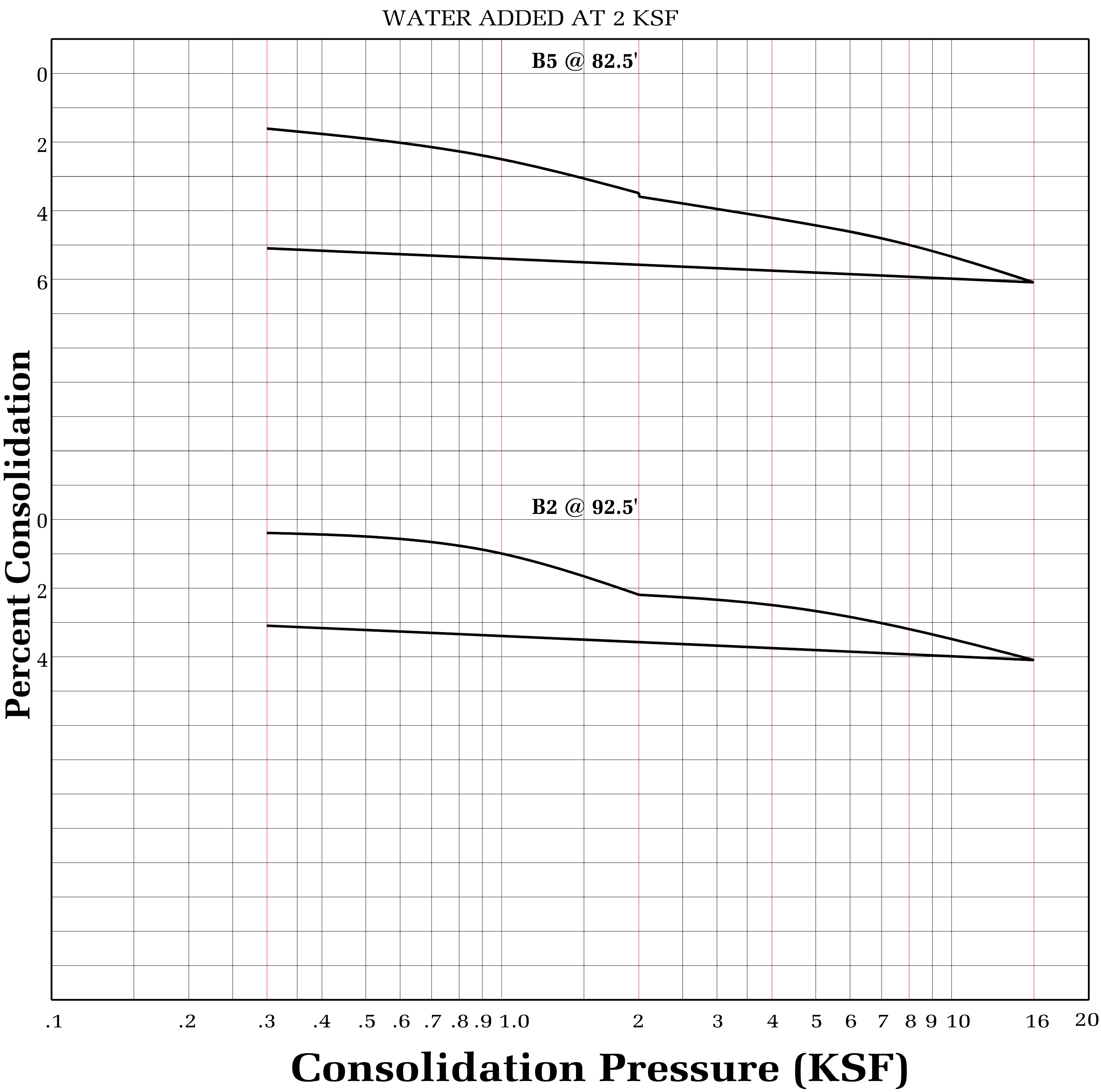


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 | **PLATE: C-3**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800	PLATE: C-4

ASTM D-1557

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/CL
MAXIMUM DENSITY pcf.	129.1
OPTIMUM MOISTURE %	9.6

ASTM D 4829

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/CL
EXPANSION INDEX UBC STANDARD 18-2	67
EXPANSION CHARACTER	MODERATE

SULFATE CONTENT

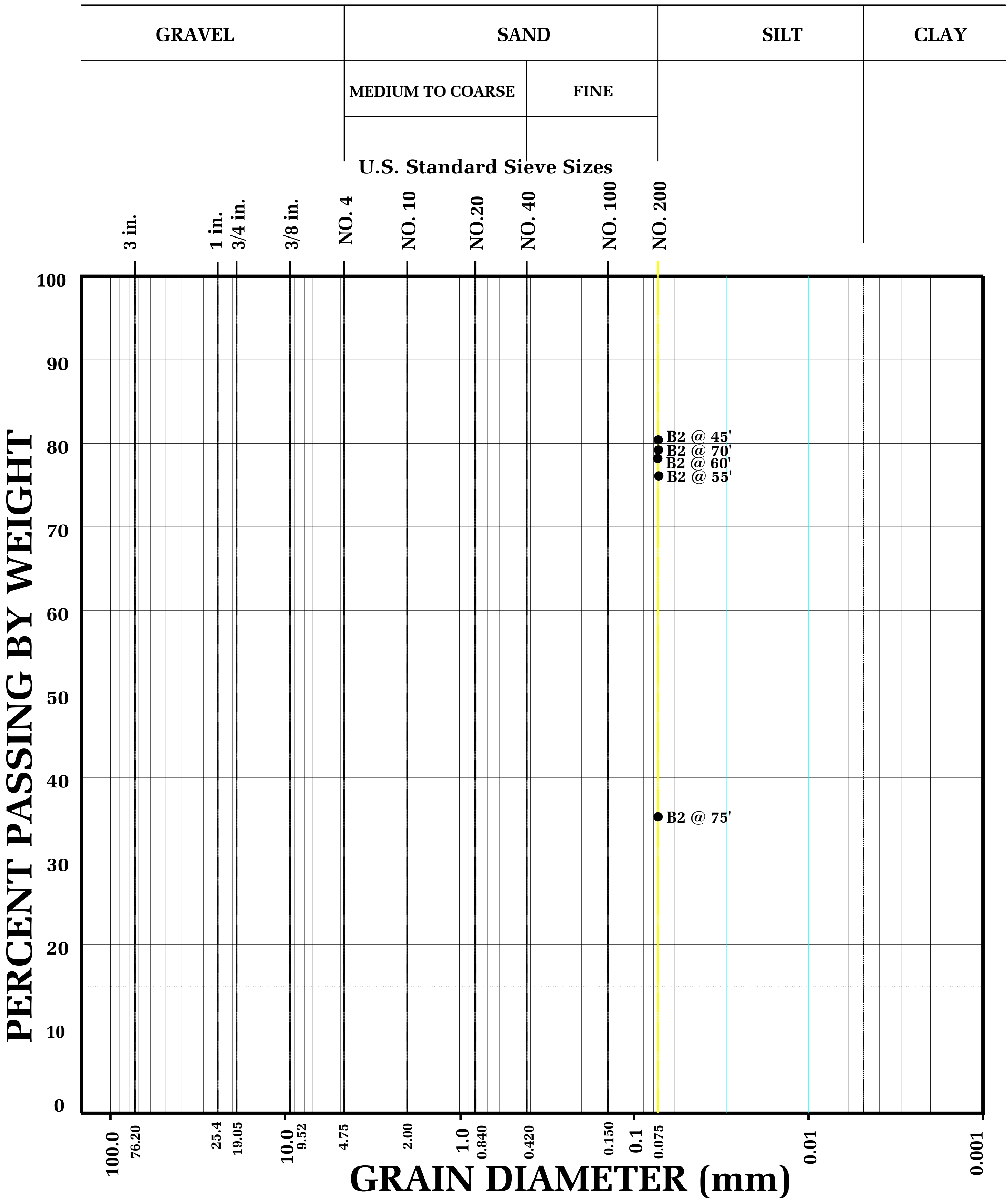
SAMPLE	B1 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 | PLATE: D

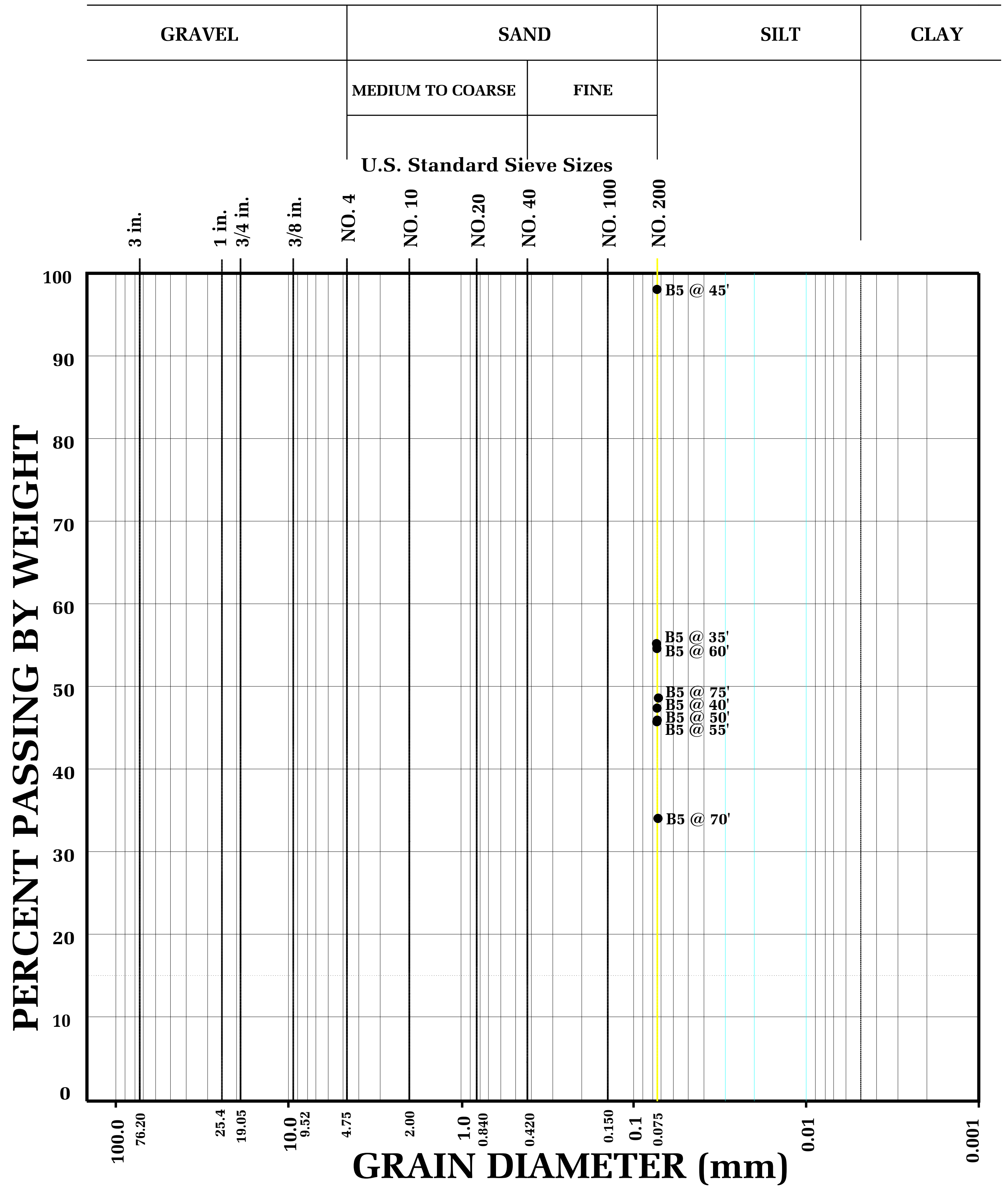


GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 | **PLATE: E-1**

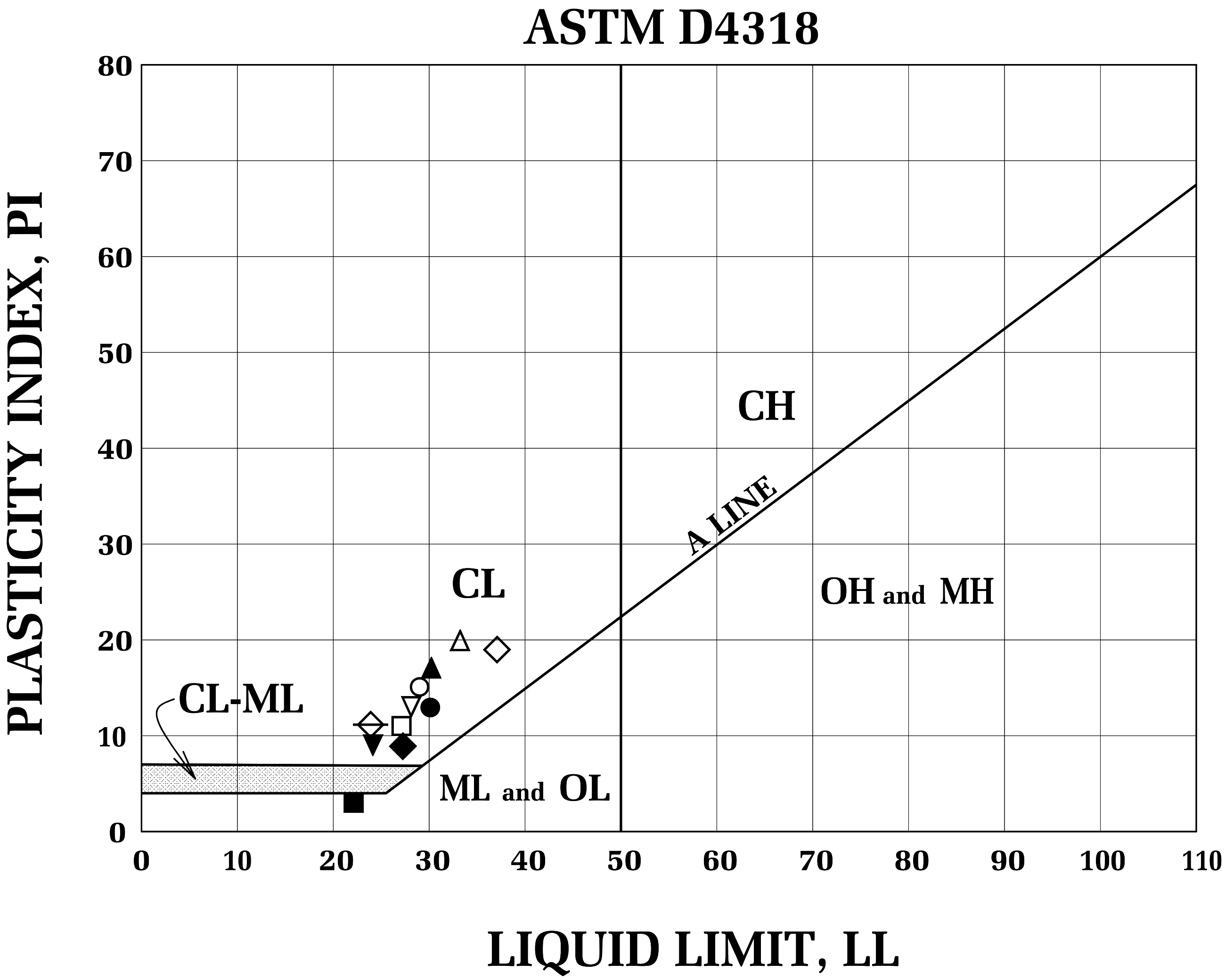


GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 | **PLATE: E-2**



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B2	45	○	29	14	15	CL
B2	55	●	30	17	13	CL
B2	60	△	33	13	20	CL
B2	70	▲	30	13	17	CL
B5	35	□	27	16	11	CL
B5	40	■	22	19	3	CL
B5	45	◇	37	19	19	CL
B5	50	◆	27	18	9	CL
B5	55	▽	28	15	13	CL
B5	60	▼	24	15	9	CL
B5	75	⯑	25	14	11	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

KILROY REALTY CORPORATION
1633 26TH ST., SANTA MONICA

FILE NO. 21800 | **PLATE: F**

OSHPD

21800

1633 26th St, Santa Monica, CA 90404, USA

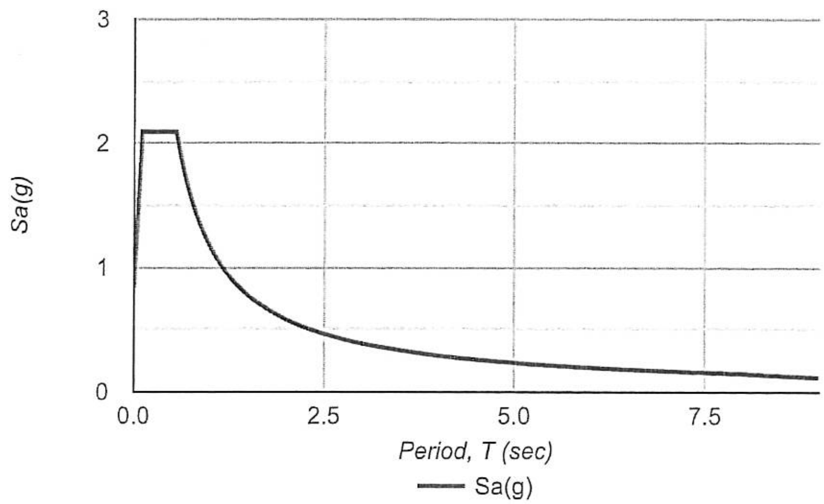
Latitude, Longitude: 34.0302081, -118.4702919

Date	5/6/2019, 1:33:07 PM
Design Code Reference Document	IBC-2015
Risk Category	III
Site Class	D - Stiff Soil

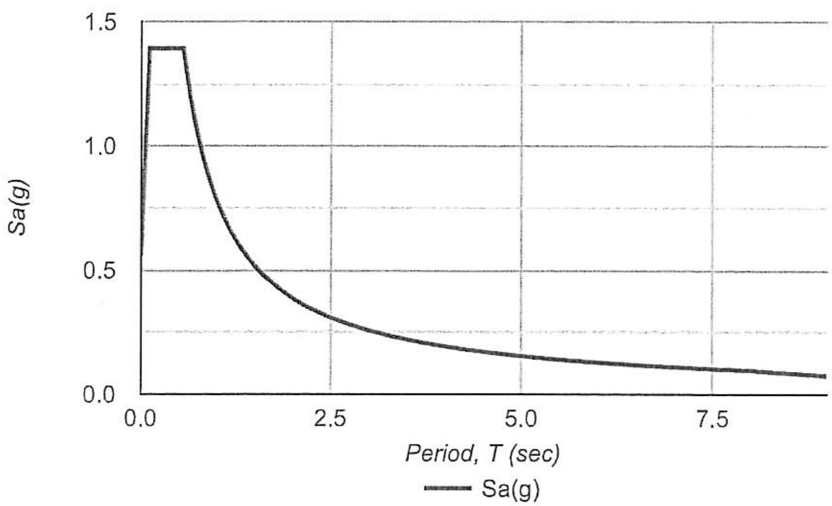
Type	Value	Description
S_S	2.09	MCE_R ground motion. (for 0.2 second period)
S_1	0.774	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.09	Site-modified spectral acceleration value
S_{M1}	1.162	Site-modified spectral acceleration value
S_{DS}	1.393	Numeric seismic design value at 0.2 second SA
S_{D1}	0.774	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.799	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.799	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.09	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.195	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.732	Factored deterministic acceleration value. (0.2 second)
S1RT	0.774	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.814	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.153	Factored deterministic acceleration value. (1.0 second)
PGAd	1.046	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.952	Mapped value of the risk coefficient at short periods

MCER Response Spectrum



Design Response Spectrum



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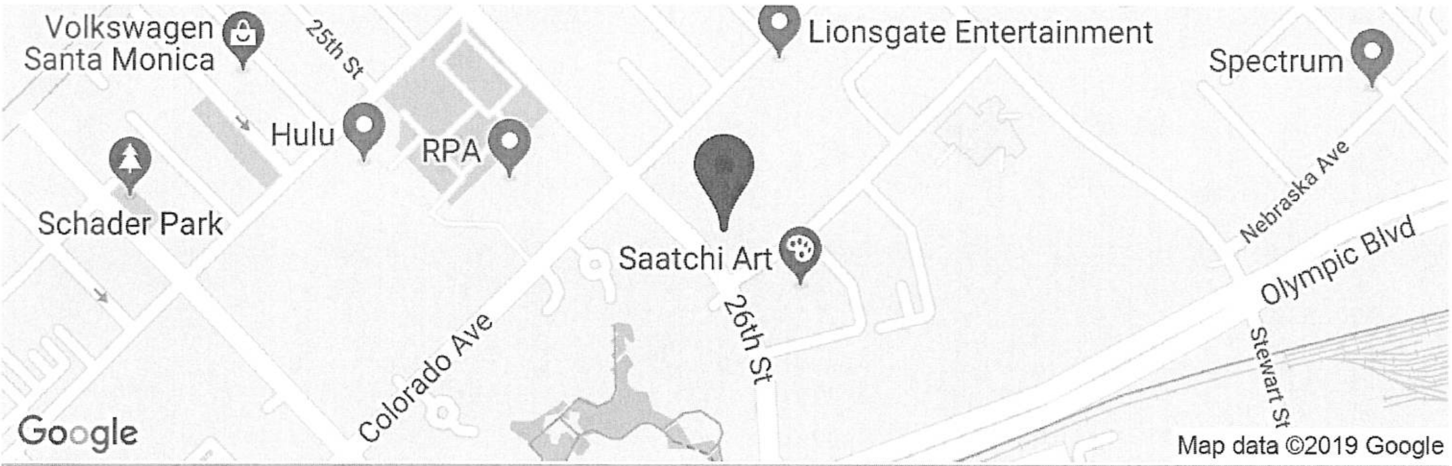
OSHPD

21800

1633 26th St, Santa Monica, CA 90404, USA

Latitude, Longitude: 34.0302081, -118.4702919



Date	5/6/2019, 1:33:07 PM
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PGAd	1.046	Factored deterministic acceleration value. (Peak Ground Acceleration)
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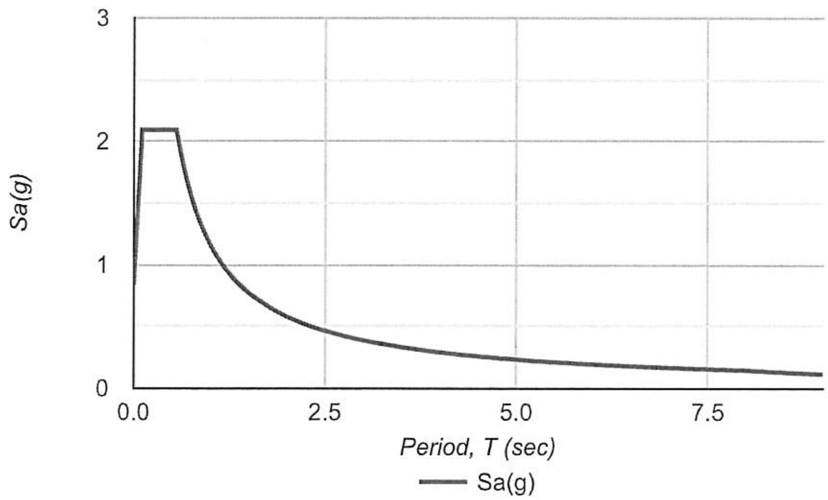
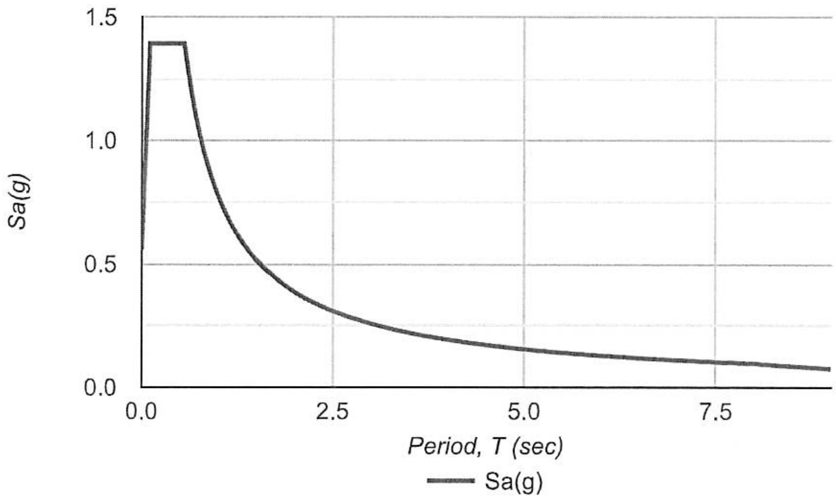
MCER Response Spectrum



Design Response Spectrum



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Geotechnologies, Inc.

Project: Kilroy Realty
File No.: 21800
Description: Liquefaction Analysis - Maximum Considered Earthquake (2% Probability of Exceedance in 50 years)
Boring Number: 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.799
Calculated Mag.Wtg.Factor:	1.203

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	43.5
Historically Highest Groundwater Level* (ft):	38.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

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

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vo} (psf)	Effective Vert. Stress σ_{vo}' (psf)	Fines Corrected $(N_1)_{60cs}$	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	120.0	Unsaturated	Unsaturated	20	5	0.0	0	120.0	120.0	47.7	1.00	0.522	2.000	Non-Liq.	0.00
2	120.0	Unsaturated	Unsaturated	20	5	0.0	0	240.0	240.0	47.7	1.00	0.520	2.000	Non-Liq.	0.00
3	120.0	Unsaturated	Unsaturated	20	5	0.0	0	360.0	360.0	47.7	1.00	0.518	2.000	Non-Liq.	0.00
4	120.0	Unsaturated	Unsaturated	20	5	0.0	0	480.0	480.0	47.7	0.99	0.516	2.000	Non-Liq.	0.00
5	120.0	Unsaturated	Unsaturated	20	5	0.0	0	600.0	600.0	47.3	0.99	0.514	2.000	Non-Liq.	0.00
6	120.0	Unsaturated	Unsaturated	20	5	0.0	0	720.0	720.0	44.2	0.99	0.513	2.000	Non-Liq.	0.00
7	120.0	Unsaturated	Unsaturated	20	5	0.0	0	840.0	840.0	41.8	0.98	0.511	2.000	Non-Liq.	0.00
8	120.0	Unsaturated	Unsaturated	34	10	0.0	0	960.0	960.0	62.5	0.98	0.508	2.000	Non-Liq.	0.00
9	120.0	Unsaturated	Unsaturated	34	10	0.0	0	1080.0	1080.0	64.4	0.97	0.506	2.000	Non-Liq.	0.00
10	120.0	Unsaturated	Unsaturated	34	10	0.0	0	1200.0	1200.0	62.6	0.97	0.504	2.000	Non-Liq.	0.00
11	120.0	Unsaturated	Unsaturated	34	10	0.0	0	1320.0	1320.0	61.1	0.97	0.502	2.000	Non-Liq.	0.00
12	120.0	Unsaturated	Unsaturated	34	10	0.0	0	1440.0	1440.0	59.7	0.96	0.500	2.000	Non-Liq.	0.00
13	120.0	Unsaturated	Unsaturated	35	15	0.0	0	1560.0	1560.0	60.2	0.96	0.497	2.000	Non-Liq.	0.00
14	120.0	Unsaturated	Unsaturated	35	15	0.0	0	1680.0	1680.0	59.0	0.95	0.495	2.000	Non-Liq.	0.00
15	120.0	Unsaturated	Unsaturated	35	15	0.0	0	1800.0	1800.0	64.8	0.95	0.492	2.000	Non-Liq.	0.00
16	120.0	Unsaturated	Unsaturated	35	15	0.0	0	1920.0	1920.0	63.7	0.94	0.490	2.000	Non-Liq.	0.00
17	120.0	Unsaturated	Unsaturated	35	15	0.0	0	2040.0	2040.0	62.7	0.94	0.487	2.000	Non-Liq.	0.00
18	120.0	Unsaturated	Unsaturated	35	15	0.0	0	2160.0	2160.0	61.7	0.93	0.484	2.000	Non-Liq.	0.00
19	120.0	Unsaturated	Unsaturated	35	15	0.0	0	2280.0	2280.0	60.9	0.93	0.482	2.000	Non-Liq.	0.00
20	120.0	Unsaturated	Unsaturated	37	20	0.0	0	2400.0	2400.0	63.5	0.92	0.479	2.000	Non-Liq.	0.00
21	120.0	Unsaturated	Unsaturated	37	20	0.0	0	2520.0	2520.0	62.7	0.92	0.476	2.000	Non-Liq.	0.00
22	120.0	Unsaturated	Unsaturated	37	20	0.0	0	2640.0	2640.0	61.9	0.91	0.473	2.000	Non-Liq.	0.00
23	120.0	Unsaturated	Unsaturated	37	20	0.0	0	2760.0	2760.0	61.2	0.91	0.471	2.000	Non-Liq.	0.00
24	120.0	Unsaturated	Unsaturated	37	20	0.0	0	2880.0	2880.0	60.5	0.90	0.468	2.000	Non-Liq.	0.00
25	120.0	Unsaturated	Unsaturated	34	25	0.0	0	3000.0	3000.0	55.0	0.89	0.465	2.000	Non-Liq.	0.00
26	120.0	Unsaturated	Unsaturated	34	25	0.0	0	3120.0	3120.0	54.5	0.89	0.462	2.000	Non-Liq.	0.00
27	120.0	Unsaturated	Unsaturated	34	25	0.0	0	3240.0	3240.0	53.9	0.88	0.459	2.000	Non-Liq.	0.00
28	120.0	Unsaturated	Unsaturated	34	25	0.0	0	3360.0	3360.0	56.2	0.88	0.456	2.000	Non-Liq.	0.00
29	120.0	Unsaturated	Unsaturated	34	25	0.0	0	3480.0	3480.0	55.7	0.87	0.453	2.000	Non-Liq.	0.00
30	120.0	Unsaturated	Unsaturated	19	30	0.0	0	3600.0	3600.0	27.6	0.87	0.450	0.401	Non-Liq.	0.00
31	120.0	Unsaturated	Unsaturated	19	30	0.0	0	3720.0	3720.0	27.3	0.86	0.447	0.384	Non-Liq.	0.00
32	120.0	Unsaturated	Unsaturated	19	30	0.0	0	3840.0	3840.0	26.9	0.85	0.444	0.369	Non-Liq.	0.00
33	120.0	Unsaturated	Unsaturated	19	30	0.0	0	3960.0	3960.0	26.5	0.85	0.441	0.355	Non-Liq.	0.00
34	120.0	Unsaturated	Unsaturated	19	30	0.0	0	4080.0	4080.0	26.2	0.84	0.438	0.343	Non-Liq.	0.00
35	120.0	Unsaturated	Unsaturated	36	35	0.0	0	4200.0	4200.0	56.1	0.84	0.434	1.916	Non-Liq.	0.00
36	120.0	Unsaturated	Unsaturated	36	35	0.0	0	4320.0	4320.0	55.7	0.83	0.431	1.896	Non-Liq.	0.00
37	120.0	Unsaturated	Unsaturated	36	35	0.0	0	4440.0	4440.0	55.3	0.82	0.428	1.877	Non-Liq.	0.00
38	120.0	Unsaturated	Unsaturated	36	35	0.0	0	4560.0	4560.0	54.9	0.82	0.425	1.858	Non-Liq.	0.00
39	120.0	Unsaturated	Saturated	36	35	0.0	0	4680.0	4617.6	54.7	0.81	0.428	1.849	4.3	0.00
40	120.0	Unsaturated	Saturated	75	40	0.0	0	4800.0	4675.2	113.7	0.81	0.430	1.840	4.3	0.00
41	120.0	Unsaturated	Saturated	75	40	0.0	0	4920.0	4732.8	113.3	0.80	0.432	1.832	4.2	0.00
42	120.0	Unsaturated	Saturated	75	40	0.0	0	5040.0	4790.4	113.0	0.79	0.434	1.823	4.2	0.00
43	120.0	Unsaturated	Saturated	75	40	0.0	0	5160.0	4848.0	112.6	0.79	0.436	1.814	4.2	0.00
44	120.0	Saturated	Saturated	26	45	80.4	15	5280.0	4905.6	43.5	0.78	0.437	1.806	4.1	0.00
45	120.0	Saturated	Saturated	26	45	80.4	15	5400.0	4963.2	43.4	0.78	0.439	1.798	4.1	0.00
46	120.0	Saturated	Saturated	26	45	80.4	15	5520.0	5020.8	43.2	0.77	0.440	1.790	4.1	0.00
47	120.0	Saturated	Saturated	26	45	80.4	15	5640.0	5078.4	43.1	0.76	0.441	1.782	4.0	0.00
48	120.0	Saturated	Saturated	28	50	0.0	0	5760.0	5136.0	41.0	0.76	0.442	1.774	4.0	0.00
49	120.0	Saturated	Saturated	28	50	0.0	0	5880.0	5193.6	40.8	0.75	0.443	1.766	4.0	0.00
50	120.0	Saturated	Saturated	28	50	0.0	0	6000.0	5251.2	40.7	0.75	0.443	1.758	4.0	0.00
51	120.0	Saturated	Saturated	28	50	0.0	0	6120.0	5308.8	40.5	0.74	0.444	1.750	3.9	0.00
52	120.0	Saturated	Saturated	28	50	0.0	0	6240.0	5366.4	40.4	0.74	0.444	1.742	3.9	0.00
53	120.0	Saturated	Saturated	28	50	0.0	0	6360.0	5424.0	40.2	0.73	0.445	1.735	3.9	0.00
54	120.0	Saturated	Saturated	28	50	0.0	0	6480.0	5481.6	40.1	0.72	0.445	1.727	3.9	0.00
55	120.0	Saturated	Saturated	38	55	76.1	13	6600.0	5539.2	60.6	0.72	0.445	1.720	3.9	0.00
56	120.0	Saturated	Saturated	38	55	76.1	13	6720.0	5596.8	60.5	0.71	0.445	1.713	3.8	0.00
57	120.0	Saturated	Saturated	38	55	76.1	13	6840.0	5654.4	60.3	0.71	0.445	1.705	3.8	0.00
58	120.0	Saturated	Saturated	38	55	76.1	13	6960.0	5712.0	60.2	0.70	0.445	1.698	3.8	0.00
59	120.0	Saturated	Saturated	38	55	76.1	13	7080.0	5769.6	60.1	0.70	0.444	1.691	3.8	0.00
60	120.0	Saturated	Saturated	35	60	78.2	20	7200.0	5827.2	55.6	0.69	0.444	1.684	Non-Liq.	0.00
61	120.0	Saturated	Saturated	35	60	78.2	20	7320.0	5884.8	55.5	0.69	0.444	1.677	Non-Liq.	0.00
62	120.0	Saturated	Saturated	35	60	78.2	20	7440.0	5942.4	55.4	0.68	0.443	1.670	Non-Liq.	0.00
63	120.0	Saturated	Saturated	45	65	0.0	0	7560.0	6000.0	63.9	0.68	0.443	1.663	3.8	0.00
64	120.0	Saturated	Saturated	45	65	0.0	0	7680.0	6057.6	63.7	0.67	0.442	1.656	3.7	0.00
65	120.0	Saturated	Saturated	45	65	0.0	0	7800.0	6115.2	63.6	0.67	0.441	1.650	3.7	0.00
66	120.0	Saturated	Saturated	45	65	0.0	0	7920.0	6172.8	63.4	0.66	0.441	1.643	3.7	0.00
67	120.0	Saturated	Saturated	45	65	0.0	0	8040.0	6230.4	63.2	0.66	0.440	1.636	3.7	0.00
68	120.0	Saturated	Saturated	29	70	79.2	17	8160.0	6288.0	45.5	0.65	0.439	1.630	3.7	0.00
69	120.0	Saturated	Saturated	29	70	79.2	17	8280.0	6345.6	45.4	0.65	0.439	1.623	3.7	0.00
70	120.0	Saturated	Saturated	29	70	79.2	17	8400.0	6403.2	45.3	0.64	0.438	1.617	3.7	0.00
71	120.0	Saturated	Saturated	29	70	79.2	17	8520.0	6460.8	45.2	0.64	0.437	1.611	3.7	0.00
72	120.0	Saturated	Saturated	29	70	79.2	17	8640.0	6518.4	45.0	0.63	0.436	1.604	3.7	0.00
73	120.0	Saturated	Saturated	39	75	35.2	0	8760.0	6576.0	59.5	0.63	0.435	1.598	3.7	0.00
74	120.0	Saturated	Saturated	39	75	35.2	0	8880.0	6633.6	59.4	0.63	0.435	1.592	3.7	0.00
75	120.0	Saturated	Saturated	39	75	35.2	0	9000.0	6691.2	59.3	0.62	0.434	1.586	3.7	0.00
76	120.0	Saturated	Saturated	39	75	35.2	0	9120.0	6748.8	59.2	0.62	0.433	1.580	3.6	0.00
77	120.0	Saturated	Saturated	39	75	35.2	0	9240.0	6806.4	59.1	0.61	0.432	1.574	3.6	0.00
78	120.0	Saturated	Saturated	46	80	0.0	0	9360.0	6864.0	63.0	0.61	0.431	1.568	3.6	0.00
79	120.0	Saturated	Saturated	46	80	0.0	0	9480.0	6921.6	62.9	0.60	0.430	1.562	3.6	0.00
80	120.0	Saturated	Saturated	46	80	0.0	0	9600.0	6979.2	62.7	0.60	0.429	1.556	3.6	0.00
81	120.0	Saturated	Saturated	46	80	0.0	0	9720.0	7036.8	62.6	0.60	0.429	1.550	3.6	0.00
82	120.0	Saturated	Saturated	46	80	0.0	0	9840.0	7094.4	62.5	0.59	0.428	1.544	3.6	0.00
83	120.0	Saturated	Saturated	46	80	0.0	0	9960.0	7152.0	62.3	0.59	0.427	1.539	3.6	0.00
84	120.0	Saturated	Saturated	46	80	0.0	0	10080.0	7209.6	62.2	0.59	0.426	1.533	3.6	0.00
85	120.0	Saturated	Saturated	72	85	0.0	0	10200.0	7267.2	97.2	0.58	0.425	1.527	3.6	0.00
86	120.0	Saturated	Saturated	72	85	0.0	0	10320.0	7324.8	97.0	0.58	0.425	1.522	3.6	0.00
87	120.0	Saturated	Saturated	72	85	0.0	0	10440.0	7382.4	96.8	0.58	0.424	1.516	3.6	0.00
88	120.0	Saturated	Saturated	72	85	0.0	0	10560.0	7440.0	96.6	0.57	0.423	1.511	3.6	0.00
89	120.0	Saturated	Saturated	72	85	0.0	0	10680.0	7497.6	96.4	0.57	0.423	1.505	3.6	0.00
90	120.0	Saturated	Saturated	85	90	0.0	0	10800.0	7555.2	113.5	0.57	0.422	1.500	3.6	0.00
91	120.0	Saturated	Saturated	85	90	0.0	0	10920.0	7612.8	113.3	0.57	0.421	1.494	3.5	0.00
92	120.0	Saturated	Saturated	85	90	0.0	0	11040.0	7670.4	113.1	0.56	0.421	1.489	3.5	0.00
93	120.0	Saturated	Saturated	85	90	0.0	0	11160.0	7728.0	112.9	0.56	0.420	1.484	3.5	0.00
94	120.0	Saturated	Saturated	85	90	0.0	0	11280.0	7785.6	112.7	0.56	0.420	1.478	3.5	0.00
95	120.0	Saturated	Saturated	36	95	0.0	0	11400.0	7843.2	47.6	0.56	0.419	1.473	3.5	0.00
96	120.0	Saturated	Saturated	36	95	0.0	0	11520.0	7900.8	47.5	0.55	0.419	1.468	3.5	0.00
97	120.0	Saturated	Saturated	36	95	0.0	0	11640.0	7958.4	47.4	0.55	0.419	1.463	3.5	0.00
98	120.0	Saturated	Saturated	36	95	0.0	0	11760.0	8016.0	47.3	0.55	0.418	1.458	3.5	0.00
99	120.0	Saturated	Saturated	36	95	0.0	0	11880.0	8073.6	47.3	0.55	0.418	1.453	3.5	0.00
100	120.0	Saturated	Saturated	36	100	0.0	0	12000.0	8131.2	47.2	0.55	0.418	1.447	3.5	0.00

Total Liquefaction Settlement, S = 0.00 inches

Geotechnologies, Inc.

Project: Kilroy Realty
File No.: 21800
Description: Liquefaction Analysis - Maximum Considered Earthquake (2% Probability of Exceedance in 50 years)
Boring Number: 5

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.799
Calculated Mag.Wtg.Factor:	1.203

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	43.5
Historically Highest Groundwater Level* (ft):	38.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

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

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	120.0	Unsaturated	Unsaturated	28	5	0.0	0	120.0	120.0	66.7	1.00	0.522	2.000	Non-Liq.	0.00
2	120.0	Unsaturated	Unsaturated	28	5	0.0	0	240.0	240.0	66.7	1.00	0.520	2.000	Non-Liq.	0.00
3	120.0	Unsaturated	Unsaturated	28	5	0.0	0	360.0	360.0	66.7	1.00	0.518	2.000	Non-Liq	0.00
4	120.0	Unsaturated	Unsaturated	28	5	0.0	0	480.0	480.0	61.4	0.99	0.516	2.000	Non-Liq	0.00
5	120.0	Unsaturated	Unsaturated	28	5	0.0	0	600.0	600.0	60.1	0.99	0.514	2.000	Non-Liq	0.00
6	120.0	Unsaturated	Unsaturated	28	5	0.0	0	720.0	720.0	57.0	0.99	0.513	2.000	Non-Liq.	0.00
7	120.0	Unsaturated	Unsaturated	28	5	0.0	0	840.0	840.0	54.5	0.98	0.511	2.000	Non-Liq	0.00
8	120.0	Unsaturated	Unsaturated	21	10	0.0	0	960.0	960.0	41.5	0.98	0.508	2.000	Non-Liq	0.00
9	120.0	Unsaturated	Unsaturated	21	10	0.0	0	1080.0	1080.0	41.9	0.97	0.506	2.000	Non-Liq.	0.00
10	120.0	Unsaturated	Unsaturated	21	10	0.0	0	1200.0	1200.0	40.4	0.97	0.504	2.000	Non-Liq.	0.00
11	120.0	Unsaturated	Unsaturated	21	10	0.0	0	1320.0	1320.0	39.1	0.97	0.502	2.000	Non-Liq.	0.00
12	120.0	Unsaturated	Unsaturated	21	10	0.0	0	1440.0	1440.0	38.0	0.96	0.500	2.000	Non-Liq	0.00
13	120.0	Unsaturated	Unsaturated	21	10	0.0	0	1560.0	1560.0	36.8	0.96	0.497	2.000	Non-Liq	0.00
14	120.0	Unsaturated	Unsaturated	21	10	0.0	0	1680.0	1680.0	35.6	0.95	0.495	1.609	Non-Liq.	0.00
15	120.0	Unsaturated	Unsaturated	27	15	0.0	0	1800.0	1800.0	50.0	0.95	0.492	2.000	Non-Liq	0.00
16	120.0	Unsaturated	Unsaturated	27	15	0.0	0	1920.0	1920.0	49.1	0.94	0.490	2.000	Non-Liq	0.00
17	120.0	Unsaturated	Unsaturated	27	15	0.0	0	2040.0	2040.0	48.4	0.94	0.487	2.000	Non-Liq	0.00
18	120.0	Unsaturated	Unsaturated	27	15	0.0	0	2160.0	2160.0	47.6	0.93	0.484	2.000	Non-Liq.	0.00
19	120.0	Unsaturated	Unsaturated	27	15	0.0	0	2280.0	2280.0	47.0	0.93	0.482	2.000	Non-Liq.	0.00
20	120.0	Unsaturated	Unsaturated	24	20	0.0	0	2400.0	2400.0	41.1	0.92	0.479	2.000	Non-Liq.	0.00
21	120.0	Unsaturated	Unsaturated	24	20	0.0	0	2520.0	2520.0	40.5	0.92	0.476	2.000	Non-Liq.	0.00
22	120.0	Unsaturated	Unsaturated	24	20	0.0	0	2640.0	2640.0	40.0	0.91	0.473	2.000	Non-Liq	0.00
23	120.0	Unsaturated	Unsaturated	24	20	0.0	0	2760.0	2760.0	39.3	0.91	0.471	2.000	Non-Liq	0.00
24	120.0	Unsaturated	Unsaturated	24	20	0.0	0	2880.0	2880.0	38.7	0.90	0.468	2.000	Non-Liq.	0.00
25	120.0	Unsaturated	Unsaturated	30	25	0.0	0	3000.0	3000.0	48.5	0.89	0.465	2.000	Non-Liq	0.00
26	120.0	Unsaturated	Unsaturated	30	25	0.0	0	3120.0	3120.0	48.0	0.89	0.462	2.000	Non-Liq.	0.00
27	120.0	Unsaturated	Unsaturated	30	25	0.0	0	3240.0	3240.0	47.6	0.88	0.459	2.000	Non-Liq	0.00
28	120.0	Unsaturated	Unsaturated	54	30	0.0	0	3360.0	3360.0	89.3	0.88	0.456	2.000	Non-Liq	0.00
29	120.0	Unsaturated	Unsaturated	54	30	0.0	0	3480.0	3480.0	88.5	0.87	0.453	2.000	Non-Liq	0.00
30	120.0	Unsaturated	Unsaturated	54	30	0.0	0	3600.0	3600.0	87.7	0.87	0.450	2.000	Non-Liq.	0.00
31	120.0	Unsaturated	Unsaturated	54	30	0.0	0	3720.0	3720.0	86.9	0.86	0.447	2.000	Non-Liq.	0.00
32	120.0	Unsaturated	Unsaturated	54	30	0.0	0	3840.0	3840.0	86.2	0.85	0.444	1.980	Non-Liq	0.00
33	120.0	Unsaturated	Unsaturated	18	35	55.1	11	3960.0	3960.0	30.3	0.85	0.441	0.527	Non-Liq.	0.00
34	120.0	Unsaturated	Unsaturated	18	35	55.1	11	4080.0	4080.0	30.0	0.84	0.438	0.503	Non-Liq.	0.00
35	120.0	Unsaturated	Unsaturated	18	35	55.1	11	4200.0	4200.0	29.6	0.84	0.434	0.481	Non-Liq.	0.00
36	120.0	Unsaturated	Unsaturated	18	35	55.1	11	4320.0	4320.0	29.4	0.83	0.431	0.462	Non-Liq.	0.00
37	120.0	Unsaturated	Unsaturated	18	35	47.4	3	4440.0	4440.0	29.1	0.82	0.428	0.445	Non-Liq	0.00
38	120.0	Unsaturated	Unsaturated	20	40	47.4	3	4560.0	4560.0	32.4	0.82	0.425	0.683	Non-Liq	0.00
39	120.0	Unsaturated	Saturated	20	40	47.4	3	4680.0	4617.6	32.3	0.81	0.428	0.666	1.6	0.00
40	120.0	Unsaturated	Saturated	20	40	47.4	3	4800.0	4675.2	32.1	0.81	0.430	0.651	1.5	0.00
41	120.0	Unsaturated	Saturated	20	40	47.4	3	4920.0	4732.8	32.0	0.80	0.432	0.636	1.5	0.00
42	120.0	Unsaturated	Saturated	20	40	47.4	3	5040.0	4790.4	31.9	0.79	0.434	0.622	1.4	0.00
43	120.0	Unsaturated	Saturated	9	45	98.0	19	5160.0	4848.0	14.9	0.79	0.436	0.170	Non-Liq.	0.00
44	120.0	Saturated	Saturated	9	45	98.0	19	5280.0	4905.6	14.9	0.78	0.437	0.169	Non-Liq.	0.00
45	120.0	Saturated	Saturated	9	45	98.0	19	5400.0	4963.2	14.8	0.78	0.439	0.168	Non-Liq.	0.00
46	120.0	Saturated	Saturated	9	45	98.0	19	5520.0	5020.8	14.8	0.77	0.440	0.168	Non-Liq.	0.00
47	120.0	Saturated	Saturated	9	45	98.0	19	5640.0	5078.4	14.7	0.76	0.441	0.167	Non-Liq.	0.00
48	120.0	Saturated	Saturated	9	45	98.0	19	5760.0	5136.0	14.7	0.76	0.442	0.166	Non-Liq.	0.00
49	120.0	Saturated	Saturated	9	45	98.0	19	5880.0	5193.6	14.6	0.75	0.443	0.166	Non-Liq.	0.00
50	120.0	Saturated	Saturated	30	50	45.9	9	6000.0	5251.2	49.7	0.75	0.443	1.758	4.0	0.00
51	120.0	Saturated	Saturated	30	50	45.9	9	6120.0	5308.8	49.6	0.74	0.444	1.750	3.9	0.00
52	120.0	Saturated	Saturated	30	50	45.9	9	6240.0	5366.4	49.5	0.74	0.444	1.742	3.9	0.00
53	120.0	Saturated	Saturated	28	55	45.7	13	6360.0	5424.0	45.8	0.73	0.445	1.735	3.9	0.00
54	120.0	Saturated	Saturated	28	55	45.7	13	6480.0	5481.6	45.7	0.72	0.445	1.727	3.9	0.00
55	120.0	Saturated	Saturated	28	55	45.7	13	6600.0	5539.2	45.6	0.72	0.445	1.720	3.9	0.00
56	120.0	Saturated	Saturated	28	55	45.7	13	6720.0	5596.8	45.4	0.71	0.445	1.713	3.8	0.00
57	120.0	Saturated	Saturated	28	55	45.7	13	6840.0	5654.4	45.3	0.71	0.445	1.705	3.8	0.00
58	120.0	Saturated	Saturated	24	60	54.6	9	6960.0	5712.0	37.6	0.70	0.445	1.698	3.8	0.00
59	120.0	Saturated	Saturated	24	60	54.6	9	7080.0	5769.6	37.5	0.70	0.444	1.669	3.8	0.00
60	120.0	Saturated	Saturated	24	60	54.6	9	7200.0	5827.2	37.4	0.69	0.444	1.612	3.6	0.00
61	120.0	Saturated	Saturated	24	60	54.6	9	7320.0	5884.8	37.2	0.69	0.444	1.557	3.5	0.00
62	120.0	Saturated	Saturated	24	60	54.6	9	7440.0	5942.4	37.1	0.68	0.443	1.505	3.4	0.00
63	120.0	Saturated	Saturated	44	65	0.0	0	7560.0	6000.0	62.5	0.68	0.443	1.663	3.8	0.00
64	120.0	Saturated	Saturated	44	65	0.0	0	7680.0	6057.6	62.3	0.67	0.442	1.656	3.7	0.00
65	120.0	Saturated	Saturated	44	65	0.0	0	7800.0	6115.2	62.1	0.67	0.441	1.650	3.7	0.00
66	120.0	Saturated	Saturated	44	65	0.0	0	7920.0	6172.8	62.0	0.66	0.441	1.643	3.7	0.00
67	120.0	Saturated	Saturated	44	65	0.0	0	8040.0	6230.4	61.8	0.66	0.440	1.636	3.7	0.00
68	120.0	Saturated	Saturated	36	70	34.0	0	8160.0	6288.0	56.0	0.65	0.439	1.630	3.7	0.00
69	120.0	Saturated	Saturated	36	70	34.0	0	8280.0	6345.6	55.8	0.65	0.439	1.623	3.7	0.00
70	120.0	Saturated	Saturated	36	70	34.0	0	8400.0	6403.2	55.7	0.64	0.438	1.617	3.7	0.00
71	120.0	Saturated	Saturated	36	70	34.0	0	8520.0	6460.8	55.6	0.64	0.437	1.611	3.7	0.00
72	120.0	Saturated	Saturated	36	70	34.0	0	8640.0	6518.4	55.5	0.63	0.436	1.604	3.7	0.00
73	120.0	Saturated	Saturated	34	75	48.6	11	8760.0	6576.0	52.7	0.63	0.435	1.598	3.7	0.00
74	120.0	Saturated	Saturated	34	75	48.6	11	8880.0	6633.6	52.6	0.63	0.435	1.592	3.7	0.00
75	120.0	Saturated	Saturated	34	75	48.6	11	9000.0	6691.2	52.5	0.62	0.434	1.586	3.7	0.00
76	120.0	Saturated	Saturated	34	75	48.6	11	9120.0	6748.8	52.4	0.62	0.433	1.580	3.6	0.00
77	120.0	Saturated	Saturated	34	75	48.6	11	9240.0	6806.4	52.3	0.61	0.432	1.574	3.6	0.00
78	120.0	Saturated	Saturated	66	80	0.0	0	9360.0	6864.0	90.4	0.61	0.431	1.568	3.6	0.00
79	120.0	Saturated	Saturated	66	80	0.0	0	9480.0	6921.6	90.2	0.60	0.430	1.562	3.6	0.00
80	120.0	Saturated	Saturated	66	80	0.0	0	9600.0	6979.2	90.0	0.60	0.429	1.556	3.6	0.00
81	120.0	Saturated	Saturated	66	80	0.0	0	9720.0	7036.8	89.8	0.60	0.429	1.550	3.6	0.00
82	120.0	Saturated	Saturated	66	80	0.0	0	9840.0	7094.4	89.6	0.59	0.428	1.544	3.6	0.00
83	120.0	Saturated	Saturated	66	80	0.0	0	9960.0	7152.0	89.4	0.59	0.427	1.539	3.6	0.00
84	120.0	Saturated	Saturated	66	80	0.0	0	10080.0	7209.6	89.3	0.59	0.426	1.533	3.6	0.00
85	120.0	Saturated	Saturated	68	85	0.0	0	10200.0	7267.2	91.8	0.58	0.425	1.527	3.6	0.00
86	120.0	Saturated	Saturated	68	85	0.0	0	10320.0	7324.8	91.6	0.58	0.425	1.522	3.6	0.00
87	120.0	Saturated	Saturated	68	85	0.0	0	10440.0	7382.4	91.4	0.58	0.424	1.516	3.6	0.00
88	120.0	Saturated	Saturated	68	85	0.0	0	10560.0	7440.0	91.2	0.57	0.423	1.511	3.6	0.00
89	120.0	Saturated	Saturated	68	85	0.0	0	10680.0	7497.6	91.0	0.57	0.423	1.505	3.6	0.00
90	120.0	Saturated	Saturated	69	90	0.0	0	10800.0	7555.2	92.2	0.57	0.422	1.500	3.6	0.00
91	120.0	Saturated	Saturated	69	90	0.0	0	10920.0	7612.8	92.0	0.57	0.421	1.494	3.5	0.00
92	120.0	Saturated	Saturated	69	90	0.0	0	11040.0	7670.4	91.8	0.56	0.421	1.489	3.5	0.00
93	120.0	Saturated	Saturated	69	90	0.0	0	11160.0	7728.0	91.6	0.56	0.420	1.484	3.5	0.00
94	120.0	Saturated	Saturated	69	90	0.0	0	11280.0	7785.6	91.4	0.56	0.420	1.478	3.5	0.00
95	120.0	Saturated	Saturated	48	95	0.0	0	11400.0	7843.2	63.5	0.56	0.419	1.473	3.5	0.00
96	120.0	Saturated	Saturated	48	95	0.0	0	11520.0	7900.8	63.4	0.55	0.419	1.468	3.5	0.00
97	120.0	Saturated	Saturated	48	95	0.0	0	11640.0	7958.4	63.3	0.55	0.419	1.463	3.5	0.00
98	120.0	Saturated	Saturated	48	95	0.0	0	11760.0	8016.0	63.1	0.55	0.418	1.458	3.5	0.00
99	120.0	Saturated	Saturated	48	95	0.0	0	11880.0	8073.6	63.0	0.55	0.418	1.453	3.5	0.00
100	120.0	Saturated	Saturated	79	100	0.0	0	12000.0	8131.2	103.5	0.55	0.418	1.447	3.5	0.00
											Total Liquefaction Settlement, S =			0.00 inches	

Geotechnologies, Inc.

Project: Kilroy Realty Corporation
File No.: 21800
Description: Alluvium
7/3/2019

Friction Pile Capacity Calculation

Input Data:

Unit Weight of Overlying Soil Layer	γ_1	115 pcf
Thickness of Overlying Soil Layer	H_1	10 feet
Unit Weight of Bearing Strata	γ_2	125 pcf
Friction Angle of Bearing Strata	ϕ_2	26 degrees
Cohesion of Bearing Strata	c_2	385 psf
Minimum Embedment into Bearing Strata	H_2	10 feet
Unit Weight of Water	γ_w	62.4 pcf
Depth to Groundwater from Pile Cap	H_w	10 feet

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

Pile Dimension:

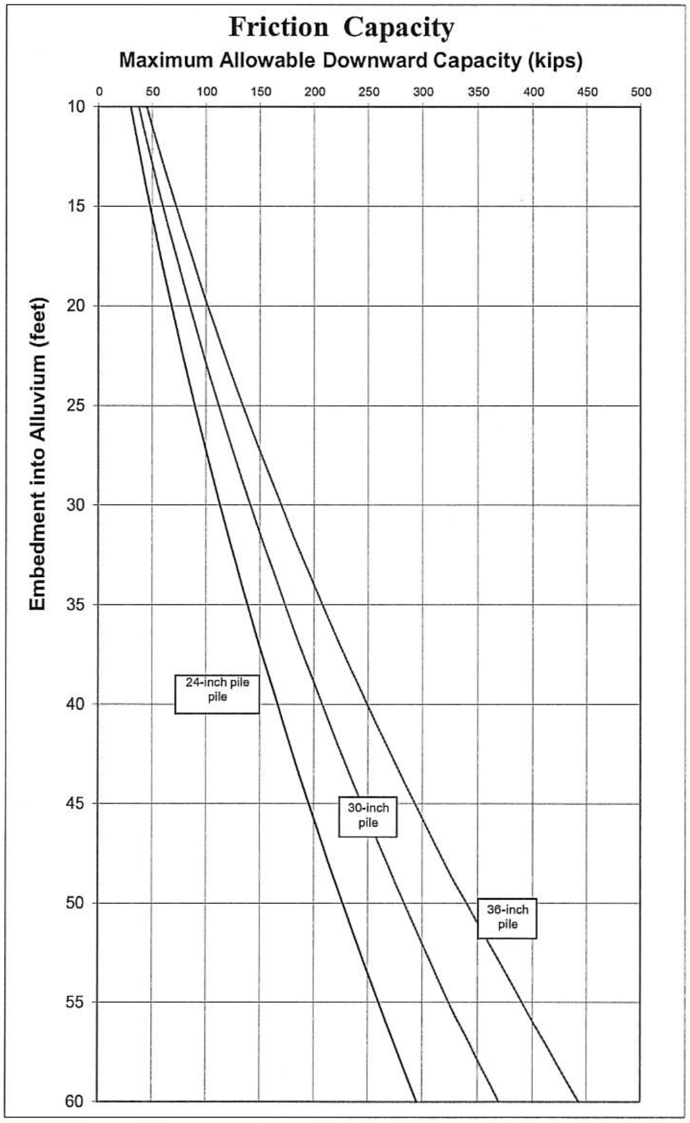
24	in. Diam.	3.14	ft^2 Area
30	in. Diam.	4.91	ft^2 Area
36	in. Diam.	7.07	ft^2 Area

Critical Depth Limit (Dc):
30 B

Lateral Earth Pressure Coefficient: $K_c = 0.80$
Applied Factor of Safety: $FS = 2$
Factored Skin Friction $f_{ult} = [c_2 + K_c * \sigma'_v * (\tan \phi_2)]/FS$
$f_{allow} = f_{ult}/FS$

Pile Capacity:

Total Depth of Pile (feet)	Depth of Embedment into Bearing Strata (feet)	Maximum Allowable Downward Pile Capacity: Capacity of 24 inch diameter pile (kips)	Capacity of 30 inch diameter pile (kips)	Capacity of 36 inch diameter pile (kips)
20	10	30.0	37.5	45.0
21	11	33.5	41.8	50.2
22	12	37.0	46.2	55.4
23	13	40.5	50.7	60.8
24	14	44.2	55.2	66.3
25	15	47.9	59.9	71.9
26	16	51.7	64.7	77.6
27	17	55.6	69.5	83.4
28	18	59.6	74.5	89.4
29	19	63.6	79.5	95.4
30	20	67.7	84.7	101.6
31	21	71.9	89.9	107.9
32	22	76.2	95.2	114.3
33	23	80.5	100.7	120.8
34	24	85.0	106.2	127.4
35	25	89.5	111.8	134.2
36	26	94.0	117.5	141.1
37	27	98.7	123.4	148.0
38	28	103.4	129.3	155.1
39	29	108.2	135.3	162.3
40	30	113.1	141.4	169.7
41	31	118.1	147.6	177.1
42	32	123.1	153.9	184.7
43	33	128.2	160.3	192.3
44	34	133.4	166.8	200.1
45	35	138.7	173.3	208.0
46	36	144.0	180.0	216.0
47	37	149.4	186.8	224.2
48	38	154.9	193.7	232.4
49	39	160.5	200.6	240.8
50	40	166.2	207.7	249.2
51	41	171.9	214.9	257.8
52	42	177.7	222.1	266.5
53	43	183.6	229.5	275.4
54	44	189.5	236.9	284.3
55	45	195.6	244.4	293.3
56	46	201.7	252.1	302.5
57	47	207.9	259.8	311.8
58	48	214.1	267.7	321.2
59	49	220.5	275.6	330.7
60	50	226.9	283.6	340.3
61	51	233.4	291.7	350.1
62	52	239.9	299.9	359.9
63	53	246.6	308.2	369.9
64	54	253.3	316.6	380.0
65	55	260.1	325.1	390.2
66	56	267.0	333.7	400.5
67	57	274.0	342.4	410.9
68	58	281.0	351.2	421.5
69	59	288.1	360.1	432.1
70	60	295.3	369.1	442.9



Note: 1. Minimum pile embedment depth of 20 feet
2. Uplift capacity may be designed using 50% of the downward capacity
3. Pile should be spaced a minimum of 2-1/2 diameters on center
4. See text of report for pile details and installation recommendations

Geotechnologies, Inc.

Project: Kilroy Realty Corp

File No.: 21800

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	40.0 feet
Retained Soil Unit Weight:	(γ)	125.0 pcf
Peak Ground Acceleration	(PGAm)	0.580 g
Horizontal Ground Acceleration: (1/2 of 2/3*PGAm)	(kh)	0.19 g

Seismic Increment (ΔP_{AE}):

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

$\Delta P_{AE} =$ 14500.0 lbs/ft

Force applied at 0.6H above the base of the wall
Transfer load to 2/3 of the height of the wall

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

T = 13050.0 lbs/ft

$EFP = 2*T/H^2$

EFP = 16.3 pcf Triangular shape

Geotechnologies, Inc.

Project: Kilroy Realty Corporation

File No.: 21800

Geologic Material Fill

Soil Weight	γ	120 pcf
Internal Friction Angle	ϕ	16 degrees
Cohesion	c	200 psf
Height of Retaining Wall	H	40 feet

Cantilever Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.724
$\sigma'_v = \gamma H$	4800.0 psf

$\sigma'_h =$ 3476.9 psf

EFP = 86.9 pcf

$P_o =$ 69538.8 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 87 pcf

Restrained Wall Design based on At Rest Earth Pressure

$P_o =$ 69538.8 lbs/ft

$\sigma'_{h,\,max} =$ 54.3 H (based on a trapezoidal distribution of pressure)

$\sigma'_{h,\,max} =$ 1738.5 psf

Design restrained wall for 54 H

Geotechnologies, Inc.

Project:	Kilroy Realty Corporation
File No.:	21800
Geologic Material	Alluvium

Soil Weight	γ	125 pcf
Internal Friction Angle	ϕ	28 degrees
Cohesion	c	385 psf
Height of Retaining Wall	H	40 feet

Cantilever Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$ 0.531

$\sigma'_v = \gamma H$ 5000.0 psf

$\sigma'_h =$ 2652.6 psf

EFP = 66.3 pcf

$P_o =$ 53052.8 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 66 pcf

Restrained Wall Design based on At Rest Earth Pressure

$P_o =$ 53052.8 lbs/ft

$\sigma'_{h,\,max} =$ 41.4 H (based on a trapezoidal distribution of pressure)

$\sigma'_{h,\,max} =$ 1326.3 psf

Design restrained wall for 41 H

Geotechnologies, Inc.

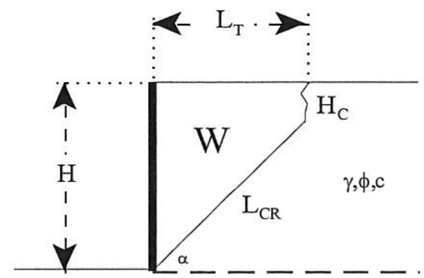
Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

Retaining Wall Design with Level Backfill
(Vector Analysis)

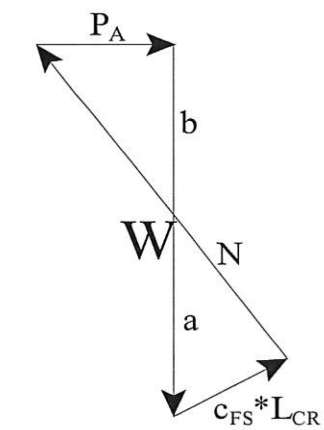
Input:

Retaining Wall Height	(H)	10.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	10.8 degrees
	(c_{FS})	133.3 psf

28



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	2.9	54	6539.9	11.0	2957.9	3582.0	2000.1
41	2.9	53	6331.0	10.9	2828.7	3502.3	2036.6
42	2.8	51	6127.4	10.7	2708.1	3419.3	2069.0
43	2.8	49	5929.0	10.6	2595.4	3333.6	2097.5
44	2.8	48	5735.6	10.4	2490.0	3245.7	2122.1
45	2.7	46	5547.1	10.3	2391.1	3156.0	2143.0
46	2.7	45	5363.3	10.1	2298.4	3064.9	2160.3
47	2.7	43	5183.9	10.0	2211.2	2972.7	2173.9
48	2.7	42	5008.9	9.8	2129.2	2879.7	2184.1
49	2.7	40	4837.9	9.7	2051.8	2786.1	2190.7
50	2.7	39	4670.9	9.5	1978.9	2692.1	2193.8
51	2.7	38	4507.7	9.4	1909.9	2597.7	2193.5
52	2.7	36	4347.9	9.3	1844.6	2503.3	2189.8
53	2.7	35	4191.5	9.1	1782.7	2408.9	2182.5
54	2.7	34	4038.3	9.0	1723.8	2314.5	2171.8
55	2.7	32	3888.1	8.9	1667.8	2220.3	2157.5
56	2.8	31	3740.7	8.7	1614.3	2126.3	2139.6
57	2.8	30	3595.9	8.6	1563.2	2032.7	2118.0
58	2.8	29	3453.6	8.5	1514.3	1939.4	2092.7
59	2.8	28	3313.7	8.3	1467.2	1846.4	2063.5
60	2.9	26	3175.9	8.2	1421.9	1754.0	2030.4
61	2.9	25	3040.1	8.1	1378.2	1662.0	1993.2
62	3.0	24	2906.2	7.9	1335.7	1570.5	1951.7
63	3.0	23	2774.0	7.8	1294.5	1479.6	1905.9
64	3.1	22	2643.4	7.7	1254.1	1389.2	1855.5
65	3.2	21	2514.1	7.5	1214.5	1299.5	1800.3



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

$P_{A,\ max}$ 2193.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 43.9 pcf

Design Wall for an Equivalent Fluid Pressure: 44 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

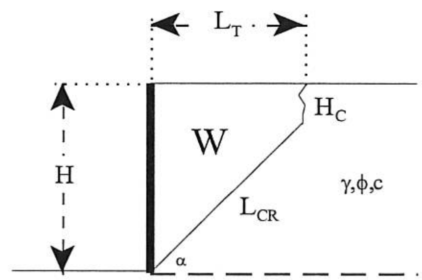
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

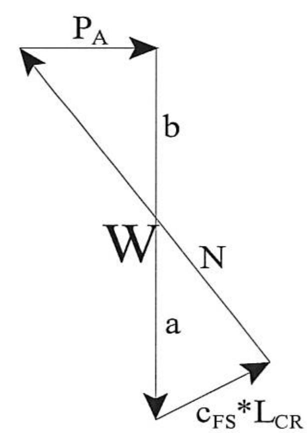
Retaining Wall Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50

28

Factored Parameters:	(ϕ_{FS})	10.8 degrees
	(c_{FS})	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	2.9	233	27991.5	26.6	7137.0	20854.5	11644.5
41	2.9	225	27037.7	26.1	6799.8	20237.9	11768.2
42	2.8	218	26118.5	25.7	6488.8	19629.7	11877.7
43	2.8	210	25231.7	25.2	6201.3	19030.4	11973.7
44	2.8	203	24375.2	24.8	5935.0	18440.1	12056.6
45	2.7	196	23547.1	24.4	5688.0	17859.1	12126.9
46	2.7	190	22745.7	24.0	5458.5	17287.2	12184.7
47	2.7	183	21969.2	23.6	5244.7	16724.4	12230.5
48	2.7	177	21216.1	23.3	5045.4	16170.7	12264.3
49	2.7	171	20485.1	22.9	4859.3	15625.9	12286.5
50	2.7	165	19774.7	22.6	4685.1	15089.6	12297.0
51	2.7	159	19083.8	22.3	4521.9	14561.8	12296.0
52	2.7	153	18411.1	22.0	4368.8	14042.2	12283.4
53	2.7	148	17755.5	21.7	4224.9	13530.6	12259.2
54	2.7	143	17116.1	21.4	4089.5	13026.5	12223.2
55	2.7	137	16491.8	21.1	3961.9	12529.9	12175.3
56	2.8	132	15881.8	20.8	3841.4	12040.4	12115.3
57	2.8	127	15285.2	20.5	3727.6	11557.7	12042.9
58	2.8	123	14701.3	20.3	3619.7	11081.6	11957.7
59	2.8	118	14129.2	20.0	3517.4	10611.7	11859.3
60	2.9	113	13568.2	19.8	3420.3	10147.9	11747.3
61	2.9	108	13017.7	19.5	3327.8	9689.9	11621.0
62	3.0	104	12477.0	19.3	3239.5	9237.5	11479.9
63	3.0	100	11945.5	19.0	3155.2	8790.3	11323.3
64	3.1	95	11422.5	18.8	3074.4	8348.2	11150.2
65	3.2	91	10907.6	18.6	2996.7	7910.9	10959.8



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

$P_{A, max}$ 12297.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 61.5 pcf

Design Wall for an Equivalent Fluid Pressure: 62 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

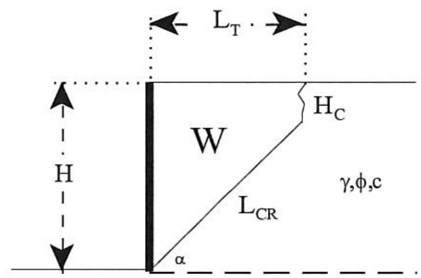
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

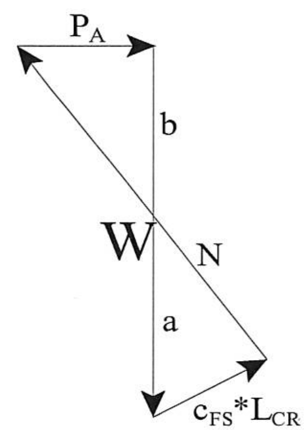
Retaining Wall Height	(H)	30.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50

28

Factored Parameters:	(ϕ_{FS})	10.8 degrees
	(c_{FS})	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	2.9	531	63744.1	42.1	11316.1	52427.9	29274.2
41	2.9	513	61548.7	41.3	10770.9	50777.9	29527.0
42	2.8	495	59436.8	40.6	10269.4	49167.5	29750.7
43	2.8	478	57402.7	39.9	9807.1	47595.6	29946.7
44	2.8	462	55441.1	39.2	9380.1	46061.0	30115.9
45	2.7	446	53547.1	38.5	8984.9	44562.2	30259.1
46	2.7	431	51716.3	37.9	8618.6	43097.7	30377.0
47	2.7	416	49944.6	37.3	8278.3	41666.4	30470.3
48	2.7	402	48228.3	36.7	7961.7	40266.6	30539.3
49	2.7	388	46563.7	36.2	7666.7	38897.1	30584.5
50	2.7	375	44947.7	35.7	7391.3	37556.4	30605.9
51	2.7	361	43377.3	35.1	7133.9	36243.4	30603.8
52	2.7	349	41849.6	34.7	6893.0	34956.6	30578.2
53	2.7	336	40362.1	34.2	6667.2	33694.9	30528.8
54	2.7	324	38912.3	33.7	6455.2	32457.1	30455.5
55	2.7	312	37498.0	33.3	6256.0	31242.0	30357.9
56	2.8	301	36117.1	32.9	6068.6	30048.5	30235.5
57	2.8	290	34767.5	32.5	5891.9	28875.6	30087.7
58	2.8	279	33447.4	32.1	5725.2	27722.2	29913.9
59	2.8	268	32155.0	31.7	5567.6	26587.3	29713.0
60	2.9	257	30888.7	31.3	5418.6	25470.1	29484.2
61	2.9	247	29647.0	30.9	5277.4	24369.6	29226.2
62	3.0	237	28428.3	30.6	5143.3	23284.9	28937.6
63	3.0	227	27231.2	30.3	5015.9	22215.3	28616.8
64	3.1	217	26054.5	29.9	4894.6	21159.9	28262.1
65	3.2	207	24896.8	29.6	4778.8	20118.0	27871.5



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

$P_{A,\ max}$ 30605.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 68.0 pcf

Design Wall for an Equivalent Fluid Pressure: 68 pcf

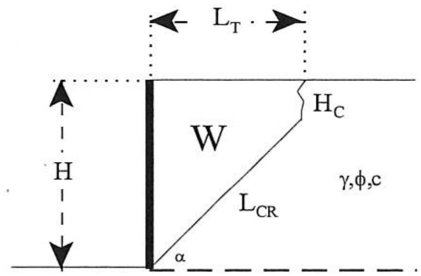
Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

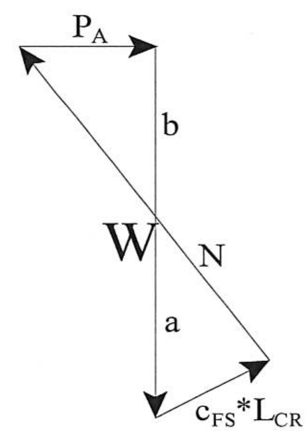
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	40.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
		28
Factored Parameters:	(ϕ_{FS})	10.8 degrees
	(c_{FS})	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	2.9	948	113797.7	57.7	15495.2	98302.5	54889.2
41	2.9	916	109864.2	56.6	14741.9	95122.3	55312.9
42	2.8	884	106082.6	55.5	14050.0	92032.6	55688.0
43	2.8	854	102442.2	54.5	13412.9	89029.3	56016.4
44	2.8	824	98933.4	53.6	12825.2	86108.2	56299.8
45	2.7	796	95547.1	52.7	12281.9	83265.2	56539.6
46	2.7	769	92275.2	51.8	11778.7	80496.6	56737.2
47	2.7	743	89110.3	51.0	11311.8	77798.4	56893.4
48	2.7	717	86045.2	50.2	10877.9	75167.3	57009.0
49	2.7	692	83073.8	49.4	10474.1	72599.7	57084.6
50	2.7	668	80189.9	48.7	10097.5	70092.4	57120.6
51	2.7	645	77388.2	48.0	9745.9	67642.3	57117.0
52	2.7	622	74663.6	47.3	9417.2	65246.4	57074.1
53	2.7	600	72011.4	46.7	9109.5	62901.9	56991.4
54	2.7	579	69427.1	46.1	8821.0	60606.2	56868.6
55	2.7	558	66906.7	45.5	8550.2	58356.6	56705.1
56	2.8	537	64446.4	44.9	8295.7	56150.7	56500.1
57	2.8	517	62042.6	44.4	8056.2	53986.4	56252.6
58	2.8	497	59691.9	43.9	7830.6	51861.2	55961.3
59	2.8	478	57391.1	43.3	7617.9	49773.3	55624.8
60	2.9	459	55137.4	42.9	7416.9	47720.5	55241.2
61	2.9	441	52928.0	42.4	7227.0	45701.0	54808.6
62	3.0	423	50760.1	41.9	7047.1	43712.9	54324.6
63	3.0	405	48631.3	41.5	6876.6	41754.7	53786.5
64	3.1	388	46539.3	41.0	6714.8	39824.5	53191.3
65	3.2	371	44481.8	40.6	6560.9	37920.9	52535.4



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

$P_{A,\ max}$ 57120.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 71.4 pcf

Design Wall for an Equivalent Fluid Pressure: 71 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Alluviumn

Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	26.0 degrees
Cohesion of Retained Soils	(c)	385.0 psf
Factor of Safety	(FS)	1.50

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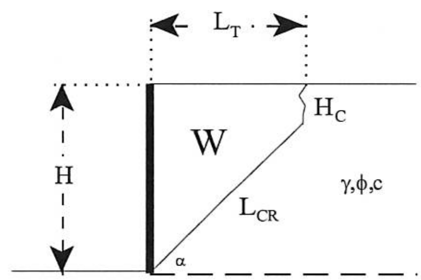
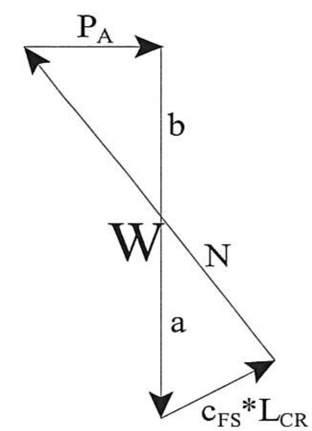
Factored Parameters:	(ϕ_{FS})	18.0 degrees
	(c_{FS})	256.7 psf



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
40	6.8	32	3995.9	5.0	3237.1	758.8	306.4
41	6.6	32	4034.0	5.1	3215.1	818.9	347.4
42	6.5	32	4041.6	5.3	3173.4	868.2	386.3
43	6.3	32	4024.7	5.4	3117.4	907.4	422.9
44	6.2	32	3988.1	5.5	3051.2	937.0	456.7
45	6.1	31	3935.5	5.5	2977.8	957.8	487.8
46	6.0	31	3870.0	5.6	2899.5	970.5	515.8
47	5.9	30	3793.8	5.6	2818.0	975.9	540.7
48	5.8	30	3709.1	5.6	2734.6	974.5	562.3
49	5.8	29	3617.3	5.6	2650.2	967.1	580.8
50	5.7	28	3519.7	5.6	2565.6	954.1	595.9
51	5.7	27	3417.4	5.5	2481.1	936.2	607.7
52	5.7	26	3311.1	5.5	2397.2	913.9	616.2
53	5.7	26	3201.6	5.4	2314.0	887.7	621.3
54	5.7	25	3089.5	5.4	2231.6	857.8	623.0
55	5.7	24	2975.0	5.3	2150.2	824.9	621.3
56	5.7	23	2858.7	5.2	2069.6	789.1	616.3
57	5.7	22	2740.7	5.1	1989.8	751.0	607.8
58	5.7	21	2621.3	5.0	1910.6	710.7	596.1
59	5.8	20	2500.6	4.9	1831.9	668.6	581.0
60	5.8	19	2378.6	4.8	1753.6	625.1	562.6
61	5.9	18	2255.5	4.7	1675.2	580.3	540.9
62	6.0	17	2131.2	4.5	1596.6	534.7	516.1
63	6.1	16	2005.7	4.4	1517.4	488.3	488.1
64	6.2	15	1878.9	4.2	1437.3	441.6	457.1
65	6.3	14	1750.7	4.1	1355.8	394.9	423.3



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

$P_{A,\ max}$ 623.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 12.5 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Alluvium

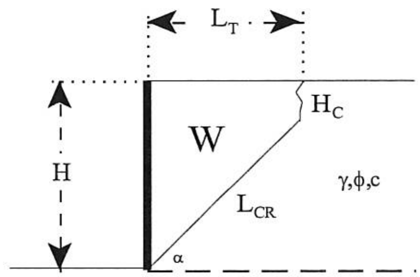
Retaining Wall Design with Level Backfill (Vector Analysis)

Input:

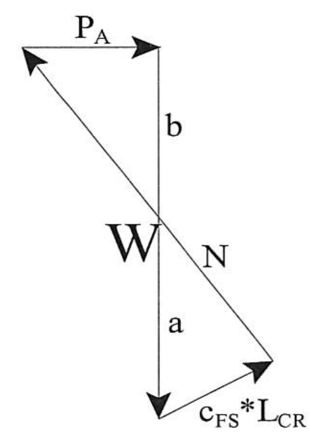
Retaining Wall Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	26.0 degrees
Cohesion of Retained Soils	(c)	385.0 psf
Factor of Safety	(FS)	1.50

28

Factored Parameters:	(ϕ_{FS})	18.0 degrees
	(c_{FS})	256.7 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
40	6.8	211	26341.3	20.5	13379.3	12962.0	5233.8
41	6.6	205	25603.4	20.4	12741.8	12861.6	5456.2
42	6.5	199	24865.6	20.2	12146.2	12719.4	5659.8
43	6.3	193	24131.6	20.1	11589.9	12541.8	5845.1
44	6.2	187	23404.3	19.9	11070.2	12334.1	6012.5
45	6.1	181	22685.5	19.7	10584.4	12101.1	6162.6
46	6.0	176	21976.6	19.5	10130.1	11846.5	6295.7
47	5.9	170	21278.5	19.3	9704.7	11573.8	6412.2
48	5.8	165	20591.7	19.1	9306.1	11285.6	6512.5
49	5.8	159	19916.4	18.8	8932.0	10984.4	6596.9
50	5.7	154	19252.8	18.6	8580.5	10672.3	6665.6
51	5.7	149	18600.8	18.4	8249.8	10351.0	6718.9
52	5.7	144	17960.2	18.2	7938.2	10022.0	6756.8
53	5.7	139	17330.8	18.0	7644.1	9686.7	6779.6
54	5.7	134	16712.1	17.7	7366.1	9346.0	6787.2
55	5.7	129	16103.9	17.5	7102.9	9001.1	6779.8
56	5.7	124	15505.7	17.3	6853.1	8652.6	6757.2
57	5.7	119	14917.1	17.1	6615.7	8301.4	6719.4
58	5.7	115	14337.6	16.8	6389.5	7948.1	6666.4
59	5.8	110	13766.7	16.6	6173.5	7593.2	6597.9
60	5.8	106	13204.0	16.4	5966.7	7237.3	6513.7
61	5.9	101	12648.8	16.1	5768.2	6880.6	6413.5
62	6.0	97	12100.8	15.9	5577.1	6523.7	6297.2
63	6.1	92	11559.3	15.6	5392.4	6166.9	6164.3
64	6.2	88	11023.9	15.4	5213.4	5810.6	6014.4
65	6.3	84	10494.0	15.1	5039.0	5454.9	5847.2



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

$P_{A, max}$ 6787.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 33.9 pcf

Design Wall for an Equivalent Fluid Pressure: 34 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Alluvium

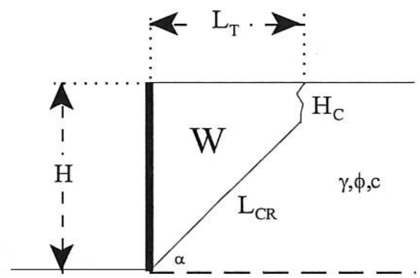
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

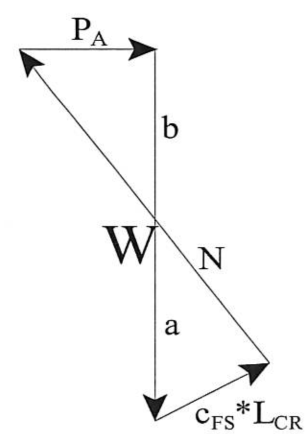
Retaining Wall Height	(H)	30.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	26.0 degrees
Cohesion of Retained Soils	(c)	385.0 psf
Factor of Safety	(FS)	1.50

28

Factored Parameters:	(ϕ_{FS})	18.0 degrees
	(c_{FS})	256.7 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
40	6.8	509	63583.6	36.1	23521.5	40062.1	16176.2
41	6.6	492	61552.4	35.6	22268.5	39283.9	16665.2
42	6.5	477	59572.2	35.2	21119.0	38453.2	17110.7
43	6.3	461	57643.2	34.7	20062.4	37580.8	17514.5
44	6.2	446	55764.6	34.3	19089.2	36675.4	17878.1
45	6.1	431	53935.5	33.8	18191.1	35744.4	18203.1
46	6.0	417	52154.4	33.4	17360.7	34793.7	18490.6
47	5.9	403	50419.6	32.9	16591.5	33828.1	18741.8
48	5.8	390	48729.3	32.5	15877.5	32851.8	18957.6
49	5.8	377	47081.6	32.1	15213.7	31867.9	19138.9
50	5.7	364	45474.7	31.7	14595.4	30879.3	19286.4
51	5.7	351	43906.6	31.3	14018.5	29888.1	19400.5
52	5.7	339	42375.4	30.9	13479.2	28896.2	19481.8
53	5.7	327	40879.3	30.5	12974.3	27905.1	19530.5
54	5.7	315	39416.6	30.1	12500.6	26916.0	19546.9
55	5.7	304	37985.4	29.7	12055.6	25929.9	19530.9
56	5.7	293	36584.1	29.3	11636.6	24947.5	19482.6
57	5.7	282	35211.1	29.0	11241.6	23969.5	19401.7
58	5.7	271	33864.8	28.6	10868.3	22996.4	19288.0
59	5.8	260	32543.6	28.3	10515.0	22028.6	19140.9
60	5.8	250	31246.2	27.9	10179.9	21066.3	18960.1
61	5.9	240	29971.0	27.5	9861.2	20109.8	18744.7
62	6.0	230	28716.7	27.2	9557.5	19159.2	18493.9
63	6.1	220	27482.0	26.8	9267.4	18214.6	18206.8
64	6.2	210	26265.6	26.5	8989.4	17276.1	17882.3
65	6.3	201	25066.1	26.1	8722.3	16343.8	17519.1



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

$P_{A,\ max}$ 19546.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 43.4 pcf

Design Wall for an Equivalent Fluid Pressure: 43 pcf

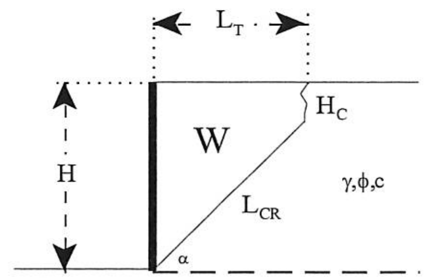
Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Alluvium

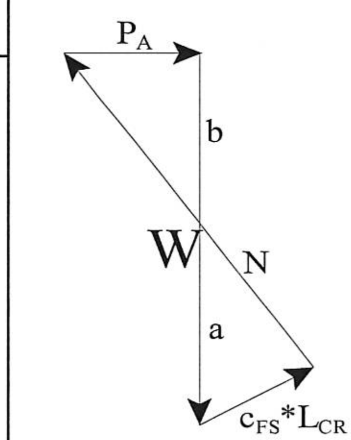
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	40.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	26.0 degrees
Cohesion of Retained Soils	(c)	385.0 psf
Factor of Safety	(FS)	1.50
		28
Factored Parameters:	(ϕ_{FS})	18.0 degrees
	(c_{FS})	256.7 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
40	6.8	926	115722.8	51.6	33663.7	82059.1	33133.7
41	6.6	895	111881.0	50.9	31795.2	80085.8	33974.3
42	6.5	865	108161.5	50.1	30091.9	78069.7	34738.9
43	6.3	836	104559.3	49.4	28534.9	76024.4	35431.0
44	6.2	809	101069.1	48.7	27108.3	73960.8	36053.6
45	6.1	781	97685.5	48.0	25797.8	71887.7	36609.4
46	6.0	755	94403.3	47.3	24591.4	69811.9	37100.6
47	5.9	730	91217.1	46.6	23478.2	67738.9	37529.4
48	5.8	705	88122.0	46.0	22449.0	65673.0	37897.7
49	5.8	681	85112.9	45.3	21495.5	63617.4	38206.8
50	5.7	657	82185.3	44.7	20610.3	61574.9	38458.1
51	5.7	635	79334.6	44.1	19787.2	59547.4	38652.5
52	5.7	612	76556.7	43.6	19020.2	57536.4	38791.0
53	5.7	591	73847.3	43.0	18304.4	55543.0	38874.0
54	5.7	570	71202.8	42.5	17635.1	53567.7	38901.8
55	5.7	549	68619.5	41.9	17008.3	51611.3	38874.6
56	5.7	529	66093.9	41.4	16420.2	49673.7	38792.3
57	5.7	509	63622.7	40.9	15867.5	47755.2	38654.6
58	5.7	490	61202.8	40.4	15347.2	45855.6	38460.8
59	5.8	471	58831.3	39.9	14856.6	43974.7	38210.2
60	5.8	452	56505.2	39.4	14393.0	42112.2	37901.8
61	5.9	434	54222.0	39.0	13954.2	40267.8	37534.3
62	6.0	416	51979.0	38.5	13538.0	38441.0	37106.2
63	6.1	398	49773.7	38.1	13142.4	36631.3	36615.7
64	6.2	381	47603.9	37.6	12765.5	34838.3	36060.8
65	6.3	364	45467.1	37.2	12405.5	33061.6	35439.1



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

$P_{A,\ max}$ 38901.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 48.6 pcf

Design Wall for an Equivalent Fluid Pressure: 49 pcf

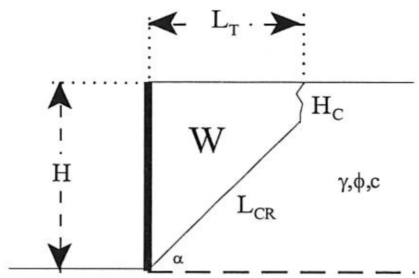
Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

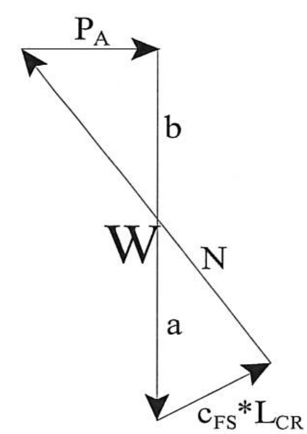
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	10.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.25
		28
Factored Parameters:	(ϕ_{FS})	12.9 degrees
	(c_{FS})	160.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.7	51	6157.5	9.8	3343.4	2814.1	1438.8
41	3.7	50	5978.5	9.7	3202.6	2775.9	1481.0
42	3.6	48	5801.0	9.6	3069.9	2731.1	1518.9
43	3.5	47	5625.5	9.5	2944.8	2680.7	1552.7
44	3.5	45	5452.2	9.4	2826.9	2625.4	1582.5
45	3.5	44	5281.5	9.2	2715.6	2565.9	1608.4
46	3.4	43	5113.4	9.1	2610.5	2502.9	1630.4
47	3.4	41	4948.1	9.0	2511.3	2436.8	1648.6
48	3.4	40	4785.5	8.9	2417.4	2368.0	1663.1
49	3.4	39	4625.6	8.8	2328.5	2297.1	1673.8
50	3.4	37	4468.5	8.7	2244.2	2224.2	1681.0
51	3.3	36	4314.0	8.6	2164.2	2149.8	1684.5
52	3.3	35	4162.1	8.4	2088.1	2074.0	1684.3
53	3.4	33	4012.7	8.3	2015.6	1997.1	1680.5
54	3.4	32	3865.7	8.2	1946.5	1919.3	1673.1
55	3.4	31	3721.0	8.1	1880.3	1840.7	1662.0
56	3.4	30	3578.5	8.0	1817.0	1761.5	1647.3
57	3.4	29	3438.0	7.8	1756.1	1681.9	1628.7
58	3.5	27	3299.5	7.7	1697.6	1601.9	1606.4
59	3.5	26	3162.8	7.6	1641.0	1521.8	1580.2
60	3.5	25	3027.7	7.4	1586.2	1441.5	1550.1
61	3.6	24	2894.2	7.3	1533.0	1361.2	1516.0
62	3.7	23	2762.1	7.2	1481.1	1281.0	1477.7
63	3.7	22	2631.3	7.0	1430.3	1200.9	1435.3
64	3.8	21	2501.5	6.9	1380.4	1121.1	1388.5
65	3.9	20	2372.7	6.7	1331.0	1041.7	1337.2



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

$P_{A,\,max}$ 1684.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 33.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 34 pcf

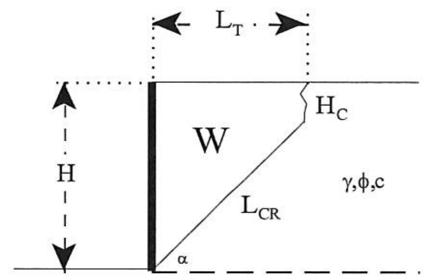
Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

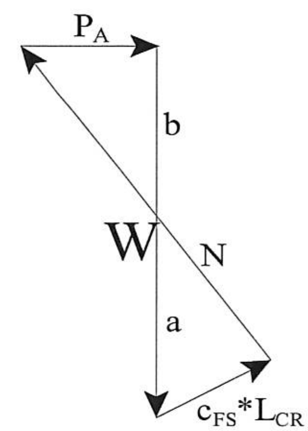
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.25
		28
Factored Parameters:	(ϕ_{FS})	12.9 degrees
	(c_{FS})	160.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.7	230	27609.1	25.3	8672.8	18936.3	9681.9
41	3.7	222	26685.1	24.9	8252.6	18432.5	9833.8
42	3.6	215	25792.0	24.5	7865.1	17926.9	9969.8
43	3.5	208	24928.1	24.1	7507.1	17421.1	10090.5
44	3.5	201	24091.8	23.8	7175.6	16916.2	10196.5
45	3.5	194	23281.5	23.4	6868.2	16413.3	10288.1
46	3.4	187	22495.8	23.0	6582.5	15913.3	10365.9
47	3.4	181	21733.3	22.7	6316.7	15416.7	10430.1
48	3.4	175	20992.7	22.4	6068.8	14924.0	10481.0
49	3.4	169	20272.8	22.0	5837.3	14435.5	10518.9
50	3.4	163	19572.3	21.7	5620.7	13951.6	10543.9
51	3.3	157	18890.1	21.4	5417.8	13472.3	10556.1
52	3.3	152	18225.3	21.1	5227.4	12997.9	10555.6
53	3.4	146	17576.7	20.8	5048.4	12528.3	10542.3
54	3.4	141	16943.5	20.6	4880.0	12063.5	10516.3
55	3.4	136	16324.8	20.3	4721.1	11603.7	10477.4
56	3.4	131	15719.7	20.0	4571.1	11148.6	10425.4
57	3.4	126	15127.4	19.8	4429.1	10698.3	10360.1
58	3.5	121	14547.1	19.5	4294.6	10252.6	10281.3
59	3.5	116	13978.3	19.2	4166.8	9811.5	10188.5
60	3.5	112	13420.0	19.0	4045.2	9374.8	10081.4
61	3.6	107	12871.8	18.7	3929.4	8942.4	9959.5
62	3.7	103	12332.9	18.5	3818.6	8514.3	9822.3
63	3.7	98	11802.7	18.3	3712.5	8090.2	9669.0
64	3.8	94	11280.7	18.0	3610.5	7670.2	9499.0
65	3.9	90	10766.3	17.8	3512.2	7254.0	9311.6



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

$P_{A,\max}$ 10556.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 52.8 pcf

Design Shoring for an Equivalent Fluid Pressure: 53 pcf

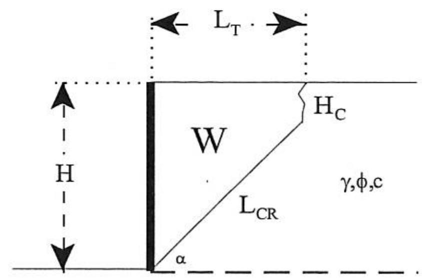
Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

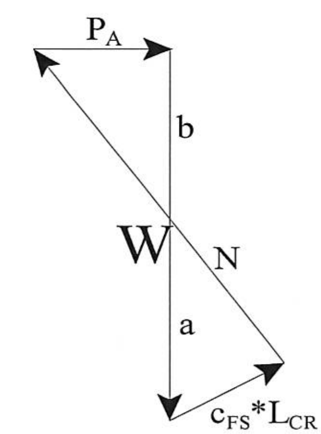
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	30.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.25
		28
Factored Parameters:	(ϕ_{FS})	12.9 degrees
	(c_{FS})	160.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.7	528	63361.7	40.9	14002.2	49359.5	25236.9
41	3.7	510	61196.2	40.2	13302.6	47893.6	25551.4
42	3.6	493	59110.4	39.5	12660.3	46450.1	25832.7
43	3.5	476	57099.2	38.8	12069.3	45029.8	26082.0
44	3.5	460	55157.7	38.1	11524.4	43633.3	26300.6
45	3.5	444	53281.5	37.5	11020.8	42260.7	26489.6
46	3.4	429	51466.5	36.9	10554.5	40912.0	26649.9
47	3.4	414	49708.8	36.4	10122.0	39586.8	26782.2
48	3.4	400	48004.9	35.8	9720.1	38284.7	26887.1
49	3.4	386	46351.4	35.3	9346.0	37005.4	26965.1
50	3.4	373	44745.3	34.8	8997.2	35748.1	27016.6
51	3.3	360	43183.7	34.3	8671.4	34512.2	27041.7
52	3.3	347	41663.8	33.8	8366.7	33297.1	27040.6
53	3.4	335	40183.3	33.4	8081.3	32102.1	27013.4
54	3.4	323	38739.8	32.9	7813.5	30926.3	26959.8
55	3.4	311	37331.0	32.5	7561.9	29769.1	26879.7
56	3.4	300	35954.9	32.1	7325.1	28629.8	26772.6
57	3.4	288	34609.6	31.7	7102.1	27507.5	26638.1
58	3.5	277	33293.2	31.3	6891.6	26401.7	26475.6
59	3.5	267	32004.1	30.9	6692.6	25311.5	26284.2
60	3.5	256	30740.5	30.5	6504.3	24236.3	26063.2
61	3.6	246	29501.0	30.2	6325.7	23175.4	25811.3
62	3.7	236	28284.2	29.8	6156.0	22128.1	25527.5
63	3.7	226	27088.5	29.5	5994.6	21093.9	25210.2
64	3.8	216	25912.7	29.1	5840.6	20072.1	24857.9
65	3.9	206	24755.5	28.8	5693.4	19062.0	24468.7



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

$P_{A,\,max}$ 27041.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 60.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 60 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

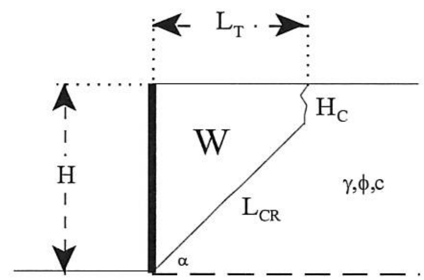
Shoring Design with Level Backfill
(Vector Analysis)

Input:

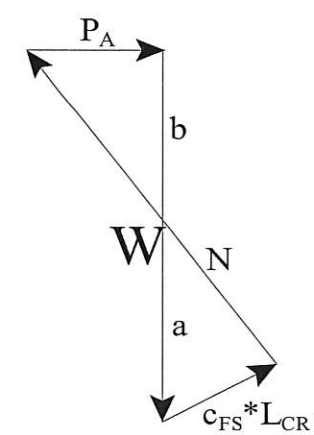
Shoring Height	(H)	40.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.25

28

Factored Parameters:	(ϕ_{FS})	12.9 degrees
	(c_{FS})	160.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
20	11.2	2025	243004.0	84.1	106465.1	136539.0	16958.6
21	9.9	1956	234757.6	84.0	93178.2	141579.4	20099.5
22	8.9	1882	225894.0	83.1	82087.7	143806.3	22982.8
23	8.1	1808	216964.7	81.7	72820.7	144143.9	25624.3
24	7.4	1735	208235.5	80.1	65035.1	143200.4	28043.1
25	6.9	1665	199832.0	78.4	58448.1	141383.9	30258.7
26	6.4	1598	191807.6	76.7	52834.0	138973.6	32289.3
27	6.0	1535	184177.9	74.9	48014.4	136163.4	34151.6
28	5.7	1474	176938.1	73.2	43848.3	133089.8	35860.8
29	5.4	1417	170073.5	71.4	40223.8	129849.7	37430.3
30	5.1	1363	163564.0	69.8	37051.7	126512.3	38872.2
31	4.9	1312	157387.6	68.2	34260.0	123127.7	40197.0
32	4.7	1263	151521.9	66.6	31790.4	119731.5	41414.2
33	4.5	1216	145945.0	65.2	29595.4	116349.6	42532.1
34	4.4	1172	140636.2	63.7	27635.8	113000.3	43558.1
35	4.2	1130	135575.9	62.4	25879.3	109696.6	44498.8
36	4.1	1090	130746.0	61.1	24298.6	106447.4	45359.9
37	4.0	1051	126129.8	59.8	22871.3	103258.5	46146.7
38	3.9	1014	121711.9	58.7	21578.1	100133.8	46863.7
39	3.8	979	117478.1	57.5	20402.8	97075.2	47514.9
40	3.7	945	113415.3	56.4	19331.6	94083.7	48103.8
41	3.7	913	109511.7	55.4	18352.6	91159.1	48633.7
42	3.6	881	105756.1	54.4	17455.5	88300.6	49107.3
43	3.5	851	102138.7	53.5	16631.6	85507.1	49527.0
44	3.5	822	98650.0	52.5	15873.1	82776.9	49895.0
45	3.5	794	95281.5	51.7	15173.4	80108.1	50213.0



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\max}$ 50213.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 62.8 pcf

Design Shoring for an Equivalent Fluid Pressure: 63 pcf

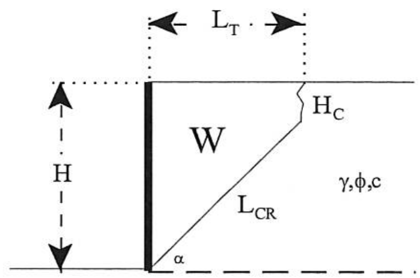
Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Fill Soils

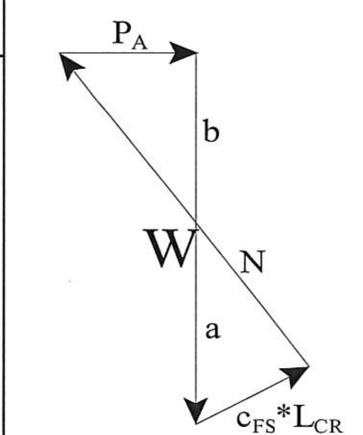
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	50.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	16.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.25
		28
Factored Parameters:	(ϕ_{FS})	12.9 degrees
	(c_{FS})	160.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
20	11.2	3261	391367.8	113.4	143458.2	247909.6	30791.3
21	9.9	3129	375432.4	111.9	124138.3	251294.2	35675.4
22	8.9	2996	359548.7	109.8	108466.8	251081.9	40127.3
23	8.1	2868	344180.7	107.3	95624.5	248556.2	44185.5
24	7.4	2746	329521.5	104.7	84985.9	244535.6	47887.7
25	6.9	2630	315635.4	102.1	76080.5	239554.9	51269.1
26	6.4	2521	302524.1	99.5	68553.3	233970.7	54361.0
27	6.0	2418	290158.8	96.9	62134.4	228024.4	57191.6
28	5.7	2321	278497.4	94.5	56616.2	221881.2	59785.4
29	5.4	2229	267492.1	92.1	51837.3	215654.8	62164.4
30	5.1	2142	257094.8	89.8	47671.0	209423.8	64347.5
31	4.9	2060	247258.7	87.6	44016.6	203242.2	66351.6
32	4.7	1983	237940.0	85.5	40793.1	197146.9	68191.5
33	4.5	1909	229097.7	83.5	37935.3	191162.5	69880.2
34	4.4	1839	220694.5	81.6	35389.6	185304.8	71429.2
35	4.2	1772	212695.9	79.8	33112.2	179583.6	72848.7
36	4.1	1709	205070.6	78.1	31066.6	174004.0	74147.5
37	4.0	1648	197790.3	76.5	29222.4	168567.9	75333.8
38	3.9	1590	190828.8	74.9	27553.9	163274.9	76414.4
39	3.8	1535	184162.5	73.4	26039.6	158123.0	77395.5
40	3.7	1481	177770.0	72.0	24661.0	153109.0	78282.7
41	3.7	1430	171631.6	70.6	23402.6	148229.0	79080.7
42	3.6	1381	165729.2	69.3	22250.7	143478.5	79793.8
43	3.5	1334	160046.6	68.1	21193.9	138852.7	80425.7
44	3.5	1288	154568.6	66.9	20221.8	134346.8	80979.5
45	3.5	1244	149281.5	65.8	19325.9	129955.6	81458.1



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

$P_{A,\ max}$ 81458.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 65.2 pcf

Design Shoring for an Equivalent Fluid Pressure: 65 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Alluvium

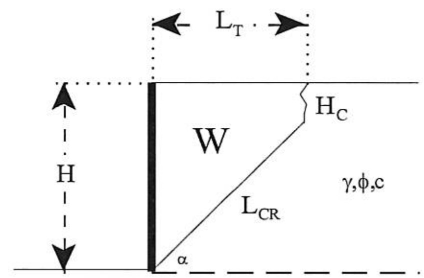
Shoring Design with Level Backfill
(Vector Analysis)

Input:

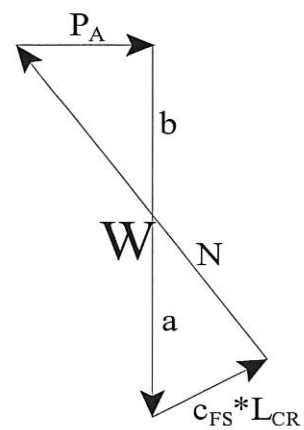
Shoring Height	(H)	30.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	26.0 degrees
Cohesion of Retained Soils	(c)	380.0 psf
Factor of Safety	(FS)	1.25

28

Factored Parameters:	(ϕ_{FS})	21.3 degrees
	(c_{FS})	304.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	9.2	486	60687.9	32.3	28561.8	32126.1	10864.7
41	8.9	472	58997.8	32.1	27024.6	31973.2	11438.6
42	8.6	458	57301.1	31.9	25604.6	31696.5	11967.6
43	8.4	445	55609.7	31.7	24292.8	31316.9	12453.0
44	8.2	431	53932.0	31.4	23080.2	30851.8	12896.1
45	8.0	418	52273.8	31.1	21958.3	30315.5	13298.1
46	7.8	405	50638.9	30.8	20919.0	29719.9	13660.2
47	7.7	392	49030.0	30.5	19955.1	29074.9	13983.4
48	7.5	380	47448.7	30.2	19059.7	28389.0	14268.8
49	7.4	367	45895.7	29.9	18226.7	27669.0	14517.3
50	7.3	355	44371.4	29.6	17450.6	26920.8	14729.6
51	7.3	343	42875.7	29.2	16726.2	26149.5	14906.3
52	7.2	331	41408.1	28.9	16049.1	25359.0	15048.1
53	7.2	320	39967.9	28.6	15415.0	24553.0	15155.3
54	7.1	308	38554.5	28.3	14820.1	23734.4	15228.4
55	7.1	297	37166.9	27.9	14261.2	22905.7	15267.5
56	7.1	286	35804.1	27.6	13734.9	22069.2	15272.9
57	7.1	276	34465.1	27.3	13238.5	21226.6	15244.4
58	7.2	265	33148.7	26.9	12769.3	20379.4	15182.0
59	7.2	255	31853.9	26.6	12324.8	19529.0	15085.6
60	7.2	245	30579.5	26.3	11902.9	18676.6	14954.7
61	7.3	235	29324.4	25.9	11501.4	17823.0	14789.0
62	7.4	225	28087.5	25.6	11118.3	16969.2	14588.0
63	7.5	215	26867.5	25.2	10751.7	16115.8	14351.1
64	7.6	205	25663.4	24.9	10399.9	15263.5	14077.4
65	7.8	196	24474.0	24.5	10061.0	14413.0	13766.1



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

$P_{A,\,max}$ 15272.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 33.9 pcf

Design Shoring for an Equivalent Fluid Pressure: 34 pcf

Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Alluvium

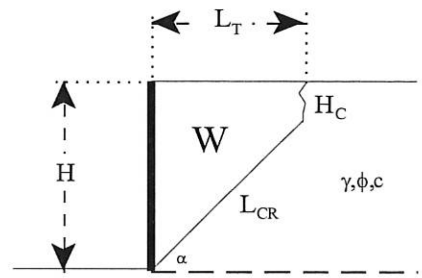
Shoring Design with Level Backfill
(Vector Analysis)

Input:

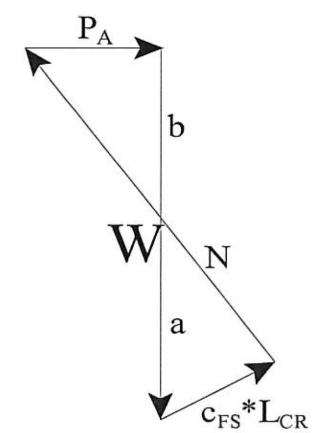
Shoring Height	(H)	40.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	26.0 degrees
Cohesion of Retained Soils	(c)	380.0 psf
Factor of Safety	(FS)	1.25

28

Factored Parameters:	(ϕ_{FS})	21.3 degrees
	(c_{FS})	304.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	9.2	903	112827.1	47.9	42314.5	70512.6	23846.6
41	8.9	875	109326.4	47.4	39839.7	69486.7	24859.3
42	8.6	847	105890.4	46.9	37586.7	68303.7	25789.3
43	8.4	820	102525.8	46.4	35531.1	66994.8	26640.1
44	8.2	794	99236.5	45.8	33651.3	65585.1	27414.6
45	8.0	768	96023.8	45.3	31928.5	64095.2	28115.8
46	7.8	743	92887.8	44.7	30346.1	62541.7	28746.1
47	7.7	719	89827.6	44.2	28889.4	60938.1	29307.8
48	7.5	695	86841.3	43.7	27545.6	59295.7	29803.1
49	7.4	671	83927.0	43.2	26303.4	57623.6	30233.8
50	7.3	649	81082.0	42.6	25152.7	55929.3	30601.3
51	7.3	626	78303.7	42.1	24084.7	54219.0	30907.1
52	7.2	605	75589.3	41.6	23091.6	52497.7	31152.2
53	7.2	583	72935.9	41.1	22166.3	50769.7	31337.6
54	7.1	563	70340.8	40.6	21302.5	49038.2	31463.8
55	7.1	542	67801.0	40.1	20494.7	47306.3	31531.4
56	7.1	523	65313.9	39.7	19737.9	45576.0	31540.6
57	7.1	503	62876.6	39.2	19027.4	43849.2	31491.4
58	7.2	484	60486.7	38.7	18359.2	42127.5	31383.7
59	7.2	465	58141.5	38.3	17729.5	40412.0	31217.0
60	7.2	447	55838.5	37.8	17134.9	38703.7	30990.8
61	7.3	429	53575.4	37.4	16572.2	37003.2	30704.3
62	7.4	411	51349.7	36.9	16038.5	35311.2	30356.3
63	7.5	393	49159.3	36.5	15531.2	33628.1	29945.7
64	7.6	376	47001.8	36.0	15047.5	31954.2	29471.0
65	7.8	359	44875.0	35.6	14585.2	30289.8	28930.3



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

$P_{A,\,max}$ 31540.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 39.4 pcf

Design Shoring for an Equivalent Fluid Pressure: 39 pcf

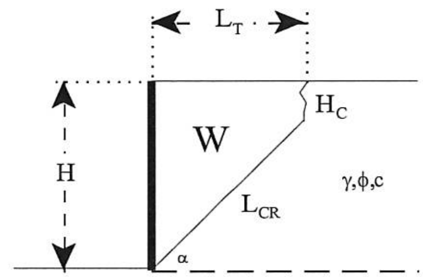
Geotechnologies, Inc.

Project: Kilroy Realty Corportion
File No.: 21800
Description: Alluvium

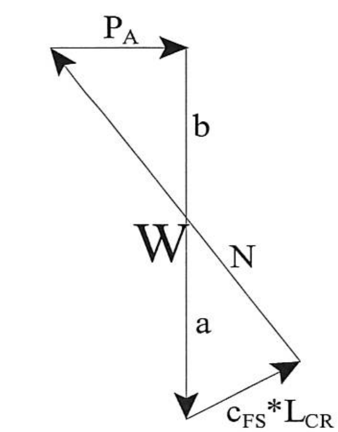
Shoring Design with Level Backfill (Vector Analysis)

Input:

Shoring Height	(H)	50.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	26.0 degrees
Cohesion of Retained Soils	(c)	380.0 psf
Factor of Safety	(FS)	1.25
		28
Factored Parameters:	(ϕ_{FS})	21.3 degrees
	(c_{FS})	304.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	9.2	1439	179863.3	63.4	56067.3	123796.0	41866.4
41	8.9	1392	174034.6	62.6	52654.9	121379.7	43424.3
42	8.6	1347	168362.3	61.8	49568.8	118793.5	44852.7
43	8.4	1303	162846.6	61.0	46769.4	116077.2	46157.5
44	8.2	1260	157485.0	60.2	44222.4	113262.6	47343.9
45	8.0	1218	152273.8	59.4	41898.8	110375.0	48416.7
46	7.8	1178	147207.8	58.7	39773.2	107434.6	49380.2
47	7.7	1138	142281.5	57.9	37823.7	104457.8	50238.4
48	7.5	1100	137489.1	57.1	36031.6	101457.5	50994.5
49	7.4	1063	132824.4	56.4	34380.1	98444.3	51651.5
50	7.3	1026	128281.4	55.7	32854.9	95426.5	52211.9
51	7.3	991	123854.1	55.0	31443.3	92410.8	52678.1
52	7.2	956	119536.6	54.3	30134.1	89402.5	53051.6
53	7.2	923	115323.3	53.6	28917.6	86405.8	53334.0
54	7.1	890	111208.8	53.0	27784.9	83423.9	53526.3
55	7.1	858	107187.7	52.3	26728.3	80459.4	53629.2
56	7.1	826	103255.0	51.7	25740.8	77514.1	53643.2
57	7.1	795	99405.8	51.1	24816.3	74589.5	53568.3
58	7.2	765	95635.6	50.5	23949.1	71686.5	53404.2
59	7.2	736	91939.9	49.9	23134.1	68805.8	53150.3
60	7.2	707	88314.5	49.4	22366.8	65947.7	52805.6
61	7.3	678	84755.3	48.8	21643.0	63112.3	52368.9
62	7.4	650	81258.4	48.2	20958.8	60299.6	51838.3
63	7.5	623	77820.1	47.7	20310.6	57509.5	51212.0
64	7.6	595	74436.7	47.1	19695.2	54741.5	50487.4
65	7.8	569	71104.8	46.6	19109.4	51995.4	49661.8



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

$P_{A,\,max}$ 53643.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 42.9 pcf

Design Shoring for an Equivalent Fluid Pressure: 43 pcf

COLUMN SETTLEMENT CALCULATIONS

(AT-GRADE FOOTINGS-SEE INSTRUCTIONS BELOW)

CLIENT: Kilroy Realty Partners
FILE NUMBER: 21800

F.F. ELEV = 115 FT
Soil Type: Alluvium

COLUMN LOAD	950	(KIPS)
DESIGN BEARING VALUE	4000	(PSF)
DEAD LOAD	75	(PERCENT)
LIVE LOAD	25	(PERCENT)
FOOTING DEPTH (DF)	4	(FEET)
SOIL DENSITY (G)	125	(PCF)
FOOTING SIZE(SQUARE)(a)	15.0	(FEET)
REAL LOAD (PR)	3500	(PSF)

Di = INITIAL DEPTH OF SLICE (FEET)
Df = FINAL DEPTH OF SLICE (FEET)
D1 = AVG. DEPTH OF SLICE BELOW ORIG. GRADE (FEET)
D2 = AVG. DEPTH BELOW FOOTING (FEET)
PV = VERTICAL PRESSURE (PERCENTAGE OF REAL LOAD)
PV FROM WESTERGAARD CHARTS AT DEPTH D2
ENTER THE PERCENTAGE OF CONSOLIDATION FROM PLATE C AT THE INITIAL TO FINAL PRESSURES.

Di	Df	ELEVATION FTG. BOT. (FT)	D1	(z) D2	a/z	PV	SOIL PRESSURES (KIPS) INITIAL	FINAL	PERCENT CONSOL.	SLICE THICKNESS (INCHES)	SETTLE. (INCHES)	SAMPLE
5	10	105	7.5	3.5	4.3	71	0.9	3.4	2.6	60	1.56	B2 @42.5
10	20	95	15	11	1.4	33	1.9	3.0	0.4	120	0.48	B6 @45
20	30	85	25	21	0.7	11	3.1	3.5	0.1	120	0.12	B7 @55
30	40	75	35	31	0.5	7	4.4	4.6	0.1	120	0.12	B2 @67.5
40	50	65	45	41	0.4	5	5.6	5.8	0.1	120	0.12	B5 @72.5

2.40	INCHES
x 2/3 REDUCTION	
1.60	**INCHES**

COLUMN SETTLEMENT CALCULATIONS

(AT-GRADE FOOTINGS-SEE INSTRUCTIONS BELOW)

CLIENT: Kilroy Realty Corporation
FILE NUMBER: 21800

F.F. ELEV = 115 FT
Soil Type: Dense Alluvium

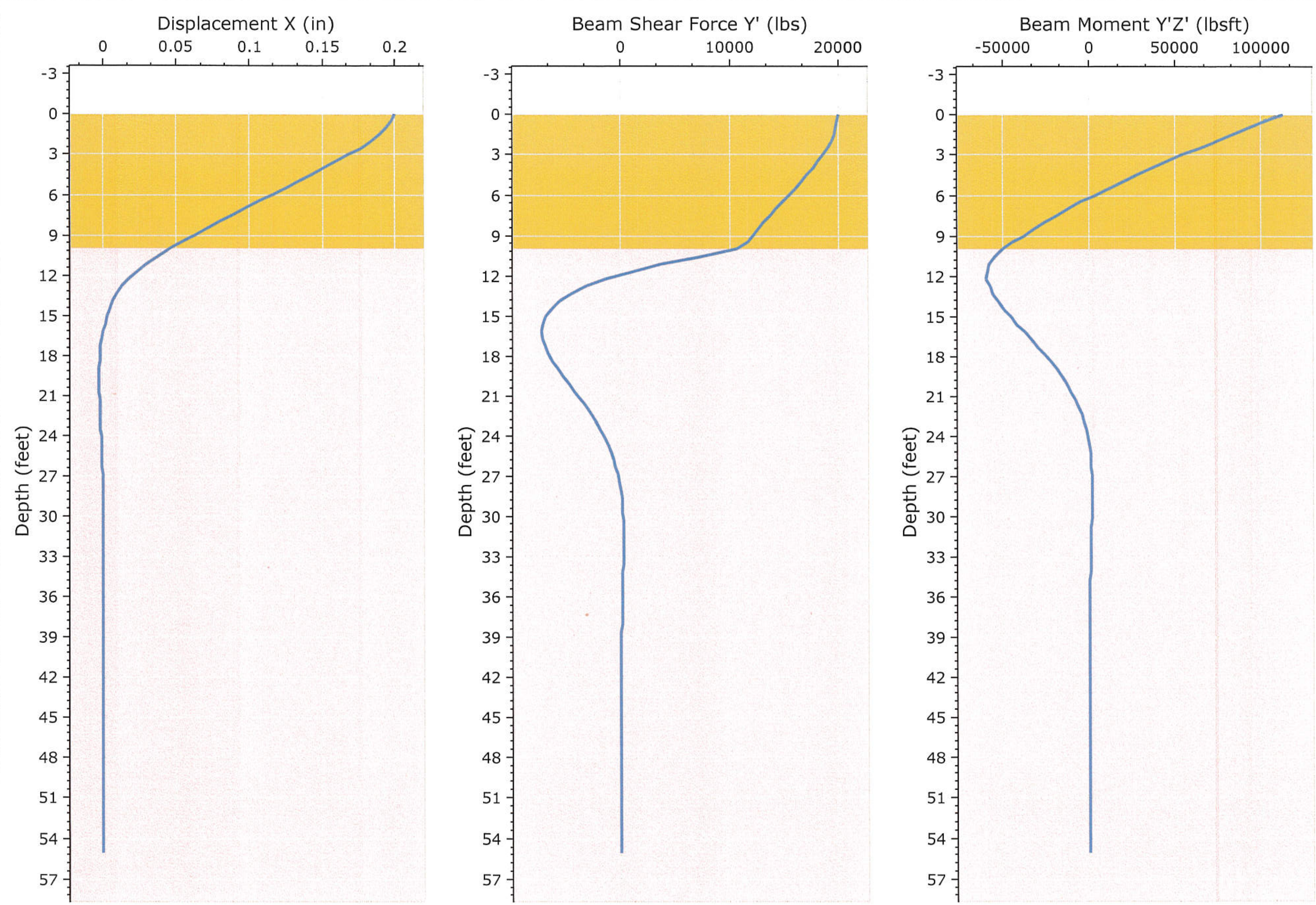
COLUMN LOAD	58750	(KIPS)
DESIGN BEARING VALUE	4000	(PSF)
DEAD LOAD	75	(PERCENT)
LIVE LOAD	25	(PERCENT)
FOOTING DEPTH (DF)	4	(FEET)
SOIL DENSITY (G)	125	(PCF)
FOOTING SIZE(SQUARE)(a)	200.0	(FEET)
REAL LOAD (PR)	3500	(PSF)

Note: Dense Alluvium

Di = INITIAL DEPTH OF SLICE (FEET)
Df = FINAL DEPTH OF SLICE (FEET)
D1 = AVG. DEPTH OF SLICE BELOW ORIG. GRADE (FEET)
D2 = AVG. DEPTH BELOW FOOTING (FEET)
PV = VERTICAL PRESSURE (PERCENTAGE OF REAL LOAD)
PV FROM WESTERGAARD CHARTS AT DEPTH D2
ENTER THE PERCENTAGE OF CONSOLIDATION FROM PLATE C AT THE INITIAL TO FINAL PRESSURES.

Di	Df	ELEVATION FTG. BOT. (FT)	D1	(z) D2	a/z	PV	SOIL PRESSURES (KIPS) INITIAL	FINAL	PERCENT CONSOL.	SLICE THICKNESS (INCHES)	SETTLE. (INCHES)	SAMPLE
10	20	95	15	11	18.2	92	1.9	5.1	0.3	120	0.36	B7 @45'
20	40	75	30	26	7.7	83	3.8	6.7	0.7	240	1.68	B6 @ 45'
40	80	35	60	56	3.6	66	7.5	9.8	0.3	480	1.44	B5 @72.5
80	120	-5	100	96	2.1	46	12.5	14.1	0.2	480	0.96	B5 @82.5
120	200	-85	160	156	1.3	30	20.0	21.1	0.1	960	0.96	B2 @ 92.5

5.40	INCHES
x 2/3 REDUCTION	
3.60	**INCHES**



rocscience	Project	File No 21800 Kilroy Realty	
	Analysis Description	24-inch diameter, 20K lateral Load	
	Drawn By: RTK	Company: Geotechnologies, Inc.	
RSPILE 2.013	Date: 7/3/2019, 3:03:05 PM	File Name: 21800 24in fixed.rspile2	

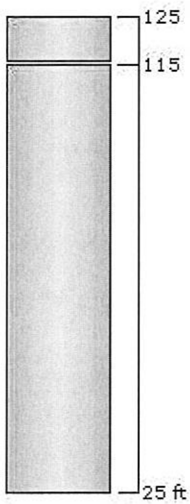
RSPile Analysis Information
File No 21800 Kilroy Realty

Project Summary

Document Name	21800 24in fixed.rspile2
Project Title	File No 21800 Kilroy Realty
Analysis	24-inch diameter, 20K lateral Load
Author	RTK
Company	Geotechnologies, Inc.
Date Created	7/3/2019, 3:03:05 PM
Last saved with RSPile version	2.013

Soil Layers

Layer Name	Color	Layer Type	Thickness [ft]	Depth [ft]
Fill		Silt (Cemented C - Phi Coil)	10	-125
Alluvium		Silt (Cemented C - Phi Coil)	90	-115



Soil Properties

Fill

Property	Value
Name	Fill
Color	
Soil Type	Silt (Cemented C - Phi Coil)
Unit Weight (lbs/ft3)	115
Sat. Unit Weight (lbs/ft3)	115
Friction Angle (degrees)	16
Initial Stiffness (lbs/ft3)	20000
Cohesion (psf)	100

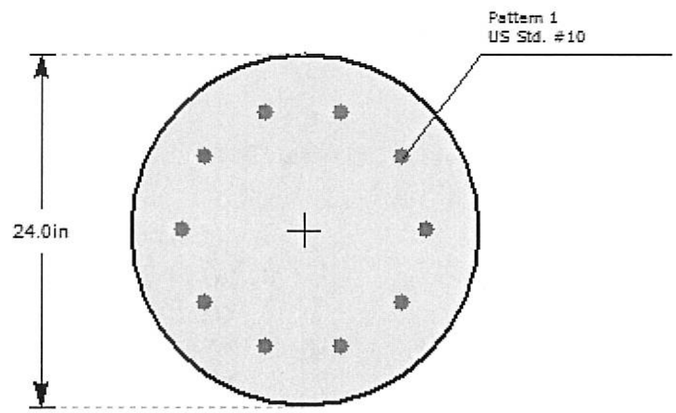
Alluvium

Property	Value
Name	Alluvium
Color	
Soil Type	Silt (Cemented C - Phi Coil)
Unit Weight (lbs/ft3)	125
Sat. Unit Weight (lbs/ft3)	62
Friction Angle (degrees)	385
Initial Stiffness (lbs/ft3)	200000
Cohesion (psf)	26

Pile Properties

Pile Property 1

Property	Value
Name	Pile Property 1
Color	
Pile Type	Reinforced Concrete
Pile Cross Section	Circle
Diameter (ft)	2
Compressive Strength (psf)	576000



Reinforcement

#	Location X (in)	Location Y (in)	Rebar Size	Bundled	Yield Stress (psf)	Elastic Modulus (psf)
1	8.365	0	US Std. #10		8640000.00	4176000000.00
2	6.767	4.917	US Std. #10		8640000.00	4176000000.00
3	2.585	7.956	US Std. #10		8640000.00	4176000000.00
4	-2.585	7.956	US Std. #10		8640000.00	4176000000.00
5	-6.767	4.917	US Std. #10		8640000.00	4176000000.00
6	-8.365	1.024e-015	US Std. #10		8640000.00	4176000000.00
7	-6.767	-4.917	US Std. #10		8640000.00	4176000000.00
8	-2.585	-7.956	US Std. #10		8640000.00	4176000000.00
9	2.585	-7.956	US Std. #10		8640000.00	4176000000.00
10	6.767	-4.917	US Std. #10		8640000.00	4176000000.00

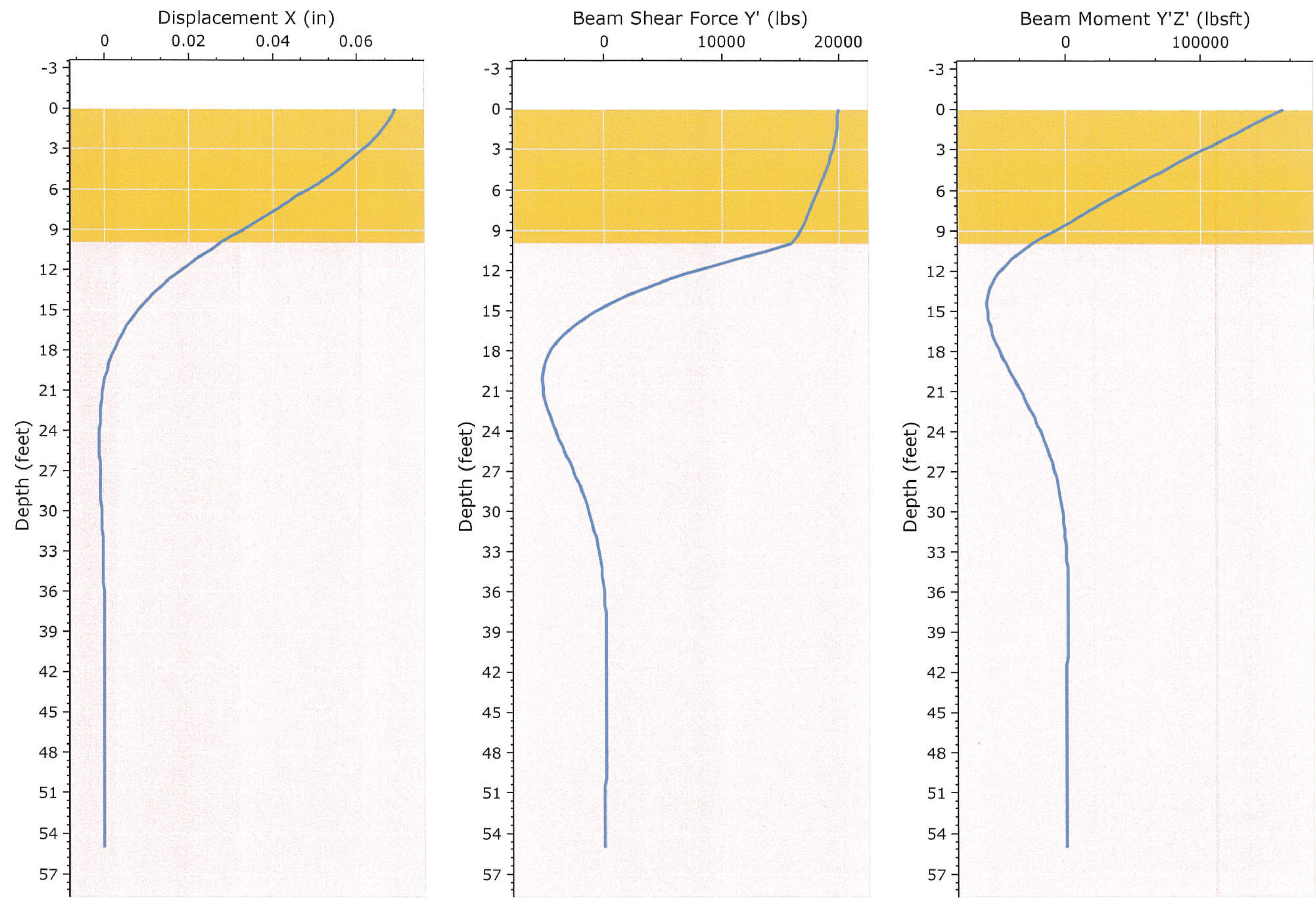
Pile Settings

Pile 1

General	
Property	Pile Property 1
Location	0.121, 0.088
Elevation:	125 (ft)
Length:	55 (ft)

Orientation	
Elevation (ft)	125
Length (ft)	55
Ground Slope Angle (°)	0
Alpha Angle (°)	0
Beta Angle (°)	90
Rotation Angle (°)	0

Loading	
Loading Type	Static
Load Factor Profile	None
Type	**Value**
Shear X, (lbs)	20000
Slope Y, (deg)	0



rocscience

RSPILE 2.013

Project	File No 21800 Kilroy Realty		
Analysis Description	36-inch diameter, 20K lateral Load		
Drawn By	RTK	*Company*	Geotechnologies, Inc.
Date	7/3/2019, 3:03:05 PM	*File Name*	21800 36in fixed.rspile2

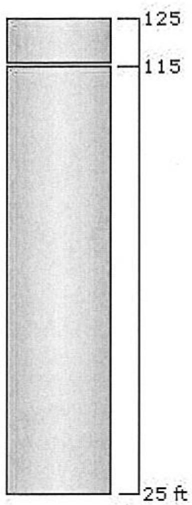
RSPile Analysis Information
File No 21800 Kilroy Realty

Project Summary

Document Name	21800 36in fixed.rspile2
Project Title	File No 21800 Kilroy Realty
Analysis	36-inch diameter, 20K lateral Load
Author	RTK
Company	Geotechnologies, Inc.
Date Created	7/3/2019, 3:03:05 PM
Last saved with RSPile version	2.013

Soil Layers

Layer Name	Color	Layer Type	Thickness [ft]	Depth [ft]
Fill		Silt (Cemented C - Phi Coil)	10	-125
Alluvium		Silt (Cemented C - Phi Coil)	90	-115



Soil Properties

Fill

Property	Value
Name	Fill
Color	
Soil Type	Silt (Cemented C - Phi Coil)
Unit Weight (lbs/ft3)	115
Sat. Unit Weight (lbs/ft3)	115
Friction Angle (degrees)	16
Initial Stiffness (lbs/ft3)	20000
Cohesion (psf)	100

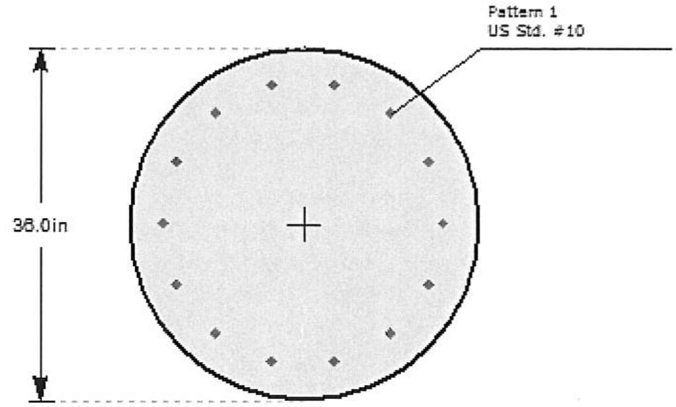
Alluvium

Property	Value
Name	Alluvium
Color	
Soil Type	Silt (Cemented C - Phi Coil)
Unit Weight (lbs/ft3)	125
Sat. Unit Weight (lbs/ft3)	62
Friction Angle (degrees)	385
Initial Stiffness (lbs/ft3)	200000
Cohesion (psf)	26

Pile Properties

Pile Property 1

Property	Value
Name	Pile Property 1
Color	
Pile Type	Reinforced Concrete
Pile Cross Section	Circle
Diameter (ft)	3
Compressive Strength (psf)	576000



Reinforcement

#	Location X (in)	Location Y (in)	Rebar Size	Bundled	Yield Stress (psf)	Elastic Modulus (psf)
1	14.37	0	US Std. #10		8640000.00	4176000000.00
2	12.94	6.233	US Std. #10		8640000.00	4176000000.00
3	8.956	11.23	US Std. #10		8640000.00	4176000000.00
4	3.197	14	US Std. #10		8640000.00	4176000000.00
5	-3.197	14	US Std. #10		8640000.00	4176000000.00
6	-8.956	11.23	US Std. #10		8640000.00	4176000000.00
7	-12.94	6.233	US Std. #10		8640000.00	4176000000.00
8	-14.37	1.759e-015	US Std. #10		8640000.00	4176000000.00
9	-12.94	-6.233	US Std. #10		8640000.00	4176000000.00
10	-8.956	-11.23	US Std. #10		8640000.00	4176000000.00
11	-3.197	-14	US Std. #10		8640000.00	4176000000.00
12	3.197	-14	US Std. #10		8640000.00	4176000000.00
13	8.956	-11.23	US Std. #10		8640000.00	4176000000.00
14	12.94	-6.233	US Std. #10		8640000.00	4176000000.00

Pile Settings

Pile 1

General	
Property	Pile Property 1
Location	0.121, 0.088
Elevation:	125 (ft)
Length:	55 (ft)

Orientation	
Elevation (ft)	125
Length (ft)	55
Ground Slope Angle (°)	0
Alpha Angle (°)	0
Beta Angle (°)	90
Rotation Angle (°)	0

Loading	
Loading Type	Static
Load Factor Profile	None
Type	**Value**
Shear X, (lbs)	20000
Slope Y, (deg)	0

Soil Corrosivity Evaluation Report for Kilroy Realty

July 6, 2019

Prepared for:
Reinard Knur
Geotechnologies, Inc
439 Western Ave.
Glendale, CA, 91201
rknur@geoteq.com

Project X Job #: S190702B
Client Job or PO #: 21800

Contents

1 Executive Summary

A corrosion evaluation of the soils at Kilroy Realty was performed to provide corrosion control recommendations for general construction materials. The site is located at 1633 26th St, Santa Monica, CA 90404 . Six (6) samples were tested to a depth of 42.5 ft. Site ground water and topography information was provided via Geotechnologies, Inc and determined to be 40 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 1,608 ohm-cm and 34,170 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 482 ohm-cm to 3,417 ohm-cm. The worst of these values is considered to be severely corrosive to general metals.

PH levels ranged between 7.4 to 8.8 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 16 mg/kg to 571 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals.

Sulfates ranged between 39 mg/kg to 565 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 1.1 mg/kg to 5.7 mg/kg. Nitrates ranged between 0.8 mg/kg to 195.0 mg/kg. Concentrations of these elements were high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be trace. REDOX ranged between + 129 mV to + 167 mV. Though sulfides were detected, the probability of corrosive bacteria was determined to be low due to very 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 565 mg/kg or 0.0565 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 571 mg/kg.

Chloride levels in these samples are enough to cause significant corrosion of metals in soil and in cement. Corrosion protection options can be one of the following:

1) Provide 3 inches minimum cement cover between soil and steel materials where cement will be placed in contact with onsite soils. Use Type II cement + Pozzolan or slag content per ACI 318-14 Table 19.3.2.1 to continue use of steel materials encased in cement[3], or
2) Provide waterproof coating with minimum 15 mil thickness to cement that is in contact with soil, or
3) Use epoxy coated steel such as Purple fusion bonded epoxy (FBE) (ASTM A934) or equivalent, or
4) Mix a chloride corrosion inhibitor such as DCI or equivalent into the cement with cement mix designed to protect embedded steel and iron that should be based on 1) Chloride content of 571 ppm in the soil, 2) desired service life, 3) cement cover. We defer to the manufacturer of the chloride inhibitor for determination of the proper admixture ratio to cement, or
5) Apply Cathodic Protection

[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65
[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*
[3] *Standard Requirements for Design of Shallow PT Cement foundations on Expansive soil*

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 moderate chloride levels. Per Nickel Institute guidelines, 316 Stainless steels should only be used in these soils.

2.4 Steel Post Tensioning Systems

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

Soil with high chloride levels is considered an aggressive environment for post-tensioning strands and anchors. Due to the high chloride levels determined on-site, implement all of the following measures:[4,5,6]

1) Completely encapsulate the tendon and anchor with polyethylene to create a watertight seal. Epoxy coated hardware would be equivalent to polyethylene coated and impermeable waterproofing system.
2) Add grease caps to the ends to provide extra protection against corrosion due to high chloride concentrations.
3) All components exposed to the job site should be protected within one working day after their exposure during installation.
4) Ensure the minimum cement cover over the tendon tail is 1-inch, or greater if required by the applicable building code.
5) Caps and sleeves should be installed within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.
6) Inspect the following to ensure the encapsulated system is completely watertight:
   a) Sheathing: Verify that all damaged areas, including pin-holes, are repaired.
   b) Stressing tails: After removal, ensure they are cut to a length for proper installation of P/T coating filled end caps.

[4] *Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.*
[5] *Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2000.*
[6] *ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Cement Institute (ACI), 2001*

c) End caps: Ensure proper installation before patching the pocket former recesses.

d) Patching: Ensure the patch is of an approved material and mix design, and installed void-free.

e) Limit the access of direct runoff onto the anchorage area by designing proper drainage.

f) Provide at least 2 inches of space between finish grade and the anchorage area, or more if required by applicable building codes.

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.[7] 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 pile per disturbed soil corrosion rates, or

2) Sacrificial metal by use of thicker pile per non-disturbed soil corrosion rates and coat portion of piles that will be minimum 12 inches below grade and 12 inches above finished grade with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or

3) Cement coated steel piles with minimum 3 inch cover of Type II cement+ Pozzolan per 2012 IBC Table 1904.2.3, and 0.40 water-cement ratio by weight and 4,000 psi strength per 2012 IBC Table 1904.2.2(1) and ACI 318 Table 4.2.2 to prevent chloride intrusion from soil to encased steel or mix chloride corrosion inhibitor such as DCI or equivalent into the cement with cement mix designed to protect embedded steel and iron that should

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

be based on 1) Chloride content of 571 ppm in the soil, 2) desired service life, 3) cement cover. We defer to the manufacturer of the chloride inhibitor for determination of the proper admixture ratio to cement.

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.[8] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[9]

Expected Corrosion Rate for Steel = 2.81 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 2.02 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

- 18.5 Years to Perforation for a 18 gage metal culvert
- 24.1 Years to Perforation for a 16 gage metal culvert
- 29.6 Years to Perforation for a 14 gage metal culvert
- 40.7 Years to Perforation for a 12 gage metal culvert
- 51.8 Years to Perforation for a 10 gage metal culvert
- 62.9 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 = 2.02 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

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

2.6 Steel Storage tanks

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

2.7 Steel Pipelines

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

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

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

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

Prevent dissimilar metal corrosion cells per NACE SP0286:

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

The corrosivity at this site is very corrosive to steel. Any piping that must be jack-bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Wax tape, or
2) Coal tar enamel, or
3) Fusion bonded epoxy
4) And install cathodic protection system per NACE SP0169.

   Or instead of CP and Dielectric coating

5) Apply 3 inch coating of Type II cement + mix chloride corrosion inhibitor such as DCI or equivalent into the cement with cement mix designed to protect embedded steel and iron that should be based on 1) Chloride content of 571 ppm in the soil, 2) desired service life, 3) cement cover. We defer to the manufacturer of the chloride inhibitor for determination of the proper admixture ratio to cement.

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 very 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) And install cathodic protection system per NACE SP0169.

   Or instead of CP and Dielectric coating

7) Apply 3 inch coating of Type II cement + mix chloride corrosion inhibitor such as DCI or equivalent into the cement with cement mix designed to protect embedded steel and iron that should be based on 1) Chloride content of 571 ppm in the soil, 2) desired service life, 3) cement cover. We defer to the manufacturer of the chloride inhibitor for determination of the proper admixture ratio to cement.

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 16 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 very 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) And install cathodic protection system per NACE SP0169.

   Or instead of CP and Dielectric coating

7) Apply 3 inch coating of Type II cement + mix chloride corrosion inhibitor such as DCI or equivalent into the cement with cement mix designed to protect embedded steel and iron that should be based on 1) Chloride content of 571 ppm in the soil, 2) desired service life, 3) cement cover. We defer to the manufacturer of the chloride inhibitor for determination of the proper admixture ratio to cement.

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

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

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
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 very 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) And install cathodic protection system per NACE SP0169.

   Or instead of CP and Dielectric coating

7) Apply 3 inch coating of Type II cement + mix chloride corrosion inhibitor such as DCI or equivalent into the cement with cement mix designed to protect embedded steel and iron that should be based on 1) Chloride content of 571 ppm in the soil, 2) desired service life,

3) cement cover. We defer to the manufacturer of the chloride inhibitor for determination of the proper admixture ratio to cement.

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 7.4. Copper is greatly affected by pH, ammonia and nitrate concentrations[10]. The highest nitrate concentration was 195.0 mg/kg and the highest ammonia concentration was 5.7 mg/kg at this site.

These 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

[10] 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.

2.11.2 Brass Fittings

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

These 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 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:[11]

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

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

Size (AWG)	Diameter (mils)	Est. Time to penetration (Yrs)
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 unsafe for aluminum. Soils at this site were determined to be too high in chlorides for aluminum. Soil contact with aluminum alloys should be avoided at this site. This can be achieved with:

1) Impermeable minimum 20 mil polyethylene coatings, or
2) Epoxy coatings with minimum 20 mil thickness free of scratches and defects, or
3) Wax tape

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

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints. 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
California No. M37102
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4 SOIL ANALYSIS LAB RESULTS

Client: Geotechnologies, Inc
Job Name: Kilroy Realty
Client Job Number: 21800
Project X Job Number: S190702B
July 6, 2019

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates**		**Chlorides**		**Resistivity**		**pH**	**Redox**	**Sulfide**	**Nitrate**	**Ammonia**
						As Rec'd	Minimum					
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)
B1	25.0	38.6	0.00386	16.2	0.0016	13,400	3,417	8.8	129.0	5.4	3.5	1.1
B2	42.5	133.6	0.0134	54.8	0.0055	1,608	1,608	8.5	148.0	0.7	0.8	5.7
B6	35.0	65.1	0.0065	61.2	0.0061	1,742	1,675	8.4	153.0	1.0	ND	4.5
B1	1.0-5.0	565.3	0.0565	570.6	0.0571	8,040	670	7.4	167.0	0.5	195.0	3.2
B2	1.0-5.0	474.1	0.0474	511.9	0.0512	34,170	482	8.4	133.0	0.2	2.0	ND
B5	1.0-5.0	411.6	0.0412	67.5	0.0067	16,750	1,139	7.8	156.0	0.2	14.1	4.4

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, 1633 26th St, Santa Monica, CA 90404

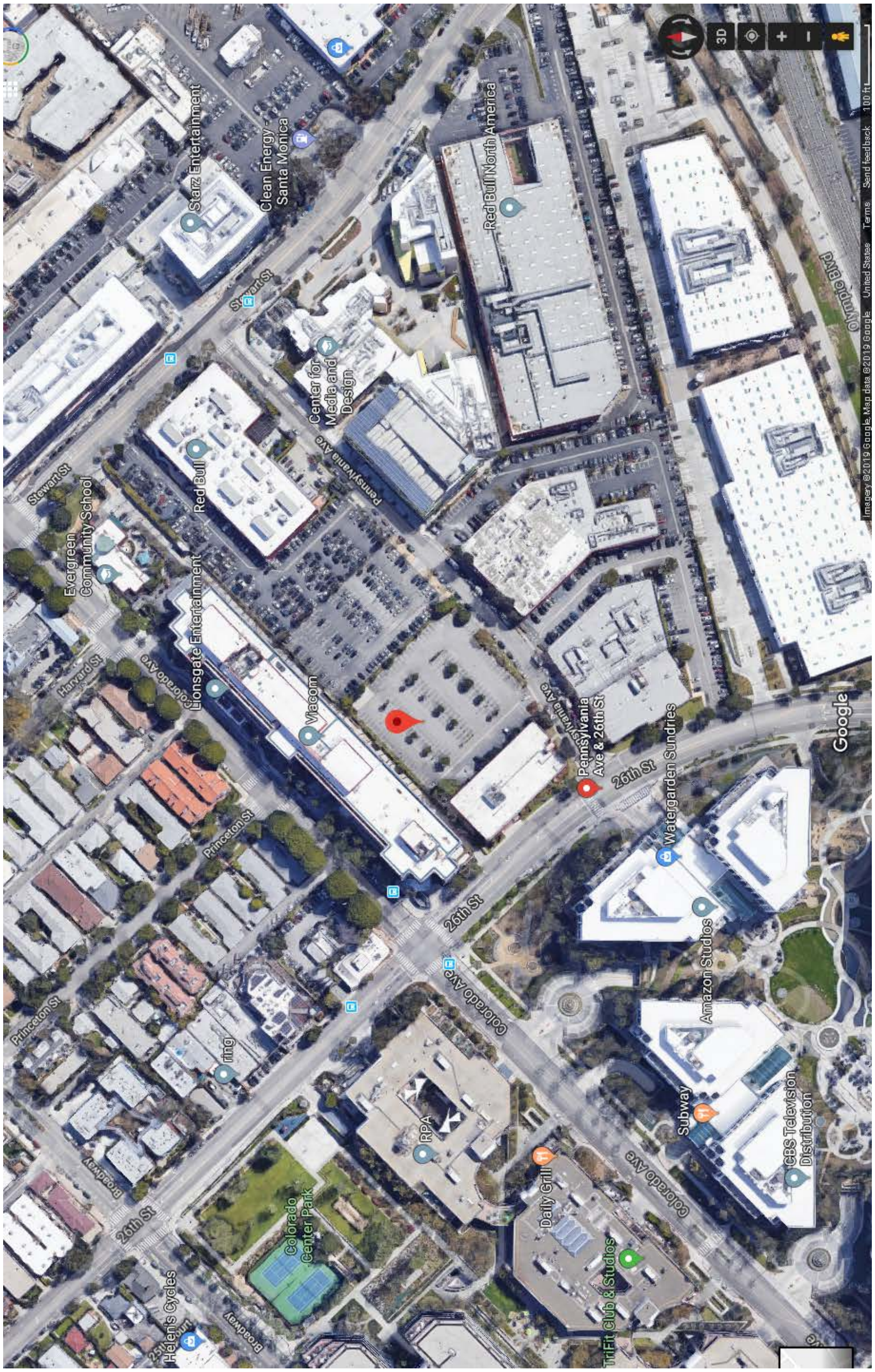


Figure 2 Satellite View, 1633 26th St, Santa Monica, CA 90404

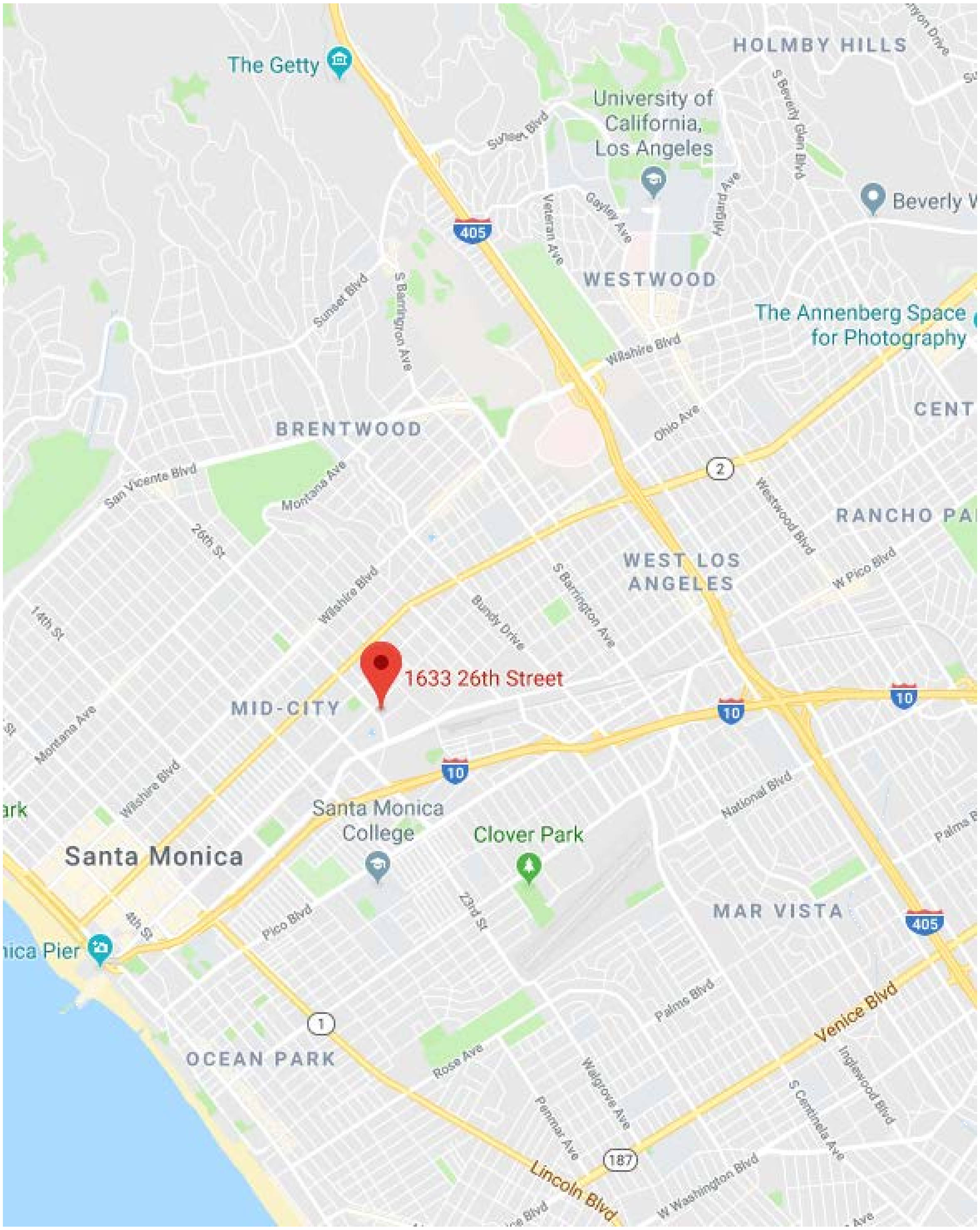


Figure 3 Vicinity Map, 1633 26th St, Santa Monica, CA 90404

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

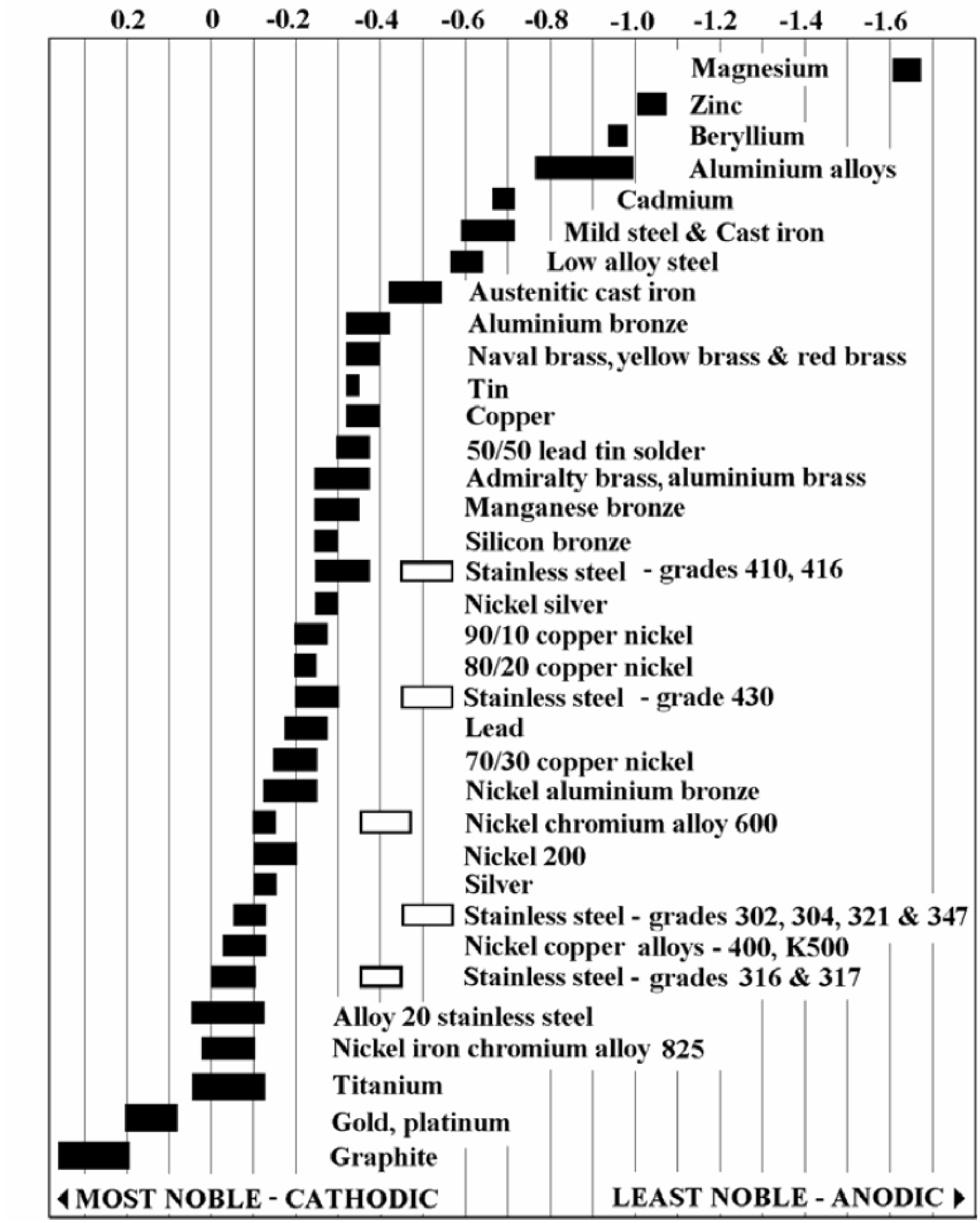
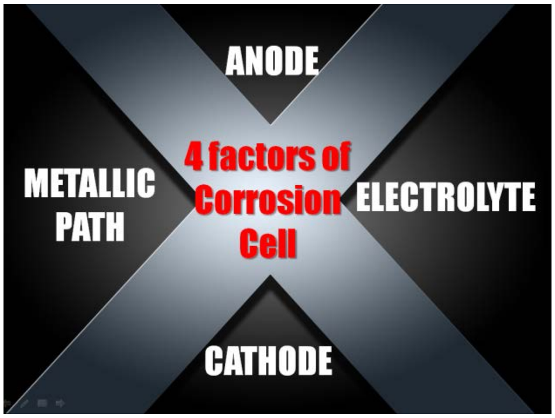
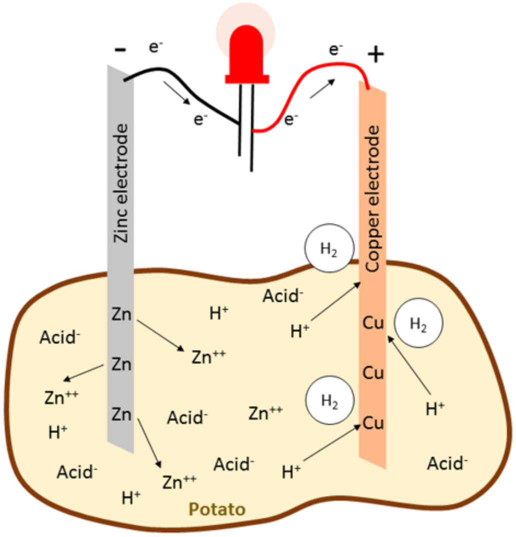


Figure 4 - Galvanic series of metals relative to CSE half cell.

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.

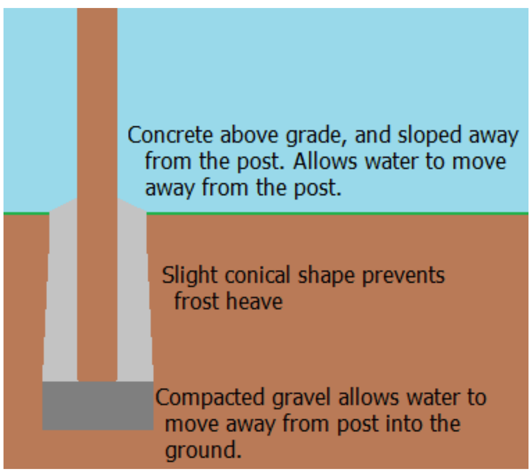
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction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

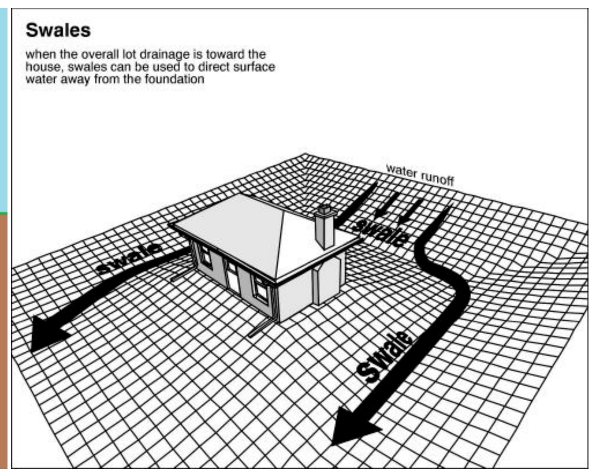
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and 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. [12,13]





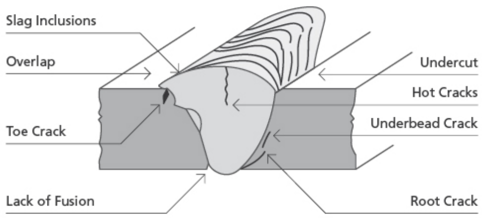
5.4.3 Avoiding Crevices

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**

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

[13] 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.



BAD **GOOD**

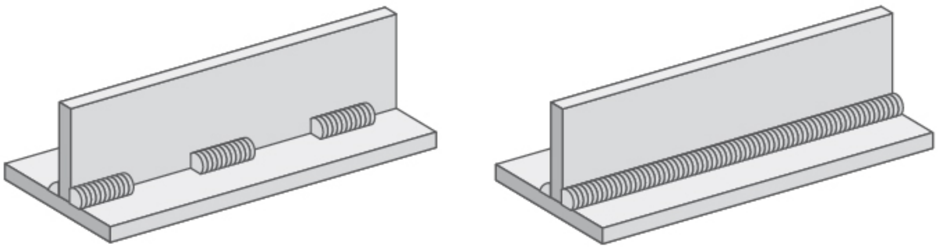


Figure 5 Defects which form weld crevices[14]

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

[14] 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.

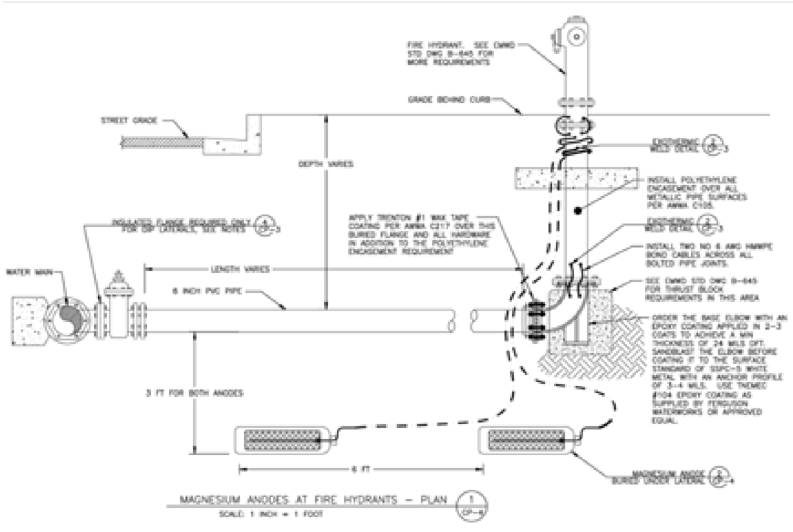


Figure 6 Sample anode design for fire hydrant underground piping

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.

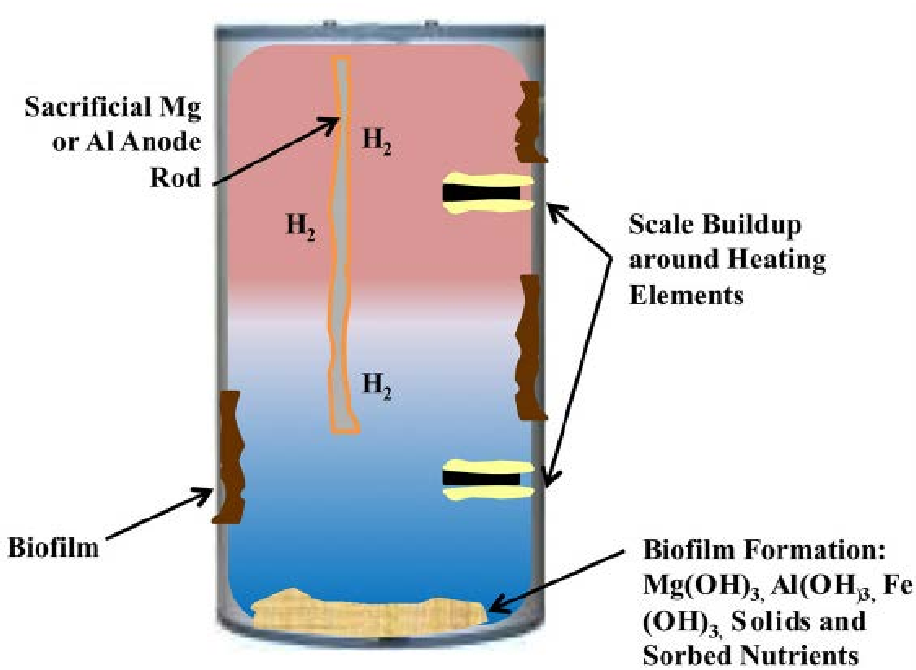


Figure 7 Cross section of boiler with anode

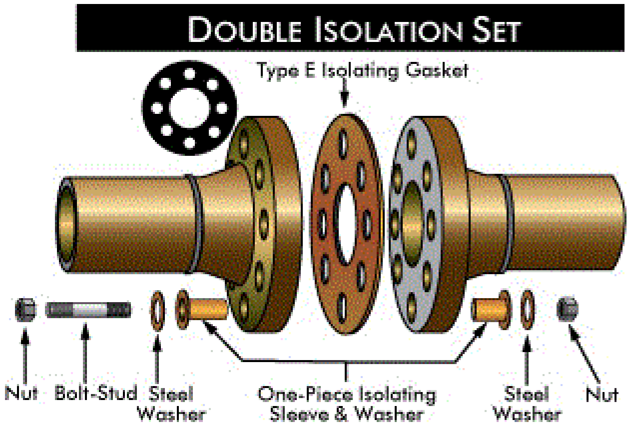
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

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.

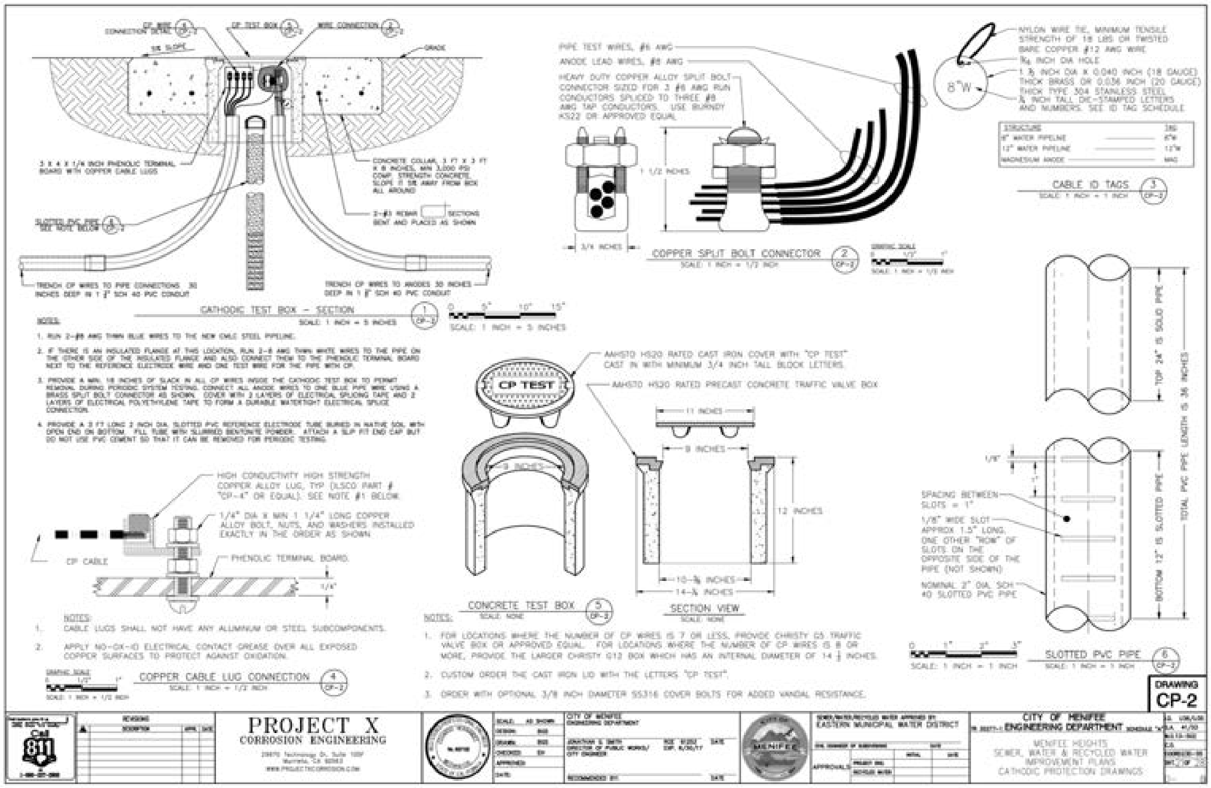


Figure 8 Sample of corrosion test station specification drawing

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.[15] 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.

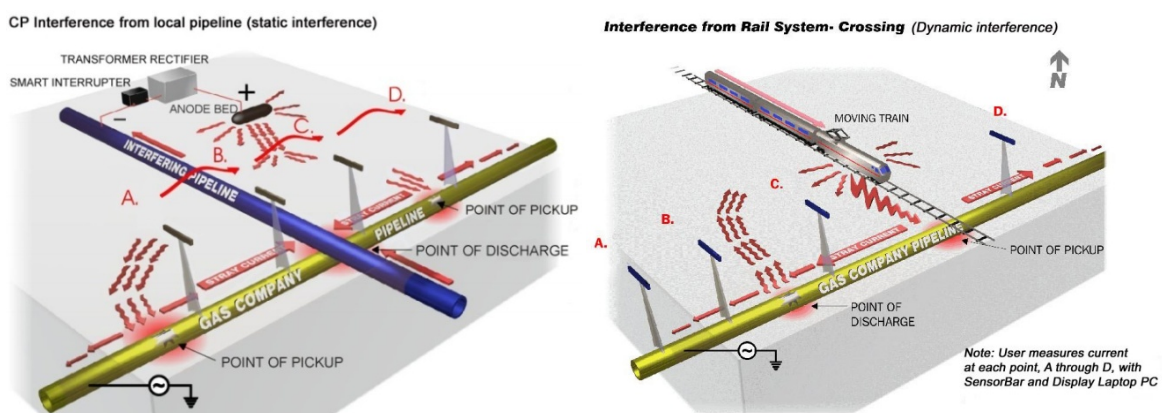


Figure 9 Examples of Stray Current[16]

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

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

Project X
Corrosion Engineering
Corrosion Control - Soil, Water, and Metallurgy Lab

S190702B Geotechno 21800 Kilroy 6 Full + Rpt

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29970 Technology Dr, Suite 105F, Murrieta, CA 92563

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Project X Job #:	
Date:	7-1-2019

Company Name:	GEOTECHNOLOGIES, INC	Contact Name:	REINHARD KNUR
		Phone No.:	(818) 240-9600
Mailing Address:	439 WESTERN AVE GLENDALE	Contact Email:	rknur@geoteq.com
Accounting Contact:	DEANNE NOONAN	Invoice Email:	accounting@geoteq.com
Project Name:	KILROY REALTY CORPORATION		
Client Project No:	21800	P.O. #:	KRT.

Turn Around Time:	5 Day Normal	3 Day RUSH 75% mark-up	2 Day RUSH 100% mark-up
	X		

Results By: ☐ Phone ☐ Fax ☐ Email ☐ Mail ☐ Overnight Mail (charges apply)

Received by:

Default Method

SPECIAL INSTRUCTIONS:
Address. 1633 26th ST. Santa Monica
Plan. Attached
HMOW @ 40' Elev. 715'

ANALYSIS REQUESTED (Please circle)

#	SAMPLE ID - BORE #	DESCRIPTION	DEPTH (ft)	DATE COLLECTED	CORROSION SERIES (Min. Resistivity, Sulfate, Chloride, Sulfide, Redox, pH, Ammonia, Nitrate)	Soil Resistivity (ASTM G187 / AASHTO T288 / Caltrans CTM643)	pH (ASTM G51 / AASHTO T289 / Caltrans CTM643)	Sulfate (ASTM D516 / AASHTO T290 / Caltrans CTM417)	Chloride (ASTM D512B / AASHTO T291 / Caltrans CTM422)	Redox Potential (SM 2580B)	BiCarbonate (SM 2320B)	Alkalinity (SM 2320B)	Acidity (SM 2310B)	Nitrate (Hach 835 / SM 4500-NO3)	Ammonia (Hach 890 / SM 4500-NH3)	Sulfide (SM 4500-S2 / SM 4500-S2)	Moisture Content (ASTM D2216)	Soil Corrosivity Evaluation Report	Metallurgical Analysis	NOTES
1	B1	Fill Soil	1-5		✓													✓		
2	B2	Fill Soil	1'-5		✓													✓		
3	B3	Fill Soil	1'-5		✓													✓		
4																				
5																				
6																				
7																				
8																				
9																				
10																				
11																				
12																				
13																				
14																				

Project X
Corrosion Engineering
Corrosion Control - Soil, Water, and Metallurgy Lab

S190702B Geotech no 21800 kilroy 6Full+Rpt

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29970 Technology Dr, Suite 105F, Murrieta, CA 92563

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Project X Job #:	
Date:	

Company Name:	Geotechnologies, Inc.	Contact Name:	Reinard Knur
Phone No.:	(818) 240-9600		
Mailing Address:	439 Western Ave.	Contact Email:	rknur@geoteq.com
Accounting Contact:	Deanne Noonan	Invoice Email:	accounting@geoteq.com
Project Name:	Kilroy Realty		
Client Project No:	21800	P.O. #:	

	5 Day Normal	3 Day RUSH 75% mark-up	2 Day RUSH 100% mark-up
Turn Around Time:	✓		

Results By: ☐ Phone ☐ Fax ☑ Email ☐ Mail ☐ Overnight Mail (charges apply)

Received by: 07/03/19 [signature]

Default Method

SPECIAL INSTRUCTIONS:

ANALYSIS REQUESTED (Please circle)

Analysis methods: CORROSION SERIES (Min. Resistivity, Sulfate, Chloride, Sulfide, Redox, pH, Ammonia, Nitrate); Soil Resistivity (ASTM G187, AASHTO T288B, Caltrans CTM643); pH (ASTM G 51, AASHTO T 289, Caltrans CTM643); Sulfate (ASTM D516, AASHTO T 290, Caltrans CTM417); Chloride (ASTM D512B, AASHTO T 291, Caltrans CTM422); Redox Potential (SM 2580B); BiCarbonate (SM 2320B); Alkalinity (SM 2520B); Acidity (SM 2510B); Nitrate (Hach 835, SM 4500-NO3); Ammonia (Hach 830, SM 4500-NH3); Sulfide (SM 4500-S2, SM 4500-S2); Moisture Content (ASTM D2216); Soil Corrosivity Evaluation Report; Metallurgical Analysis

	SAMPLE ID - BORE #	DESCRIPTION	DEPTH (ft)	DATE COLLECTED	CORROSION SERIES	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	BiCarbonate	Alkalinity	Acidity	Nitrate	Ammonia	Sulfide	Moisture Content	Soil Corrosivity Evaluation Report	Metallurgical Analysis	NOTES
1	B1		25		✓													✓		
2	B2		42.5		✓													✓		
3	B6		35		✓													✓		
4																				
5																				
6																				
7																				
8																				
9																				
10																				
11																				
12																				
13																				
14																				

JOB A-69161 DATE 9-5-69 DR. Todd CHKD.

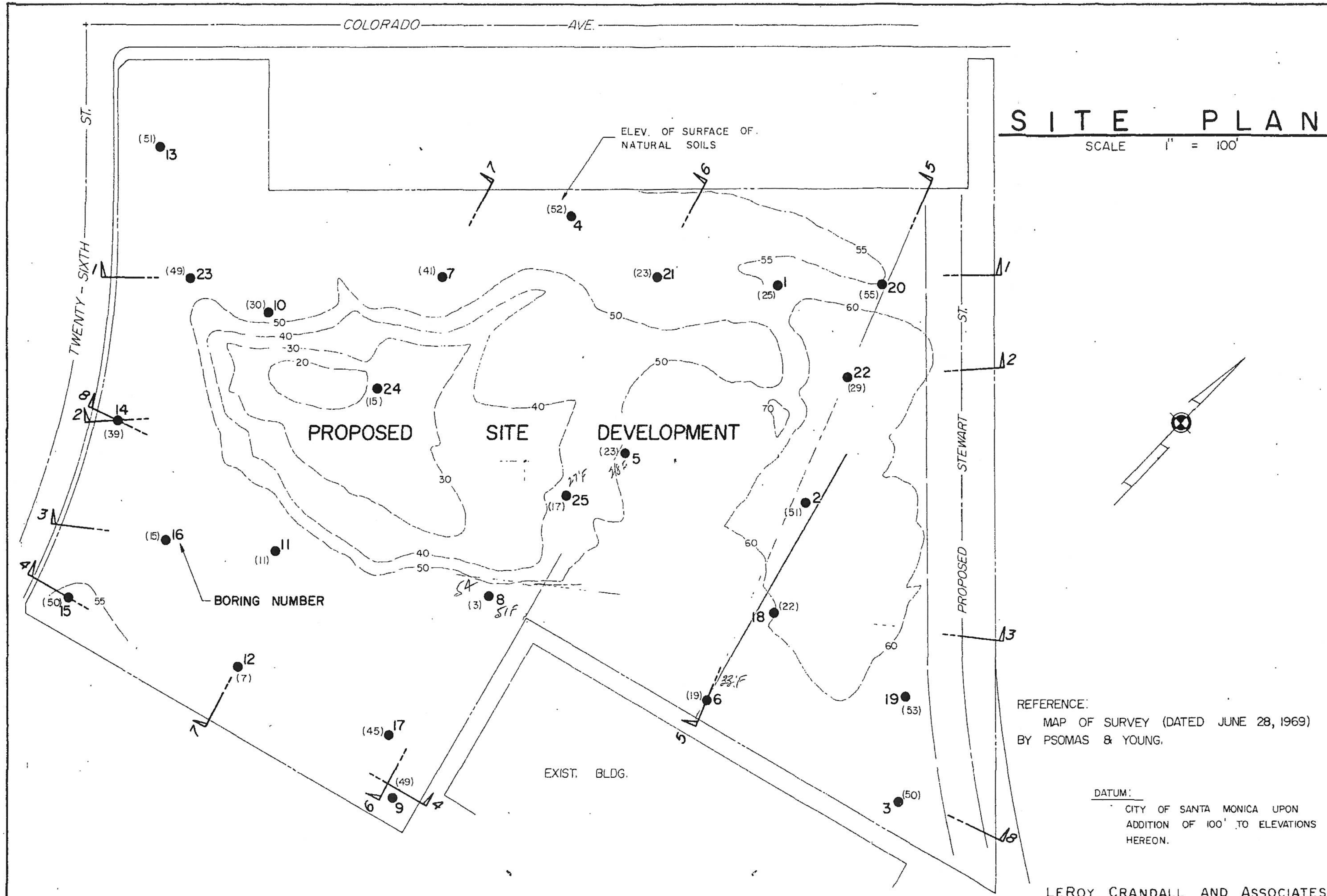
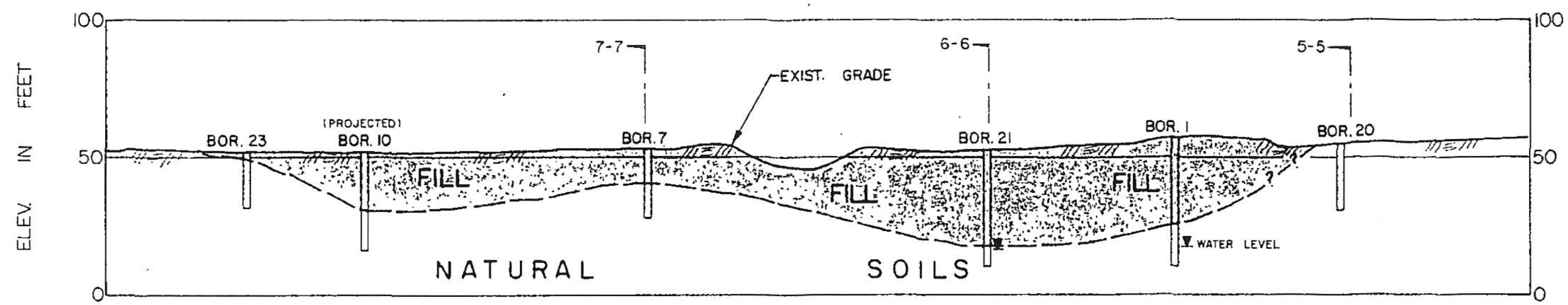
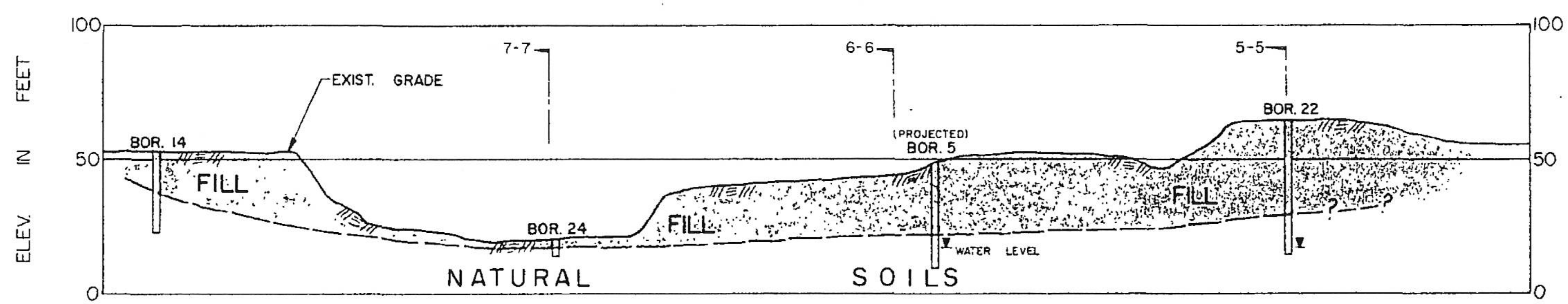


PLATE I

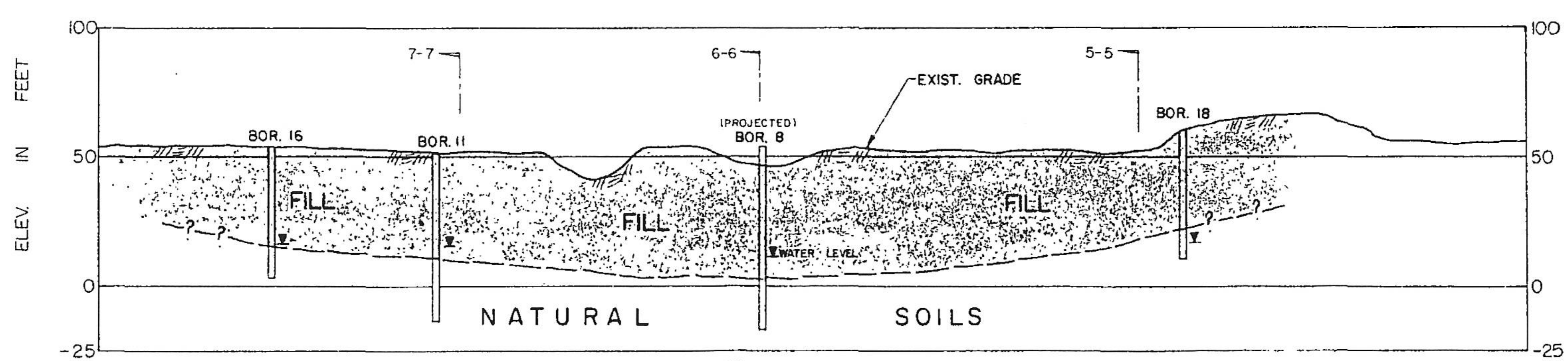
JOB A-69161 DATE 10-1-69 DR Todd O.E. SC CHKD



SECTION 1-1



SECTION 2-2



SECTION 3-3

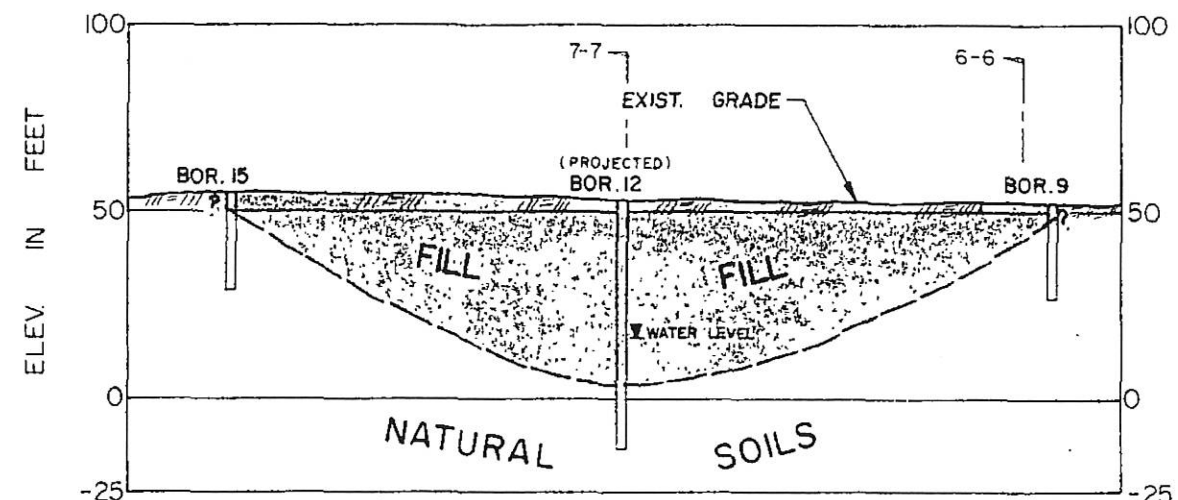
SUBSURFACE SECTIONS

SCALES : HORIZ. 1" = 100'
VERT. 1" = 50'

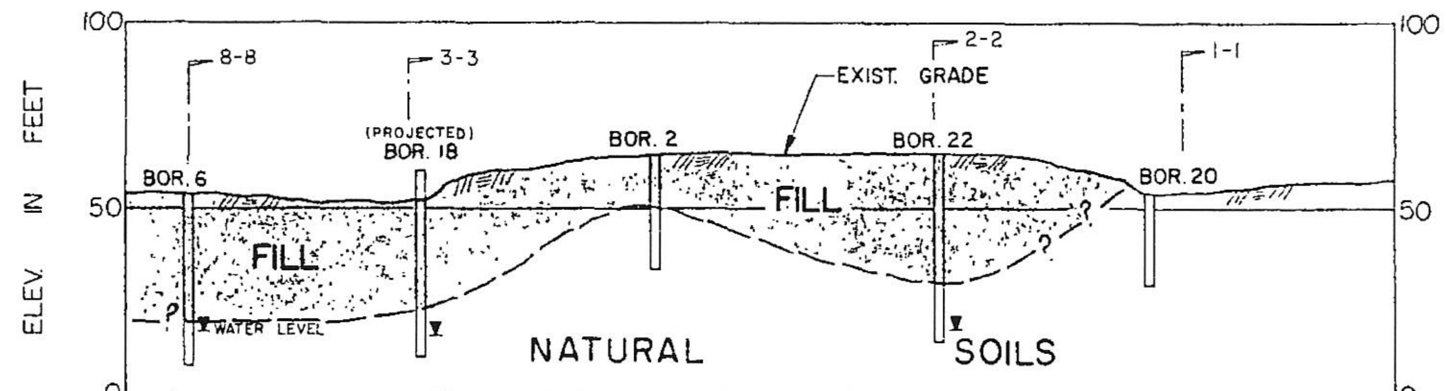
NOTE:
THE SECTIONS ARE BASED ON THE SOIL CONDITIONS AT THE BORING LOCATIONS. THE SOIL CONDITIONS BETWEEN BORINGS HAVE BEEN INTERPOLATED AND ARE NOT NECESSARILY ACCURATE.

LEROY CRANDALL AND ASSOCIATES

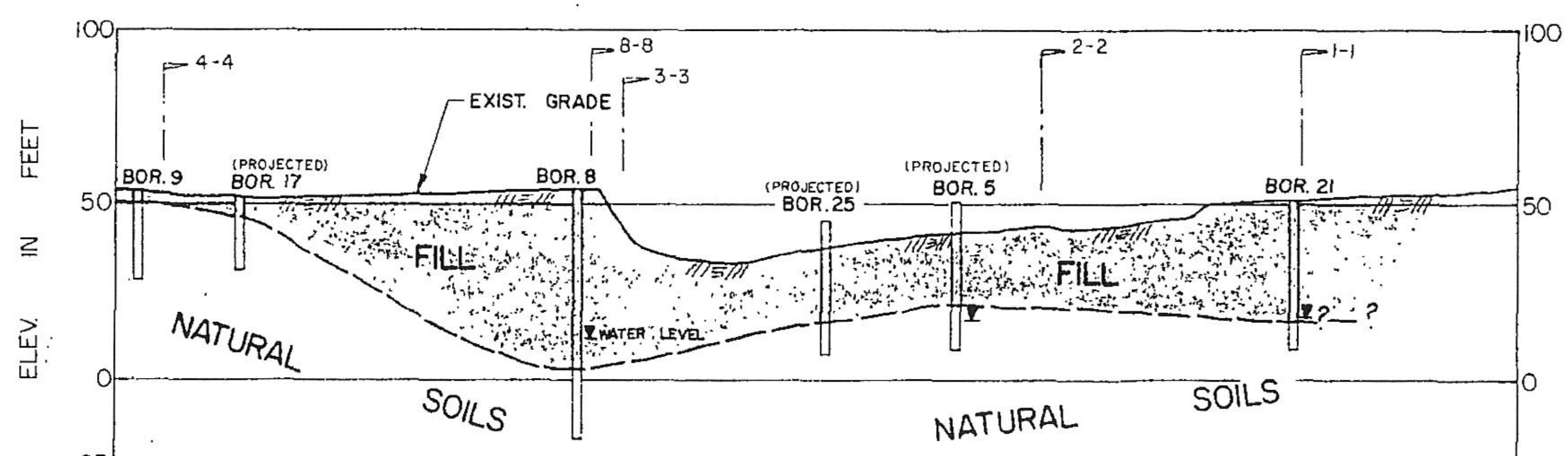
PLATE 2-A



SECTION 4-4



SECTION 5-5



SECTION 6-6

SUBSURFACE SECTIONS

SCALES : HORIZ. 1" = 100'
VERT. 1" = 50'

NOTE:
THE SECTIONS ARE BASED ON THE SOIL CONDITIONS AT THE BORING LOCATIONS. THE SOIL CONDITIONS BETWEEN BORINGS HAVE BEEN INTERPOLATED AND ARE NOT NECESSARILY ACCURATE.

JOB A-67161 DATE 10-1-67 DR. Todd O.E. SC CHKD. ccc

LEROY CRANDALL AND ASSOCIATES

PLATE 2-B

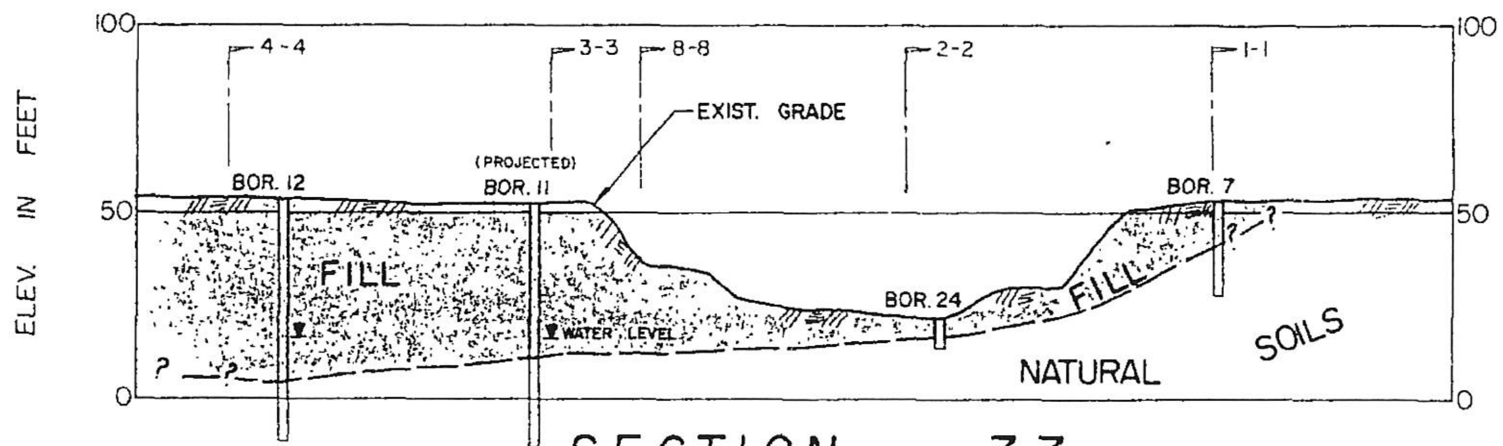
JOB A-69161 DATE 10-1-69 DR. Todd O.E. CHKD.

NOTE:

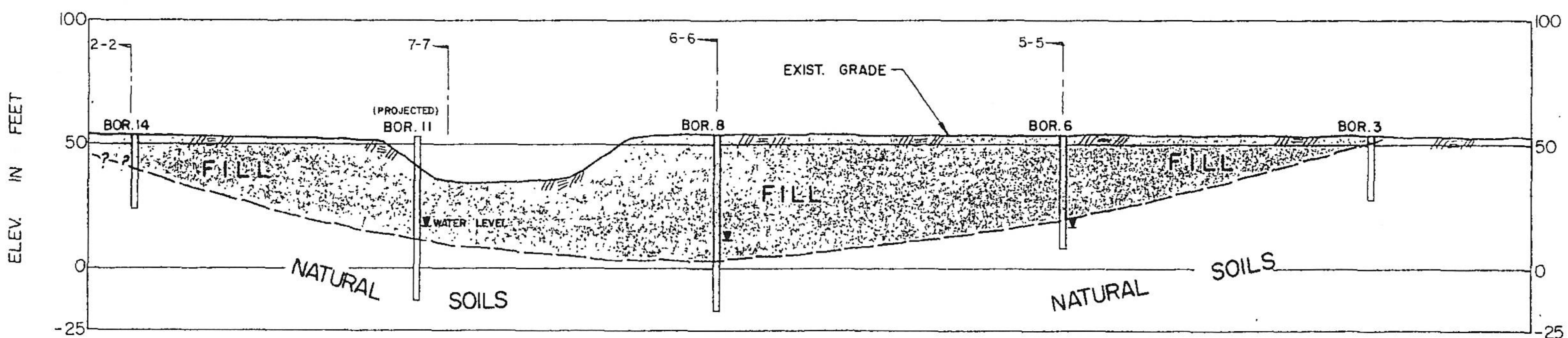
THE SECTIONS ARE BASED ON THE SOIL CONDITIONS AT THE BORING LOCATIONS. THE SOIL CONDITIONS BETWEEN BORINGS HAVE BEEN INTERPOLATED AND ARE NOT NECESSARILY ACCURATE.

SUBSURFACE SECTIONS

SCALES : HORIZ. 1" = 100'
VERT. 1" = 50'



SECTION 7-7



SECTION 8-8

LEROY CRANDALL AND ASSOCIATES

PLATE 2-C

JOB A69161 DATE 8-28-69 DR ALAN O.E. RM ka CHKD. [illegible]

BORING 7

DATE DRILLED : July 28, 1969

EQUIPMENT USED : 18"-Diameter Bucket

ELEVATION 53

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
					ML	FILL - CLAYEY SILT - 25% to 30% gravel, mottled brown
50		8.0	110			
					SM	FILL - SILTY SAND - well graded, mottled brown
	5	13.5	121			
45		10.1	112		ML	FILL - SILT - mottled brown
	10	10.5	105			
40					ML	CLAYEY SILT - greyish-brown
		13.9	97			
	15	12.4	100			
					ML	SANDY SILT - greyish-brown
35		17.3	98			
		17.3	109			
	20					
30		17.1	110			
	25					

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A69161 DATE 8-28-69 DR. ALAN O.E. RAH CHKD. ko hRC

BORING 10

DATE DRILLED July 28, 1969

EQUIPMENT USED 18"-Diameter Bucket

ELEVATION 52

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE
		8.1	104	■
50				
	5	5.7	116	■
45		11.6	95	■
	10	7.8	106	■
40				
		5.4	109	■
	15			
35				
	20	11.9	112	■
30		14.7	120	■
	25	6.0	124	■
25				
	30	12.0	119	■
20				
	35	17.2	114	■

Soil	Description
ML	FILL - SANDY SILT 5% to 10% debris, mottled brown Pieces of asphalt to 6" in size
CL	FILL - SILTY CLAY - mottled brown Pieces of concrete and wood
ML	CLAYEY SILT (PROBABLE FILL) - 15% well graded gravel, mottled brown
ML	CLAYEY SILT - greyish-brown
ML	SANDY SILT - 20% to 30% well graded gravel, greyish-brown
SM	SILTY SAND - fine, 20% well graded gravel, greyish-brown
ML	CLAYEY SILT - brown

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A69161 DATE 8-28-69 DR ALAN O.E. RM ka CHKD. [illegible]

BORING 23

DATE DRILLED: July 31, 1969
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 52

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
						1" ASPHALTIC PAVING
					ML	FILL - SILTY SAND - well graded, light brown
50		14.8	118	■		FILL - CLAYEY SILT - mottled brown
					ML	SILT - greyish-brown
	5	17.4	104	■		
45		18.0	104	■		
	10	15.6	110	■		
40						
	15	17.6	106	■		
35						
	20	19.4	104	■		

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-69161 DATE 10-1-69 DR T.F. OE Km CHKD. [illegible]

BORING 24

DATE DRILLED: September 24, 1969
EQUIPMENT USED: 6"-Diameter Hand Auger

ELEVATION 20

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft)	SAMPLE		
					CL	FILL - SILTY CLAY - greyish-brown
		17.4	100	■	ML	FILL - SANDY SILT - light greyish-brown
		27.4	89	■		
15	5				ML	SANDY SILT - some clay, few gravel, light greyish-brown
		13.2	119	■		6" cobble
		19.5	105	■		More gravel: Lens of SAND
10	10					
5	15					

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

PLATE A-24