Appendix I

Geotechnical

Appendix I.1: Geotechnical Report

GEOTECHNICAL ENGINEERING EXPLORATION

PROPOSED MIXED-USE RESIDENTIAL/COMMERCIAL BUILDING

LOTS 7 & 8, BLOCK 3, TRACT 7555

6435 WILSHIRE BOULEVARD

LOS ANGELES, CALIFORNIA

FOR BLACK EQUITIES

IRVINE GEOTECHNICAL, INC. PROJECT NUMBER IC 21035-I

JUNE 7, 2021

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INTRODUCTION

This report has been prepared per our agreement and summarizes findings of Irvine Geotechnical's geotechnical engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution, and engineering properties, relative stability and geologic structure of the earth materials underlying the site with respect to the design and construction of the proposed project.

INTENT

It is the intent of this report to assist in the design and completion of the proposed project. The recommendations are intended to reduce geotechnical risks affecting the project. The professional opinions and advice presented in this report are based upon commonly accepted standards and are subject to the general conditions described in the NOTICE section of this report.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with the client. The preliminary plans prepared by Studio Eleven Architects were considered prior to beginning work on this project. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the enclosed Site Plan and cross sections.

Exploration was conducted on March 17 and 18, 2021 with the aid of limited-access hollow-stem auger drill rig. It included drilling two borings to a maximum depth of 100 feet. Samples of the earth materials were obtained and delivered to the soils engineering laboratory of Soil Labworks, LLC for testing and analysis. The borings were logged by the staff engineer.

Office tasks included laboratory testing of selected soil samples, researching records on file at the City of Los Angeles, reviewing historical topographic maps and aerial photographs, preparing the Site Plan and cross sections and performing engineering analysis. Earth materials exposed in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project and the location of the borings are shown on the Site Plan. Subsurface distribution of the earth materials, projected geologic structure, and the proposed project are shown on Sections A through C.

RESEARCH - PREVIOUS WORK

The building and grading records of the City of Los Angeles Department of Building and Safety were researched prior to preparing this report. The records contain a geotechnical report for the design of the existing building on the site. The results of subsurface exploration, laboratory testing and engineering analysis are contained in a report by L.T. Evans, *"Foundation Investigation for Callahan Construction Company, [6435 Wilshire Boulevard, Lots 7 – 8, Block 3, Tract 7555], Los Angeles, California,"* dated June 22, 1950. Exploration included 3 18-inch diameter bucket auger borings drilled to a depth of 30 feet.

Recent geotechnical reports were located for a 15-story building under construction with an address range of 6401 through 6419 Wilshire Boulevard. The following reports were located and reviewed as part of our work on this project.

Reports by GeoDesign, Inc.:

> *Report of Geotechnical Engineering Services, Proposed Mixed-Use Development, 6401 to 6419 Wilshire Boulevard, Los Angeles, California,* dated April 10, 2015;

Addendum 3, Earthwork Adjacent to La Jolla Avenue and Wilshire Boulevard, Proposed Mixed-Use Development, 6401 to 6419 Wilshire Boulevard, Los Angeles, California, dated January 22, 2016;

Addendum 5, Updated Foundation Loading Recommendations, Proposed Mixed-Use Development, 6401 to 6419 Wilshire Boulevard, Los Angeles, California, dated December 22, 2016; and

Report of Geotechnical Construction Observation and Testing Services, Proposed Mixed-Use Development, 6401 to 6419 Wilshire Boulevard, Los Angeles, California, dated April 20, 2020

Apparently, Addenda 1, 2, and 4 were not submitted to the Grading Division for review.

Subsurface exploration included drilling 5 borings to a maximum depth of 51½ feet. The native alluvial deposits were recommended to support the building. A mat foundation was recommended to support the building at the basement level (about 12 to 14 feet below grade). The recommended allowable bearing pressure on the base of the mat was a uniform pressure of 2,800 psf. Locally, live loads were to be as higher as 5,300 psf. Settlement was estimated to be 2.25 inches, which was to occur during construction of the building. The seismic soil type used by GeoDesign is based on site-specific shear wave velocity testing contained in an earlier report by Professional Service Industries (PSI).

The Grading Division of the City of Los Angeles approved the GeoDesign reports in their *Soils Report Approval Letters* dated May 12, 2015 (Log 88004), January 22, 2016 (Log # 91605), and February 2, 2017 (Log #96341).

PSI explored the site in 2008 for a similar 16-story building. The result of subsurface exploration, laboratory testing, shear wave velocity testing, and engineering analysis are contained in their report, *"Geotechnical Exploration Report, Proposed La Jolla Project, Proposed Multi-Level Development, NWC of Wilshire Boulevard and La Jolla Avenue, Los Angeles, California,"* dated May 6, 2008. Four borings were drilled to a maximum depth of 51½ feet. The average shear wave velocity of the upper 100 feet of soils (Vs_{100}) was measured to be 923 ft/sec.

The Grading Division of the City of Los Angeles approved the PSI report in their *Soils Report Approval Letter* dated June 24, 2008 (Log 63531).

PROPOSED PROJECT

Information concerning the proposed project was provided by the client. The preliminary plans prepared by Studio Eleven Architects were a guide for preparing this report. It is proposed to demolish the existing 5-story office building and associated parking structure and redevelop the site with a mixed-use residential/commercial building. The structure will consist of 8 stories above grade on top of 2 levels of subterranean parking. Retaining walls up to 30 feet high are planned to support excavations for the basement levels. Foundation loads are unknown but are anticipated to be moderate for perimeter walls and high for columns and shear walls.

Formal plans have not been prepared and await the conclusions and recommendations of this report.

SITE DESCRIPTION

The subject property consists of a developed commercial parcel, in the Beverly Grove section of the City of Los Angeles, California. It is located on the north side of Wilshire Boulevard, between La Jolla Avenue and Sweetzer Avenue, just east of the intersection of San Vicente Boulevard and Wilshire Boulevard, and west of the La Brea Tar Pits. The site is developed with a 5-story office building that likely has one level of basement. A detached parking structure is located on the northern portion of the parcel. The parking structure has an upper level near original ground surface over one subterranean level. The surrounding area is developed with multi-level office, commercial, and residential buildings.

The site has a slight slope from north to south. Physical relief across the property is about 3 feet. Surface drainage generally is by sheet flow runoff down the contours of the land toward the south.

GROUNDWATER

Groundwater was encountered in Boring 1 at a depth of 39.5 feet (elevation 105 feet) and Boring 2 at a depth of 43 feet (elevation 103 feet). Historically highest groundwater in this area of Los Angeles is estimated to be 18 feet below the ground surface (Plate 1.2, *Historically Highest Groundwater Contours and Borehole Log Data Locations, Hollywood 7.5 Minute Quadrangle* in Seismic Hazard Zone Report for the Hollywood Quadrangle, SHZR-026).

EARTH MATERIALS

Fill

Fill, associated with previous site grading, underlies portions of the site to a maximum observed thickness of 24 inches. The fill consists of sandy clay that is grey-brown, moist, and firm to stiff.

Alluvium

Natural alluvial deposits were encountered to the total depths of the borings. The alluvium consists of weakly stratified layers of clay, clayey sand, silty sand and sand. The alluvium is yellow-brown, orange-brown, and grey-brown, moist to saturated, medium dense to dense (sand) and firm to stiff (clay).

GENERAL SEISMIC CONSIDERATIONS

Southern California is located in an active seismic region and numerous known and undiscovered earthquake faults are present in the region. Hazards associated with fault rupture and earthquakes include direct affects such as strong ground shaking and ground rupture, as well as secondary effects such as liquefaction, landsliding, and lurching. The United States Geological Survey (USGS), California Geologic Survey (CGS), Southern California Earthquake Center (SCEC), private consultants and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and early warning of strong ground shaking. Research and practice have shown that earthquake prediction is not practical or sufficiently accurate to benefit the public. Also, several recent and damaging earthquakes have occurred on faults that were unknown prior to rupture. Current standards and the California Building Code call for earthquake resistant design of structures as opposed to prediction.

Alquist-Priolo Fault Rupture Hazard Study Zone

California faults are classified as active, potentially active or inactive. Faults from past geologic periods of mountain building, but do not display any evidence of recent offset are considered ”inactive” or “potentially active.” Faults that have historically produced earthquakes or show evidence of movement within the Holocene (past 11,000 years) are considered “active faults.” Active faults that are capable of causing large earthquakes may also cause ground rupture. The Alquist-Priolo Act of 1972 was enacted to protect structures from hazards associated with fault ground rupture. No known active faults cross the subject property and the site is not located within an Alquist-Priolo Fault Rupture Hazard Study Zone. The ground rupture hazard at the site is considered low to nil.

Building Code Seismic Coefficients

Seismic design parameters within the Building Code include amplification of the seismic forces on the structure depending on the soil type, distance to seismic source and intensity of shaking. The purpose of the code seismic design parameters is to prevent collapse of structures and loss of life during strong ground shaking. Cosmetic damage should be expected.

The N_{60} blow counts for the upper 100 feet of soils was averaged following the procedures requirements of the Building Code. As shown on the AVERAGE N60 FOR BORING 1 chart and calculations appended to this report, average N_{60} blow count is 26.5 blows/ft. In accordance with Chapter 20 of ASCE 7-16, Table 20.3-1, the Site Class is D – Stiff Soil. This conclusion is similar to Site Class D determined for the referenced nearby project using shear wave velocity measurements.

The following table lists the applicable seismic coefficients for the 2020 Los Angeles Building Code.

SEISMIC COEFFICIENTS (2020 Los Angeles Building Code)		
Latitude = 34.0641°N Longitude = 118.3696°W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class Chapter 20 - ASCE 7	D-Stiff Soil	
Seismic Design Category from Table 1613.2.5(1) and 1613.2.5(2)	D	
Spectral Accelerations from Figures 1613.2.1 (1) through 1613.2.1(8)	S_s = 2.067g	S_1 = 0.737g
Site Coefficients from Tables 1613.2.3 (1) and 1613.2.3 (2)	F_A = 1.0	F_V = 1.7
Spectral Response Accelerations from Equations 16-36 and 16-37	S_{MS} = 2.067g	S_{M1} = 1.253g
Design Accelerations	S_{DS} = 1.378g	S_{D1} = 0.835g

Seismic Hazards

The principal seismic hazard to the subject property and proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking using shear panels, moment-resisting frames and reinforcement. Additional precautions may be taken to protect personal property and reduce the chance of injury, including strapping water heaters and securing furniture and appliances. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Seismic Hazard Zones

The California State Legislature enacted the Seismic Hazards Mapping Act of 1990, which was prompted by damaging earthquakes in California, and was intended to protect public safety from the effects of strong ground shaking, liquefaction, landslides, and other earthquake related hazards. The Seismic Hazards Mapping Act requires that the State Geologist delineate various ”seismic hazards zones.” The maps depicting the zones are released by the California Geological Survey.

The Seismic Hazards Mapping Act requires a site investigation by a certified engineering geologist and/or civil engineer with expertise in geotechnical engineering, for projects sited within a hazard zone. The investigation is to include recommendations for a ”minimum level of mitigation” that should reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy. The Seismic Hazards Mapping Act does not require mitigation to a level of no ground failure and/or no structural damage.

Seismic Hazard Zone delineations are based on correlation of a combination of factors, including: surface distribution of soil deposits; physical relief; depth to historic high groundwater; shear strength of the soils; and occurrence of past seismic deformation. The subject property is located within the United States Geologic Survey, Hollywood Quadrangle. Seismic hazards within the Hollywood Quadrangle were evaluated by the CGS in their report, "*Seismic Hazard Zone Report for the Beverly Hills 7.5-minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 026.*" According to the Seismic Hazard Zones Map, the subject property is within an area that has been subject to or may be subject to liquefaction.

Ground Motion

Spectral accelerations and peak ground accelerations at the site were determined for the Risk-Targeted Maximum Considered Earthquake (MCER) and Geometric Mean Peak Ground Acceleration (MCEG) following the procedures in ASCE 7-16 and the 2019 Building Code. The computed PGA for this site is 0.886. The adjusted PGA_M for Site Class D-Stiff Soil is 0.975g. According to the USGS deaggregation website (https://earthquake.usgs.gov/hazards/interactive/) and using a ground motion with a 10 percent probability of exceedance in 50 years, the modal de-aggregated earthquake PGA and moment magnitude are 0.523g and 6.36, respectively. For a ground motion with a 2 percent probability of exceedance in 50 years, the modal de-aggregated earthquake PGA and moment magnitude are 0.901g and 6.36, respectively. The modal distance to the ground motion source is 3.54 km. It is our understanding that the fundamental period of the proposed building will be less than 0.5 seconds.

Liquefaction

Liquefaction is a process that occurs when saturated sediments are subjected to repeated strain reversals during an earthquake. The strain reversals cause increased pore water pressure such that the internal pore pressure approaches the overburden pressure and the shear strength approaches zero. Liquefied soils may be subject to flow or excessive strain, which can cause settlement. Liquefaction occurs in soils below the groundwater table. Soils commonly subject to liquefaction include loose to medium dense sand and silty sand. Predominantly fine-grained soils, such as silts and clay, are less susceptible to liquefaction. Generally, plastic soils with a clay content of greater than 15 percent, a Plasticity Index greater than 18, and/or a fines content (percent passing the 200 sieve) greater than 30 to 50 percent, are not considered subject to liquefaction.

In conformance with current Grading Division=s policy, the liquefaction hazard was computed for ground motions representing recurrence intervals of 2,475 years, which is roughly represented by PGA_M. Design magnitude earthquake of 6.39 was used to magnitude weight the liquefaction resistance. It was assumed that the groundwater will be within 18 feet of the ground surface (historic high groundwater).

The stresses, strains, and safety factor for liquefaction were calculated using the methodologies by T.L. Youd, et. al., (*Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,* 1998), P.K. Robertson (*Cyclic Liquefaction and its Evaluation Based on the SPT and CPT,* 1997), P.K. Robertson, 2009, (*Guide to Cone Penetration Testing for Geotechnical Engineering*), "*Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California"* (Southern California Earthquake Center, 2002), California Geological Survey, Special Publication 117A, (*Guidelines for Evaluating and Mitigating Seismic Hazards in California*, 2008) and R. B. Seed, et. al., 2003, (*Recent Advances in Soil Liquefaction Engineering*: a Unified and Consistent Framework).

The last column of "Liquefaction Analysis Using SPT Data" lists the calculated safety factor of the soils encountered in Boring 1. The N_{60} tip resistance was converted to an equivalent SPT N_{60CS} blow count using published correlations and following the recommendations of SP117A. Plastic soils with a Plasticity Index (PI) greater than 18 were also screened from the analysis. Clay deposits between 49 and 71 feet are too plastic to liquefy. The calculations were performed for PGA_M ground motion.

LIQUEFACTION & DYNAMIC SETTLEMENT POTENTIALS	
Ground Motion (PGA_M)	
Layers (Feet) (FS<1.0)	Settlement (Inches) (FS < 1.0)
32.5	0.40
47.5	0.39
Total Settlement	1.33

Dynamic Settlement

Dissipation of excess pore pressure after liquefaction can result in settlement. The volumetric strain and accompanying settlement of saturated soils were estimated using procedures developed by Tokimatsu and Seed, 1987.

The enclosed Seismic Settlement calculations indicate 1.33 inches of dynamic settlement for the PGA_M ground motion. According to the referenced 2002 SCEC publication, differential settlement is typically of 1/2 to 2/3 of the total settlement for Holocene sediments. The liquefaction induced differential settlement potential of the site is expected to range between 0.67 to 0.89 inches.

The liquefaction potential of the site is moderate. A mat-type foundation is recommended to support the proposed residence to mitigate the liquefaction and dynamic settlement potentials. In conformance with ASCE 7-16, differential settlement of the concrete building should be limited to .003L. The structural engineer should verify that the foundations supporting the buildings do not lose their ability to carry gravity loads and that collapse of the building is prevented.

Lateral Spreading Hazard

Saturated soils that have experienced liquefaction may be subject to lateral spreading where located adjacent to free-faces, such as slopes, channels, and rivers. The site is remote to free-faces and the lateral spreading hazard at the site is nil.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon 2 borings, research of available records, consultation, years of experience observing similar properties in similar settings and review of the development plans. It is the finding of Irvine Geotechnical that construction of the proposed project is feasible from a geotechnical engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material is the alluvium, which will be exposed at the proposed basement levels. The existing fill is not recommended for foundation or slab support. A mat foundation is recommended to support the building in order to mitigate hazards associated with dynamic settlement and high groundwater.

Geotechnical Issues

Geotechnical issues affecting the site include deep excavations (20 to 35 feet high) near property boundaries. Shoring will be required for this project to support temporary excavations near property lines and the public right-of-way.

SITE PREPARATION

Surficial materials consisting of fill and disturbed soils are present on the site. Remedial grading is recommended to improve site conditions for support of any near surface slabs, paving and ramps.

General Grading Specifications

The following guidelines may be used in preparation of the grad¬ing plan and job specifications. Irvine Geotechnical would appreciate the opportunity of reviewing the plans to ensure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The site should be prepared to receive compacted fill by removing all vegetation, debris, existing fill, and disturbed soils. The exposed excavated area should be observed by the soils engineer prior to placing compacted fill. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum density.

B. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts and compacted in six-inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

C. The fill shall be compacted to at least 90 percent of the maximum laboratory density for the material used. Where cohesionless soil (less than 15 percent finer than 0.005 millimeters) is used for fill, it shall be compacted to a minimum of 95 percent relative compaction. The fill should be placed at a moisture content that is at or within 3 percent over optimum. The maximum density and optimum moisture content shall be determined by ASTM D 1557-12 or equivalent.

D. Field observation and testing shall be performed by the soils 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 90 percent compaction is obtained. One compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

Shrinking/Bulking

The following table contains the estimated bulking and shrinking factors to be used in determining earthwork volumes. The factors were determined by ratios of in-situ density or compacted density or loose density.

EARTHWORK FACTORS					
Earth Material	In-Situ Density (pcf)	Loose Density (pcf)	Compacted Density (pcf)	Bulking Factor (%)	Shrinking Factor (%)
Alluvium	108.4	100	117	8.4	7.4

Excavation Characteristics

The borings did not encounter hard, cemented bedrock or boulders. Groundwater should be anticipated for drilled excavations deeper than 37 to 39 feet. Some of the sand layers in the alluvium may have high flow rates, which may result in flowing sand, where exposed in large diameter shafts. Precautions to prevent caving within drilled foundations and shoring, such as casing or drilling muds, may be required. Lagging will be required for shoring.

Water should be pumped from drilled shafts prior to placing concrete. Alternatively, water within drilled shafts may be displaced by pouring the concrete from the bottom-up using a tremmie pipe. The compressive strength of the concrete should be increased by 1,000 psi over the design strength for concrete placed below the water table.

Dewatering is not anticipated to construct this project. Saturated ground at the base of the excavation may "pump" and lose strength. Smaller and lighter equipment may be required within of the excavations. Gravel blankets and/or geogrid mats may be required to stabilize the saturated excavation.

FOUNDATION DESIGN

General Conditions

The following foundation recommendations are minimum requirements. The structural engineer may require footings that are deeper, wider, or larger in diameter, depending on the final loads.

Mat Foundation

A mat-type foundation system is recommended to support the building and is intended to distribute the structural weight of the building uniformly to the soil and to withstand differential settlement. According to the structural engineer, the average pressure along the base of the mat is 1,800 psf. The pressures will be locally higher in areas of concentrated loading. According to the structural engineer, concentrated loads will create localized pressure along the base of the mat of 4,000 psf to 5,000 psf. The locally higher loads are acceptable from a geotechnical engineering standpoint, provided the average pressure does not exceed 1,800 psf.

Coefficient of sliding friction along the base of the mat may be assumed to be 0.25. For computing deflection of the foundation system, a modulus of subgrade reaction ks of 29 pounds/in^3 (50 kips/ft^3) may be assumed.

Spread Footings

Continuous and/or pad footings may be used to support non-habitable appurtenant structures, provided they are founded in alluvium. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24 inches square. The following chart contains the recommended allowable design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Alluvium	18	1,500	0.25	200	3,500

Increases in the bearing value are allowable at a rate of 300 pounds per square foot for each additional foot of footing width or depth to a maximum of 3,500 pounds per square foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing value shown above is 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.

The on-site soils non-expansive. Footings should be reinforced following the recommendations of the structural engineer. It is recommended that continuous footings be reinforced with a minimum of four #4 steel bars; two placed near the top and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geologist and geotechnical engineer prior to placing forms, steel or concrete.

Unless the retaining wall is designed to accommodate the surcharge, footings should not be supported by retaining wall backfill or derive support within the active wedge behind the retaining wall. Foundations adjacent to basements should be deepened below a 1:1 plane projected up from the base of the retaining wall. Alternatively, foundations adjacent to basements may be designed as a grade beam and structurally connected to the wall.

Foundation Settlement

The settlement potential of the bearing soils was calculated using a computer program Settle3, Ver. 5.010 developed by Rocscience. Based on the size of a 93-foot by 155-foot mat supporting a uniform pressure of 1,800 psf, the computed settlement is 2.90 inches. Most of the settlement is expected during construction as the mat is loaded. Differential settlement is estimated to be less than 1 inch across the width of the mat. Differential settlement is expected to be less than ½ inch over a distance of 40 feet.

RETAINING WALLS

General Design - Static Loading

Cantilevered retaining walls for this project are expected to be 12 feet high or less. Basement retaining walls could support excavations up to 30 feet.

Cantilevered retaining walls up to 12 feet high that support alluvium and approved retaining wall backfill, may be designed for an equivalent fluid pressure of 55 pounds per cubic foot. Restrained basement walls that are pinned at the top by a non-yielding floor should be designed for an at-rest earth pressure. The recommended design at-rest earth pressure on restrained basement walls is an equivalent fluid pressure of 70 pcf.

Basement walls that are constructed using shoring and bracing, may be designed for a trapezoidal distribution of pressure. The recommended design trapezoidal earth pressure on restrained retaining walls is 47H, where H is the retained height in feet.

In addition to the static design pressures, retaining walls below a depth of 18 feet (historic high ground water) should be designed to resist a hydrostatic pressure.

Retaining walls without a subdrain or retaining walls below a depth of 18 feet (historic high ground water) should be designed for a hydrostatic condition (62.4 pcf) plus the effective stress active or at-rest soil pressures. The design phi angle for the native soils along the property line is 26 degrees and the wet unit weight is 136 pcf.

The corresponding coefficient of active earth pressure (Ka) for cantilevered retaining walls is 0.39. The effective weight soil pressure on cantilevered retaining walls designed to resist a full hydrostatic condition is 28.7 pcf (0.39*(136 pcf - 62.4pcf). Thus, the total fluid pressure on cantilevered retaining walls designed for a hydrostatic condition is 91 pcf (28.7 pcf + 62.4 pcf).

The corresponding coefficient of at-rest earth pressure (K_0) for restrained basement retaining walls is 0.56. The effective weight soil pressure on cantilevered retaining walls designed to resist a full hydrostatic condition is 41.2 pcf (0.56 * (136 pcf - 62.4pcf). Thus, the total fluid pressure on restrained basement retaining walls designed for a hydrostatic condition is 103.6 pcf (41.2 pcf + 62.4 pcf).

Basement slabs should be designed to resist uplift forces for a hydrostatic condition.

Seismic Surcharge

In conformance with the Building Code, retaining walls higher than 6 feet were considered for seismic loading for the design ground motion resulting from the Maximum Considered Earthquake. The horizontal coefficient of seismic increment (K_E) and seismic increment (P_E) were estimated following procedures by Sitar, N. et. al.,2010, (Seismic Earth Pressures on Deep Building Basements, SEAOC 2010 Convention Proceedings). Spectral accelerations at the site were determined for the Maximum Considered Earthquake (MCE) following the procedures in ASCE 7-16 and the 2019 Building Code. The computed PGA_M for this site is 0.975g. The horizontal coefficient of seismic increment (K_E) was assumed to be a(PGA_M) = 0.325g.

The force required in addition to the static design force to raise the safety factor to at least 1.0 (P_E) was checked using a computerized version of the Mononobe-Okabe method. Ground motion was assumed to be 0.325g.

The recommended static and seismic forces for cantilevered and restrained retaining walls are shown in the following table. Where the unbalanced seismic force is higher than the static design pressure, the seismic increment was converted to an equivalent fluid pressure.

DESIGN EARTH PRESSURES - WALLS > 6 FEET			
Surface Slope Gradient	Static Design Force	Seismic Force*	Seismic Surcharge
Level	12ft^2 * 55 pcf /2= 3.960 kips	4.152 kips	3 pcf
Restrained	30ft^2 * 75 pcf /2= 33.750 kips	28.364 kips	0 pcf

* See calculation sheets. Combined static plus seismic pressure for FS = 1.0

Surcharge Loading

Retaining walls that are surcharged by traffic and/or structural loads should be designed to withstand the surcharge. The surcharge loads may be computed following the guidelines in City of Los Angeles P/BC 2020-83 (Retaining Wall Design) and P/BC 2017-141 (Guidelines for Determining Live Loads Surcharge from Sidewalk Pedestrian Traffic and Street Traffic) or equivalent Boussinesq methods. Irvine Geotechnical would be happy to assist the structural engineer in evaluating the surcharge pressure and the point of application from concentrated structural loads.

Subdrain

The recommended design earth pressures assume a free-draining backfill and no buildup of hydrostatic pressures. Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel. Not all subdrain systems and pipes are approved by all Building Departments. It is recommended that the Building Department be consulted when using non-conventional systems. The subdrain system should discharge to the atmosphere or to an engineered sump via gravity. Surface drains should not be connected to the subdrain system.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-12. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment and the retained height is less than 10 feet, retaining walls should be backfilled with 3/4-inch crushed gravel to within 2 feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches or the retained height is more than 10 feet, the gravel must be vibrated or wheel-rolled and tested for compaction. The upper 2 feet of backfill above the gravel should consist of a compacted fill blanket to the surface. Retaining wall backfill should be capped with a paved surface drain or a concrete slab.

TEMPORARY EXCAVATIONS

Temporary excavations will be required to construct the proposed project. The excavations could be up to 35 feet in height and will expose fill over alluvium. Where not surcharged by existing footings or structures, the soils are capable of maintaining vertical excavations up to 5 feet. Where vertical excavations exceed 5 feet, the upper portion should be trimmed to 1:1 (45 degrees).

It should be noted that regardless of stability, excavations that remove lateral support from property lines or existing structures are not allowed by the Code. The following section from Chapter 33 of the Building Code governs temporary excavations:

3307.3 Temporary excavations and shoring

3307.3.1 General. Excavations shall not remove the lateral support from a public way, from an adjacent property or from an existing structure. For the purpose of this section, the lateral support shall be considered to have been removed when any of the following conditions exist:

1. The excavation exposes any adverse geological formations, which would affect the lateral support of a public way or an adjacent structure.

2. The excavation extends below a plane extending downward at an angle of 45 degrees from the edge of the public way or an adjacent property.

Exception: Normal footing excavations not exceeding two feet in depth will not be construed as removing lateral support.

3. The excavation extends below a plane extending downward at an angle of 45 degrees from the bottom of an existing structure.

Vertical excavations removing lateral or vertical support from existing foundations or property lines will require the use of temporary shoring.

Shoring

Temporary shoring supporting excavations up to 20 feet, should be designed for an equivalent fluid pressure of 30 pounds per cubic foot per the enclosed calculations. Where excavations are higher than 20 feet and up to 27 feet, the design pressure on shoring is 35 pcf. Where excavations exceed 27 feet, shoring should be designed for 40 pcf.

Shoring may consist of cast-in-place concrete piles with wood lagging. Shoring piles should be a minimum of 12 inches in diameter and a minimum of 6 feet below the base of the excavation. Piles may be assumed fixed 3 below the base of the excavation. For the vertical forces, piles may be designed for a skin friction of 350 pounds per square foot for that portion of pile in contact with the alluvium. Soldier piles should be spaced a maximum of 10 feet on center.

The friction value is 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. Resistance to lateral loading may be provided by passive earth pressure within the alluvial soils below the base of the excavation.

Passive earth pressure may be computed as an equivalent fluid having a density of 300 pounds per cubic foot. The maximum allowable earth pressure is 3,500 pounds per square foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 3 pile diameters on center may be considered isolated.

Surcharge Loading

Shoring that is surcharged by traffic and/or structural loads should be designed to withstand the surcharge. The surcharge loads may be computed following the guidelines in City of Los Angeles P/BC 2017-141 (Guidelines for Determining Live Loads Surcharge from Sidewalk Pedestrian Traffic and Street Traffic) or equivalent Boussinesq and site-specific methods.

Irvine Geotechnical would be happy to assist the shoring engineer in evaluating the surcharge pressure and the point of application from concentrated structural loads.

Lagging

Lagging will be required between piles. Due to arching in the soils, pressure on the lagging will be less that on the shoring piles. It is recommended that the lagging be designed for the full pressure but be limited to a maximum of 400 pounds per square foot. The void between the lagging and the back-cut should be slurry-filled and observed by a representative of the geotechnical engineer.

Raker Braces & Dead Man Footings

A bearing value of 3,500 psf may be assumed for inclined raker footings. This allowable pressure assumes an inclined load from the sloping raker brace. A coefficient of sliding friction of 0.25 may be assumed along the base of the footing. Passive pressure may be assumed to be 250 pcf.

Earth Anchors

Earth Anchors (tie backs) may be employed to assist the shoring system. Pressure grouted anchors are recommended. The active wedge angle behind shoring is assumed to be a 30-degree plane (measured from the vertical). The bonded length of earth anchors should extend at least 15 feet beyond the active wedge. For preliminary design, a bond friction between the anchors and the soil beyond the active wedge of 400 psf and 2,000 psf may be assumed for gravity-cast and post-grouted anchors, respectively. According to the Post-Tension Institute, *Recommendations for Prestressed Rock and Soils Anchors, Table 6.3,* and allowable bond stress of 2,000 psf should be acceptable for post-grouted anchors.

Deflection Monitoring

Prior to construction and excavation for the project, it is recommended that the existing conditions along the property lines be documented and surveyed. Documentation should include photographs and descriptions of the offsite structures and conditions. Survey monuments should be affixed to representative structures and to points along the property line and offsite. The survey points should be measured prior to construction to form a baseline for determining settlement and/or deformation. Upon installation of the shoring system, survey monuments should be affixed to the tops of representative piles so that deflection can be measured.

Some deflection is expected for a well designed and constructed cantilevered shoring system. Where offsite structures are located more than 35 feet of the shored excavation, it is recommended that deflection be limited to 1 inch. Where offsite structures are located within 35 feet of the shored excavation, it is recommended that the deflection be limited to ½ inch or less.

The shored excavation and offsite structures should be visually inspected everyday. Survey monuments should be measured once a month during the construction process. Should the surveys reveal offsite deformation or excessive deflection of the shoring system, the shoring engineer and geotechnical engineer should be notified. Excessive deflection may require additional anchors and/or internal bracing to restrain the shoring system.

A representative of the geotechnical engineer should be present during grading to see temporary slopes. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward them. No vehicular surcharge should be allowed within three feet of the top of the cut.

CORROSION

The corrosiveness of the near surface soils at the site were determined at the laboratory of HDR, with the results summarized in Table I - Laboratory Tests of Soil Samples. The pH of the soils is near neutral and not a factor in corrosion. The chloride content is low and not a factor in design. The sulfate content is negligible and not a factor in concrete design. The resistivity indicates that the soils are severely corrosive to ferrous metals.

CONCRETE DECKING AND PAVING

A mat foundation is recommended at the basement level. Mats located below a depth of 18 feet should be designed to resist uplift forces due to the design groundwater (historic high water).

The mat should be cast over undisturbed alluvium or approved compacted fill. The undisturbed natural soils will provide adequate support to the lower-level floor slab. Any large cobbles that are encountered at the planned subterranean level should be removed and replaced with finer compacted material to obtain a level subgrade. To provide uniform slab support, the soils required to backfill voids should be compacted to at least 95 percent.

Construction activities and prolonged exposure to the environment can cause deterioration of prepared subgrades. Therefore, we recommend that appropriate tests be conducted on the final subgrade soils immediately prior to slab on grade construction to determine their condition and requirements for any remedial grading.

Concrete slabs and concrete decking outside the mat should be cast over undisturbed alluvium or approved compacted fill.

Floor slabs within the building that are subject to vehicle loads should be at least 5 inches thick and reinforced with a minimum of #4 bars on 16-inch centers, each way. Exterior slabs should be at least 4 inches thick and reinforced with a minimum of #4 bars on 16-inch centers, each way. Care should be taken to cast the reinforcement near the center of the slab.

For interior slabs and slabs with a floor covering, a moisture barrier is recommended. For performance and concrete curing, it recommended that the vapor barrier be 10-mil thick and placed over at least two inches of clean sand and then covered by at least two inches of clean sand. The topping sand is intended to prevent punctures during placement of the reinforcing steel and to and aid in the concrete cure.

Slabs which will be provided with a moisture-sensitive floor covering should be designed to resist moisture in conformance with ACI 302.2R-06 (Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Material). Specifications for under-slab vapor retarder/barrier are typically the responsibility of the architect or flooring specialist. We would be happy to assist the architect and/or flooring specialist on their specifications for moisture protection of slabs that are to receive moisture sensitive coverings.

Many agencies require floor slabs be constructed in conformance with the Green Building Code that requires slabs be poured directly on top of the vapor barrier, which is to be underlain by four inches of gravel. Since the vapor barrier is to be placed on the gravel, it is important to exercise care to prevent damaging the moisture barrier during construction. From a geotechnical engineering standpoint, a vapor barrier may be placed over 4 inches of gravel, provided that the vapor barrier is of sufficient strength to resist punctures and tearing. If plastic sheeting is used, this may require a greater than 10 mil thickness. Bentonitic barriers such as Miraclay or Volclay may also be used as long as they conform to the minimum requirements of durability, strength and waterproofing. Vapor barriers should conform to ASTM E 1745 and ACI 302.2R-06 (Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials).

Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

It should be noted that cracking of concrete floor slabs is very common during curing. The cracking occurs because concrete shrinks as it dries. Crack control joints which are commonly used in exterior decking to control such cracking are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the slab's performance. The minor shrinkage cracks which often form in interior slabs generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile. A mortar bed or slip sheet is recommended between the slab and tile to limit, the potential for cracking.

Slabs should be protected with a polyethylene plastic vapor barrier placed beneath the slab. This barrier is intended to prevent the upward migration of moisture from the subgrade soils through the porous concrete slab. It should be noted that vapor barriers are penetrated by any number of elements including water lines, drain lines, and footings. These barriers are therefore not completely watertight. It is recommended that a surface seal be placed on slabs which will receive a wood floor. The floor installer should be consulted regarding an adequate product.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street or approved locations in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Drainage control devices require periodic cleaning, testing and maintenance to remain effective.

Infiltration

Due to the shallow depth to groundwater, expansive nature of the shallow soils, and the liquefaction potential, onsite infiltration of surface runoff is not considered feasible.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾-inch crushed gravel to help the collection of water. Yard areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

PLAN REVIEW

Formal plans ready for submittal to the Building Department should be reviewed by Irvine Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

Please advise Irvine Geotechnical at least 24 hours prior to any required site visit. The agency approved plans and permits should be at the jobsite and available to our representative. The project consultant will perform the observation and post a notice at the jobsite of his visit and findings. This notice should be given to the agency inspector.

During construction, a number of reviews by this office are recommended to verify site geotechnical conditions and conformance with the intent of the recommendations for construction. Although not all possible geotechnical observation and testing services are required by the reviewing agency, the more site reviews requested, the lower the risk of future problems. It is recommended that all grading, foundation, and drainage excavations be seen by a representative of the geotechnical engineer PRIOR to placing fill, forms, pipe, concrete, or steel. Any fill which is placed should be approved, tested, and verified if used for engineering purposes. Temporary excavations should be observed by a representative of the Geotechnical Engineer.

The following site reviews are advised or required. Should the observations reveal any unforeseen hazards, the engineer will recommend treatment.

Pre-construction meeting	Advised
Temporary excavations	Required
Shoring pile, tie backs, and lagging installation	Required
Bottom excavation for removals	Required
Subdrains	Required
Compaction of fill	Required
Foundation excavations	Required
Slab subgrade moisture barrier membrane	Advised

Slab subgrade rock placement	Advised
Slab steel placement	Advised
Subdrain and rock placement behind retaining walls	Required
Compaction of retaining wall backfill	Required
Compaction of utility trench backfill	Advised

Irvine Geotechnical requires at least a 24-hour notice prior to any required site visits. The approved plans and building/grading permits should be on the job and available to the project consultant.

FINAL INSPECTION

Many projects are required by the agency to have final geologic and soils engineering reports upon completion of the grading.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist on a site, the area should be fenced and warning signs posted. All pile excavations must be properly covered and secured. Soil generated by foundation and subgrade excavations should be either removed from the site or properly placed as a certified compacted fill. Soil must not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep.

GENERAL CONDITIONS

This report and the exploration are subject to the following NOTICE. Please read the NOTICE carefully, it limits our liability.

NOTICE

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by us and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions and excavation characteristics described herein and shown on the enclosed cross sections have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations that may occur between these excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications or recommendations during construction requires the review of the geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report is issued and made for the sole use and benefit of the client, is not transferable and is as of the exploration date. Any liability in connection herewith shall not exceed the fee for the exploration. No warranty, expressed or implied, is made or intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN OR CONCEPT FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Irvine Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
Irvine Geotechnical, Inc.

Jon A. Irvine
E.G. 1691/G.E. 2891
https://irvinegeotech-my.sharepoint.com/personal/jirvine_irvinegeotech_com/Documents/Standard Form 2021A.docx

Enc: Appendix I - Laboratory Testing by Soil Labworks
- Moisture-Density Relationship (Plate A)
- Shear Test Diagrams (Plates B-1 through B-3)
- Consolidation Diagrams (Plates C-1 through C-6)
- Atterberg Limit Charts

Appendix II – Liquefaction Analysis (4)
Historic High Groundwater
Average N_{60} Blow Count Determination
Vicinity Map
Regional Geologic Map
Log of Borings (8 Pages)
Calculation Sheets (5)
Sections A through C
Site Plan

xc: (4) Addressee

STATEMENT OF RESPONSIBILITY - SOIL TESTING BY SOIL LABWORKS, LLC

Laboratory testing by Soil Labworks, LLC was performed under the supervision of the undersigned engineer. Irvine Geotechnical and Jon A. Irvine has reviewed referenced laboratory testing report dated April 16, 2021, and the results appear to be reasonable for this area of the Los Angeles. Irvine Geotechnical and the undersigned engineer concurs with the findings of Soil Labworks, LLC and accepts professional responsibility for utilizing the data.

SL21.3607
April 16, 2021

Irvine Geotechnical
145 N. Sierra Madre Boulevard
Suite 1
Pasadena, California 91107

Subject: Laboratory Testing

Site: 6435 Wilshire Blvd
Los Angeles, California

Job: IRVINE/BLACK EQUITIES-IC21035

Laboratory testing for the subject property was performed by Soil Labworks, LLC., under the supervision of the undersigned Engineer. Samples of the earth materials were obtained from the subject property by personnel of Irvine Geotechnical and transported to the laboratory of Soil Labworks for testing and analysis. The laboratory tests performed are described and results are attached.

Services performed by this facility for the subject property were conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions.

Respectfully Submitted:

SOIL LABWORKS, LLC

REGISTERED PROFESSIONAL ENGINEER
JON ALAN IRVINE
GE 2891
Exp. 6-30-22
GEOTECHNICAL
STATE OF CALIFORNIA

JON A. IRVINE
G.E. 2891

Enc: Appendix

2500 Townsgate Road, Suite E, Westlake Village, California 91361
(805) 370-1338 FAX (805) 371-4693

APPENDIX

Laboratory Testing

Sample Retrieval - Drill Rig

Samples of earth materials were obtained at frequent intervals by driving a thick-walled steel sampler conforming to the most recent version of ASTM D 3550/D 3550M-17 with successive drops of a 140 pound hammer falling 30" using an automatic trip hammer. The earth material was retained in brass rings of 2.416 inches inside diameter and 1.00 inch height. The central portion of the sample was stored in close-fitting, water-tight containers for transportation to the laboratory. Standard Penetration Tests (SPT) were performed at discrete intervals within the 8 inch diameter, hollow stem auger borings drilled on the site. The tests were performed using the 1-3/8 inch inside diameter, split-barrel sampler in accordance with ASTMD1586-11. Standard penetration test samples were retained in air-tight bags.

Moisture Density

The field moisture content and dry density were determined for each of the soil samples. The dry density was determined in pounds per cubic foot following ASTM 2937-17e2. The moisture content was determined as a percentage of the dry soil weight conforming to ASTM 2216-19. The results are presented below in the following table. The percent saturation was calculated on the basis of an estimated specific gravity. Description of earth materials used in this report and shown on the attached Plates were provided by the client.

Test Pit/Boring No.	Sample Depth (Feet)	Soil Type	Dry Density (pcf)	Moisture Content (percent)	Percent Saturation (G_s=2.65)
B1	5	Alluvium	115.3	17.4	100
B1	10	Alluvium	95.7	27.9	100
B1	15	Alluvium	120.1	6.6	47
B1	50	Alluvium	-	25.9	-
B1	55	Alluvium	-	26.8	-
B1	60	Alluvium	-	24.8	-
B1	65	Alluvium	-	24.4	-
B1	70	Alluvium	-	35.5	-
B2	5	Alluvium	106.9	21.1	100
B2	10	Alluvium	100.6	28.6	100
B2	15	Alluvium	113.2	15.5	89
B2	20	Alluvium	117.7	14.4	94
B2	25	Alluvium	94.8	31.2	100
B2	30	Alluvium	114.6	15.8	95
B2	35	Alluvium	105.8	22.5	100

B2	40	Alluvium	103.3	23.4	100
B2	45	Alluvium	97.4	28.0	100
B2	50	Alluvium	117.0	16.9	100
B2	55	Alluvium	123.5	13.1	100
B2	60	Alluvium	100.8	24.7	100

Compaction Character

Compaction tests were performed on bulk samples of the earth materials in accordance with ASTM D1557-12ei. The results of the tests are provided on the table below and on the "Moisture-Density Relationship", A-Plates. The specific gravity of the alluvium was estimated from the compaction curves.

Test Pit/Boring No.	Sample Depth (Feet)	Soil Type	Maximum Dry Density (pcf)	Optimum Moisture Content (Percent)
B1	15-20	Alluvium	126.0	9.2

Shear Strength

The peak and ultimate shear strengths of the alluvium were determined by performing consolidated and drained direct shear tests in conformance with ASTM D3080/D3080M-11. The tests were performed in a strain-controlled machine manufactured by GeoMatic. The rate of deformation was 0.01 inches per minute. Samples were sheared under varying confining pressures, as shown on the "Shear Test Diagrams," B-Plates. Remolded samples were prepared at 90 percent of the maximum density for shear tests. The remolding procedure consists of selecting a representative sample from a bulk bag and sieving it through a No. 4 sieve. The moisture content of the material is then determined. A formula is then used to calculate the weight of the material that must fit in a ring when compacted to 90 percent of the maximum density. This calculated amount of material is then weighed out and pounded into a ring until all the material is used and the ring is full. The moisture conditions during testing are shown on the following table and on the B-Plates. The samples indicated as saturated were artificially saturated in the laboratory. All saturated samples were sheared under submerged conditions.

Test Pit/ Boring No.	Sample Depth (Feet)	Dry Density (pcf)	As-Tested Moisture Content (percent)
B2	5	106.9	26.8
B2	15	113.2	19.8
B1*	15-20	113.4	21.6

* Sample remolded to 90 % of the laboratory maximum density.
** Sample repeatedly sheared to determine residual strength.

Consolidation

One-dimensional consolidation tests were performed on samples of the alluvium in a consolidometer manufactured by GeoMatic in conformance with ASTM D2435/D2435M-11. The tests were performed on 1-inch high samples retained in brass rings. The samples were initially loaded to approximately ½ of the field over-burden pressure and then unloaded to compensate for the effects of possible disturbance during sampling. Loads were then applied in a geometric progression and resulting deformation recorded. Water was added at a specific load to determine the effect of saturation. The results are plotted on the "Consolidation Test," C-Plates.

Expansion Index

The expansive character of the alluvium was determined by performing Expansion Index Tests in accordance with UBC 18.2 and ASTM 4829-11. A bulk sample of earth material was compacted at a specific moisture content using one fifth the compacted energy for the modified proctor test. The sample was then saturated and the expansion measured. The results of the tests are provided on the following table.

Test Pit No.	Sample Depth (Feet)	Soil Type	Expansion Index
B1	15-20	Alluvium	58

Atterberg Limits

Atterberg limits determinations were performed on samples of the soil/alluvium in accordance with ASTMD4318-17e1. The test results are presented on the table below.

Test Pit/Boring No.	Sample Depth (Ft)	Soil Type	Liquid Limit	Plastic Limit	Plasticity Index
B1	50	Alluvium	41	21	20
B1	55	Alluvium	51	20	31
B1	60	Alluvium	39	18	21
B1	65	Alluvium	42	19	23
B1	70	Alluvium	43	18	25

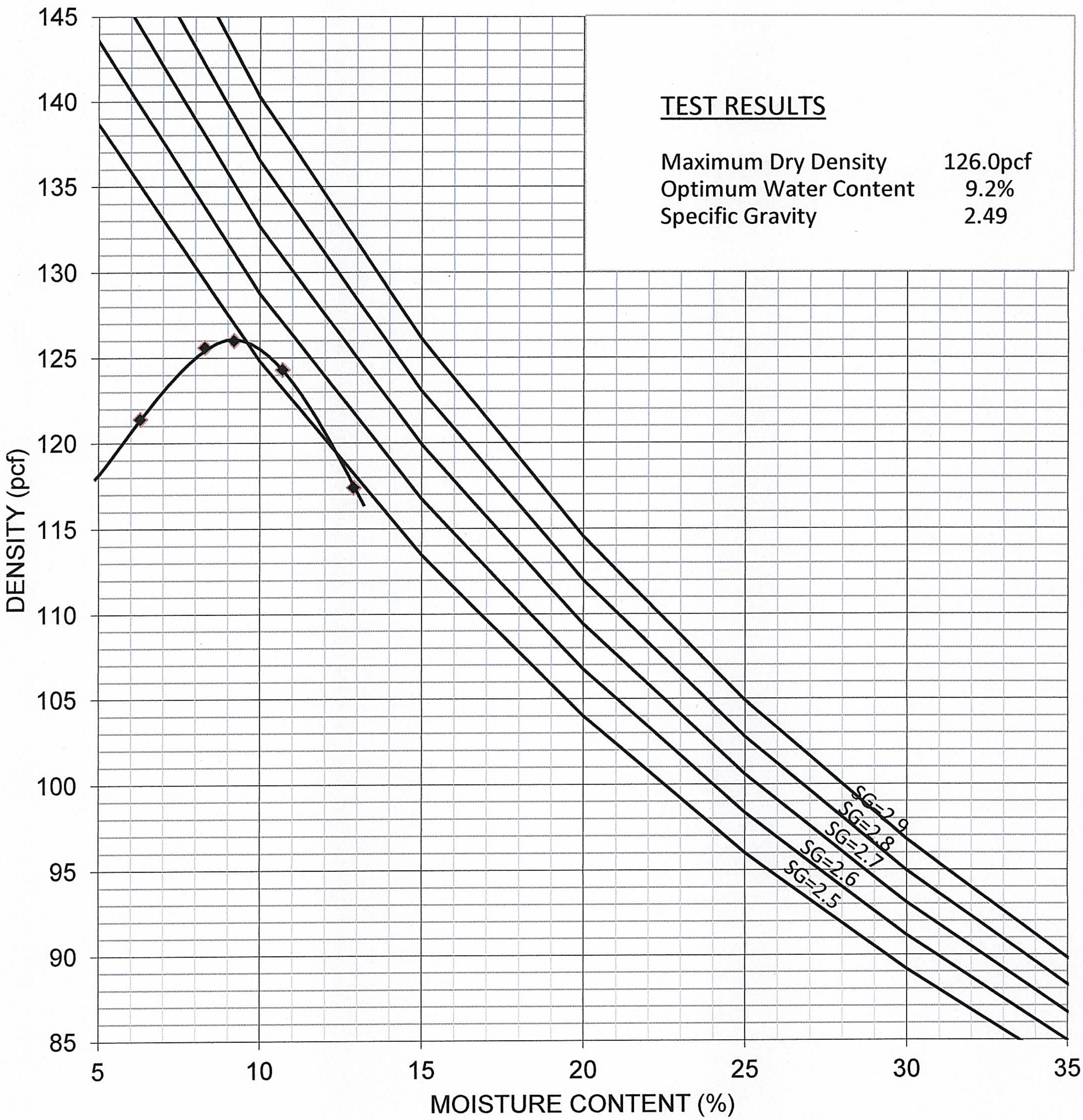
Grain Size Distribution

The amount of material in the soil finer than 1 No. 200 sieve was determined on selected samples in conformance with ASTM D1140-17. Wash sieving disperses clay and other fine material that are removed from the soil during the test. The percent of fine material in the soil sample is the calculated base on the loss of mass. The results are present in the table below.

Boring No	Depth	Soil Type	(%) Passing 200 Sieve
B1	17.5	Alluvium	59.7
B1	20	Alluvium	58.0
B1	27.5	Alluvium	44.0
B1	30	Alluvium	43.9
B1	37.5	Alluvium	33.6
B1	40	Alluvium	28.4
B1	42.5	Alluvium	29.7
B1	45	Alluvium	30.1
B1	47.5	Alluvium	22.4

SOIL LABWORKS LLC	**MOISTURE-DENSITY RELATIONSHIP A-1**
	JN: **SL21.3607** CONSULTANT: **JAI** CLIENT: **IRVINE/Black Equities-6435 Wilshire Blvd** **B1 @ 15-20'** EARTH MATERIAL: **ALLUVIUM**

NOTE: ASTM Test Method D-1557-12



TEST RESULTS

Maximum Dry Density	126.0pcf
Optimum Water Content	9.2%
Specific Gravity	2.49

SOIL LABWORKS LLC

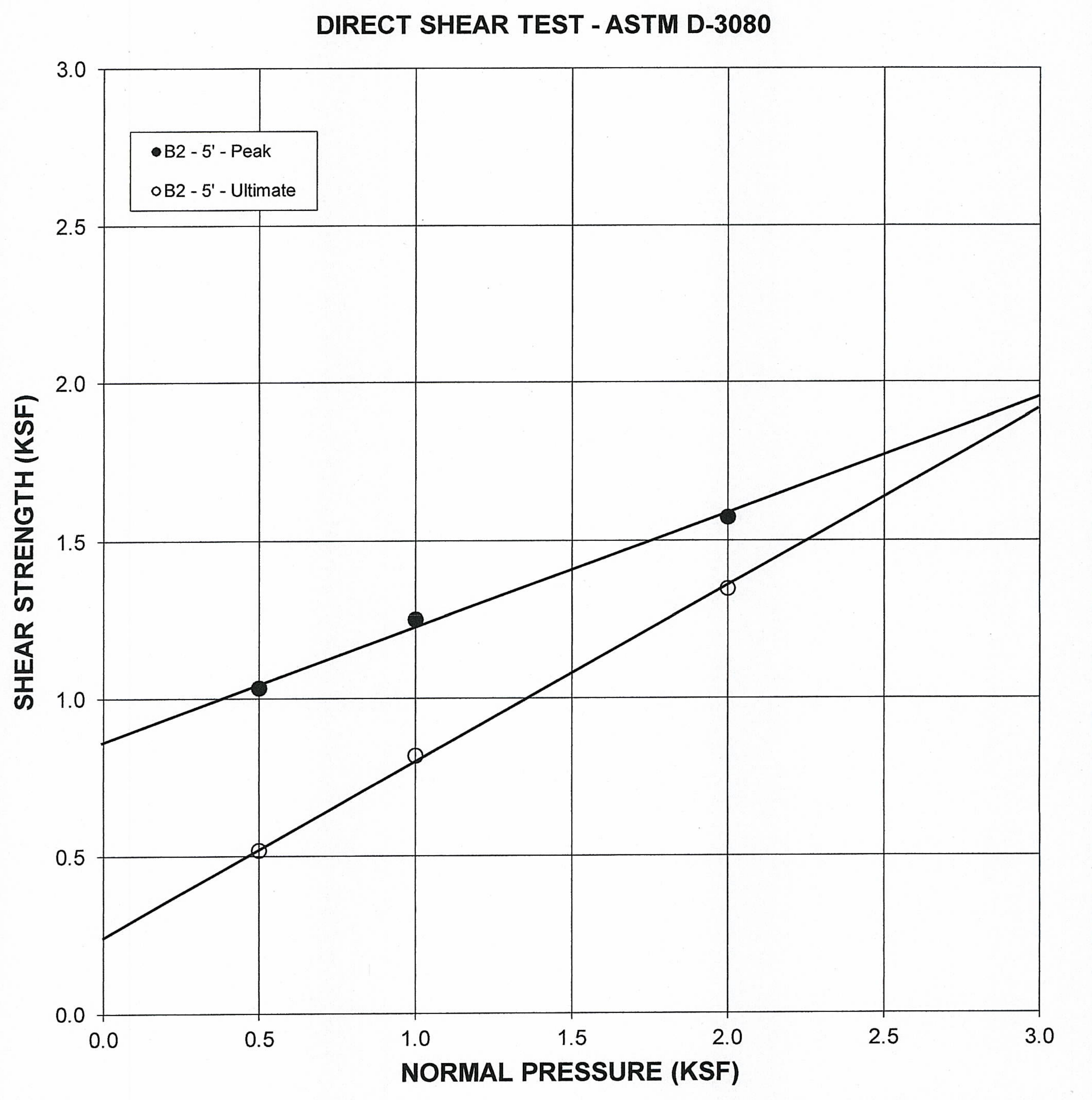
SHEAR DIAGRAM B-1

JN: SL20.3607 CONSULTANT JAI
CLIENT: Irvine/Black Equities-6435 Wilshire Blvd

EARTH MATERIAL: ALLUVIUM

	PEAK	ULTIMATE	
Phi Angle	20	29	degrees
Cohesion	860	250	psf

Average Moisture Content	26.8%
Average Dry Density (pcf)	106.6
Percent Saturation	100.0%



SOIL LABWORKS LLC

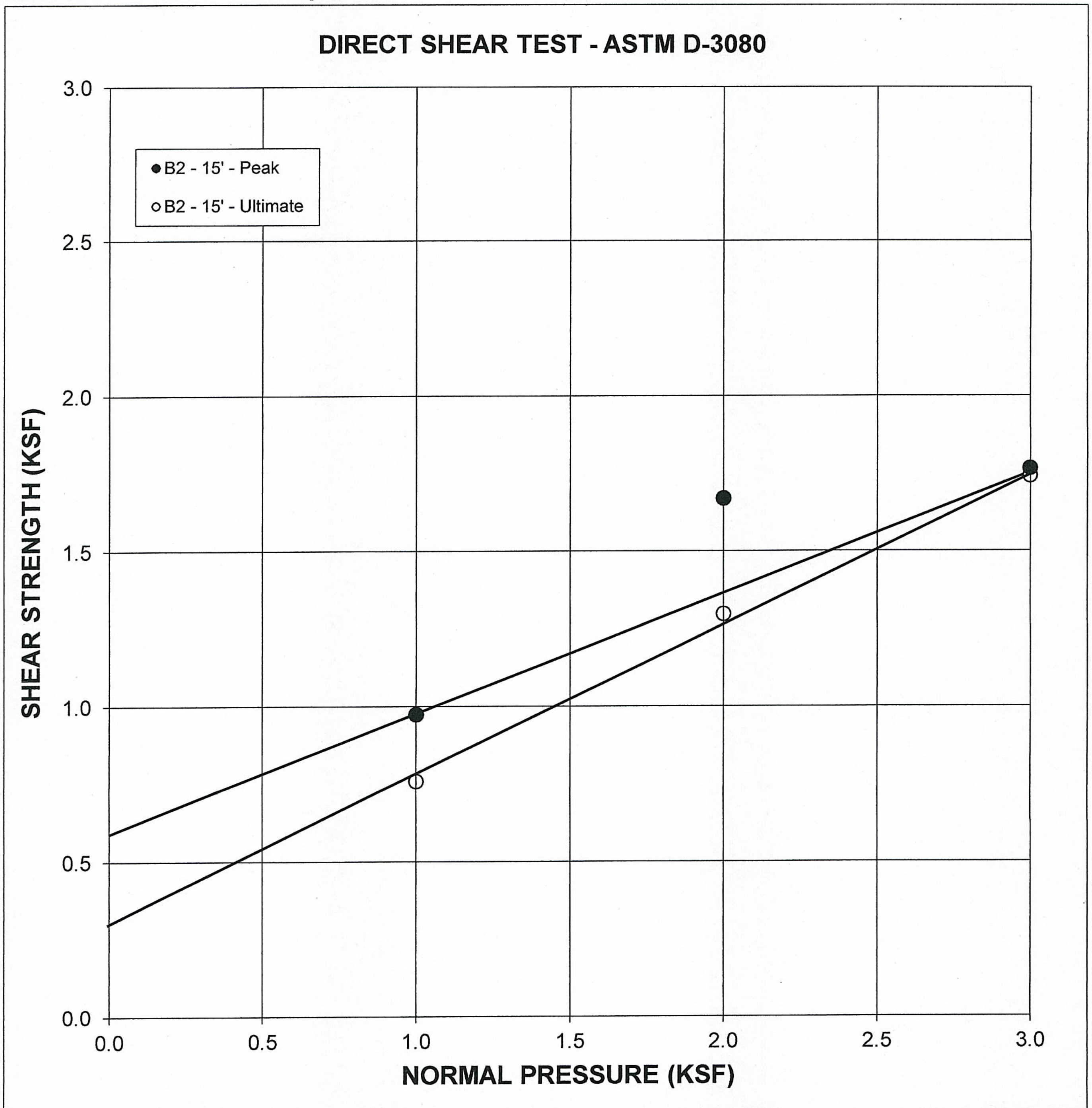
SHEAR DIAGRAM B-2

JN: SL20.3607 CONSULTANT JAI
CLIENT: Irvine/Black Equities-6435 Wilshire Blvd

EARTH MATERIAL: ALLUVIUM

	PEAK	ULTIMATE	
Phi Angle	21.5	26	degrees
Cohesion	580	290	psf

Average Moisture Content	19.8%
Average Dry Density (pcf)	113.2
Percent Saturation	100.0%



SHEAR DIAGRAM B-3

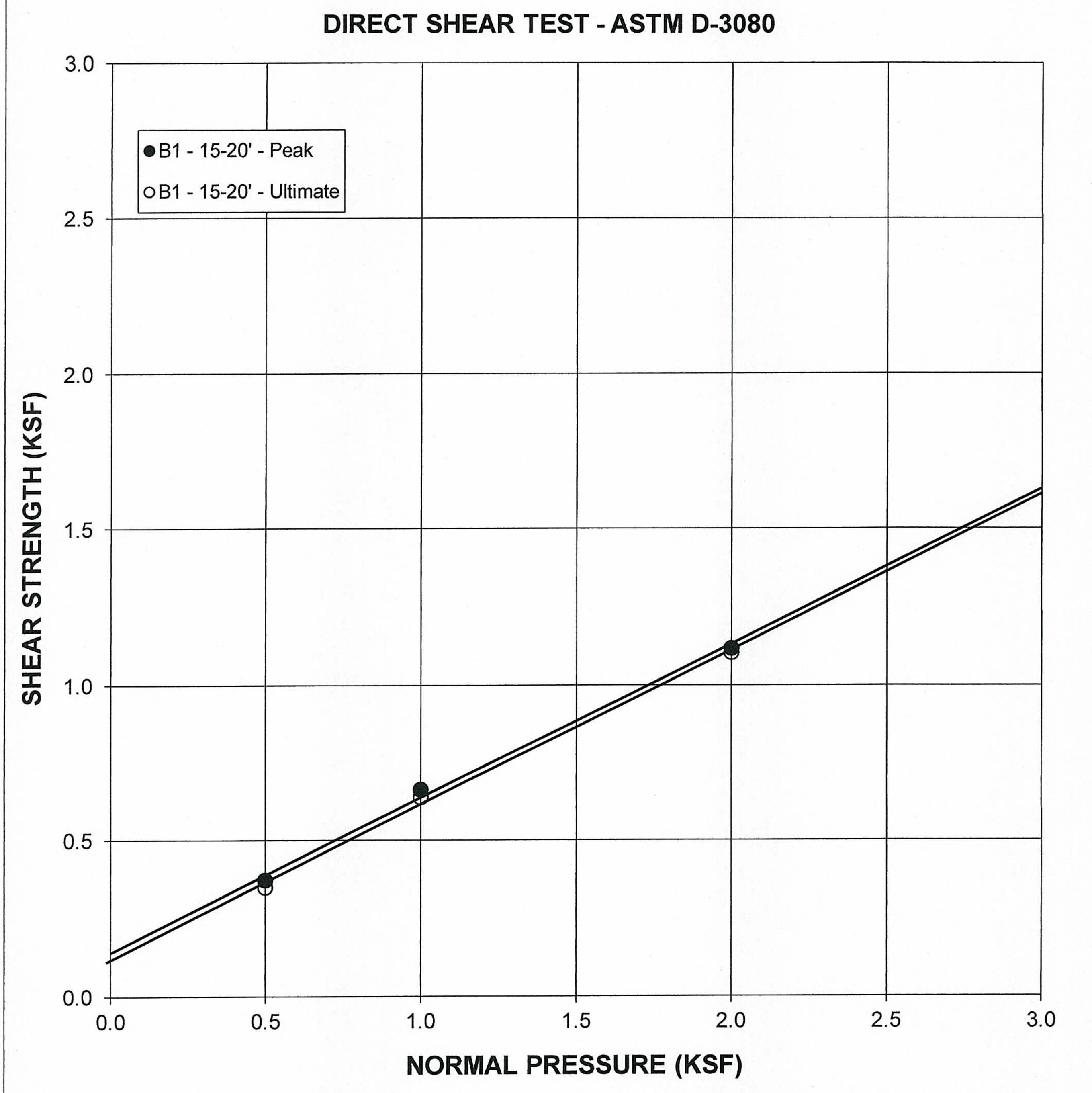
JN: **SL21.3607** CONSULTANT **JAI**
CLIENT: **Irvine/Black Equities-6435 Wilshire Blvd**

EARTH MATERIAL: **ALLUVIUM**

Sample remolded to 90 % of the laboratory maximum density

	PEAK	ULTIMATE	
Phi Angle	26.5	26.5	degrees
Cohesion	140	120	psf

Average Moisture Content	21.6%
Average Dry Density (pcf)	113.4
Percent Saturation	100.0%



SOIL LABWORKS LLC

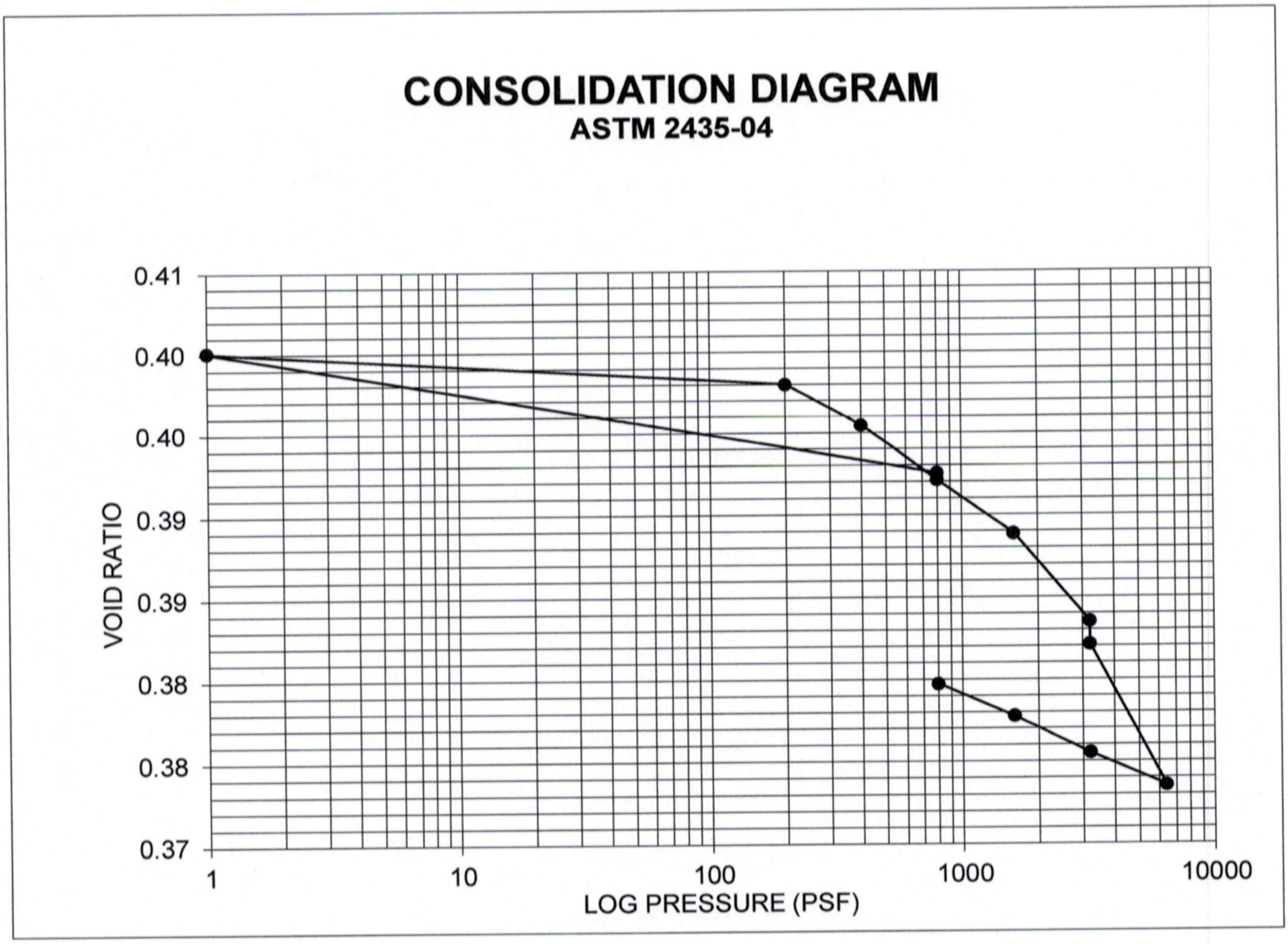
CONSOLIDATION DIAGRAM #1

IC: **21035** CONSULTANT: **JAI**

CLIENT: **6435 WILSHIRE PROJECT**

PLATE C-1

Earth Material:	Alluvium		
Sample Location:	B2@20'	Specific Gravity:	2.65
Dry Weight (pcf):	117.7	Initial Void Ratio:	0.405
Initial Moisture:	14.4%	Water Added At (psf):	3200
Initial Saturation:	94.1%	Consolidation Coef. (Cc):	0.0285
		Reloading Coef. (Cr):	0.0014



SOIL LABWORKS LLC

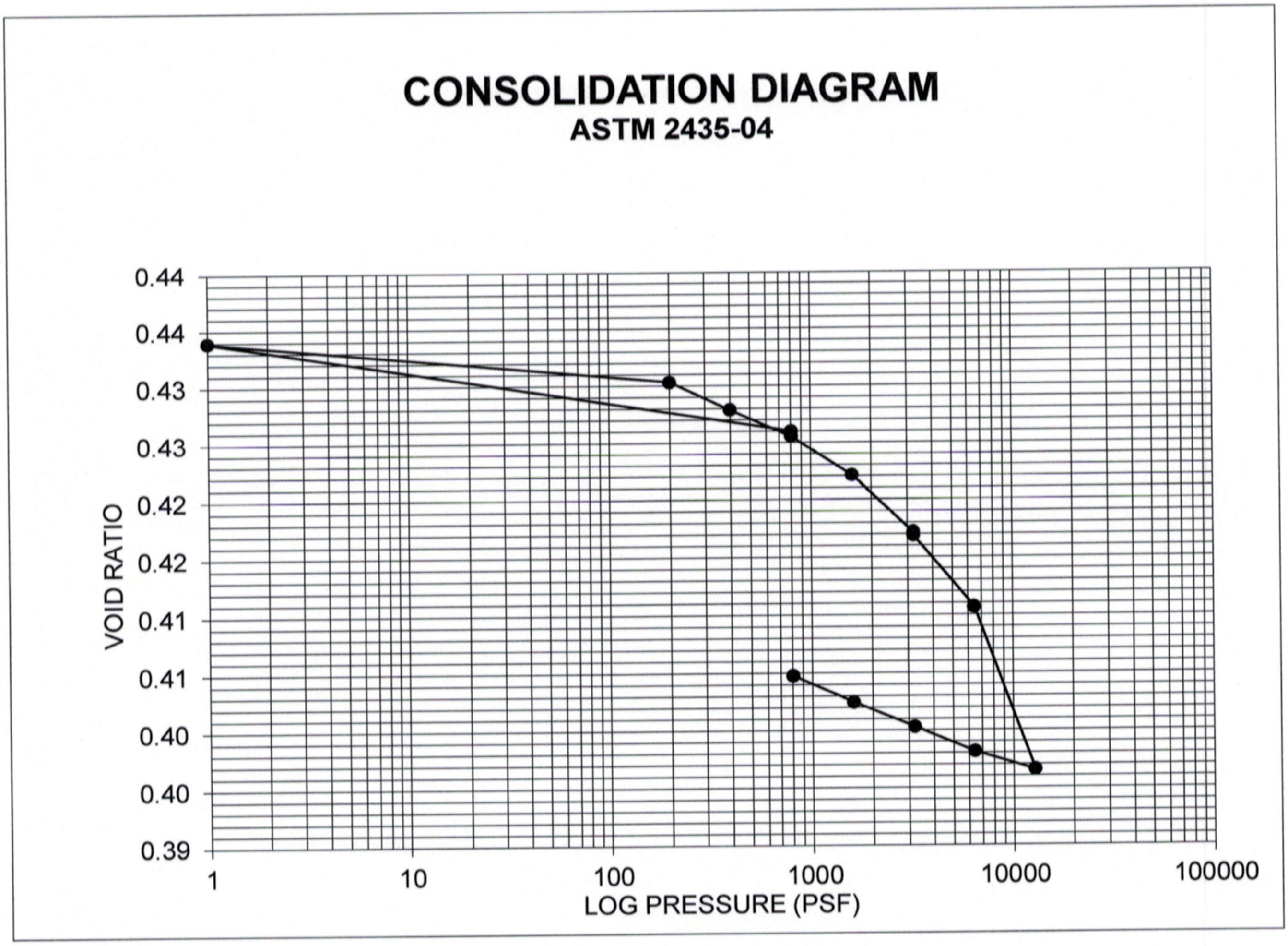
CONSOLIDATION DIAGRAM #2

IC: **21035** CONSULTANT: **JAI**

CLIENT: **6435 WILSHIRE PROJECT**

PLATE C-2

Earth Material:	Alluvium		
Sample Location:	B2@30'	Specific Gravity:	2.65
Dry Weight (pcf):	114.6	Initial Void Ratio:	0.443
Initial Moisture:	15.8%	Water Added At (psf):	3200
Initial Saturation:	94.5%	Consolidation Coef. (Cc):	0.0470
		Reloading Coef. (Cr):	0.0030



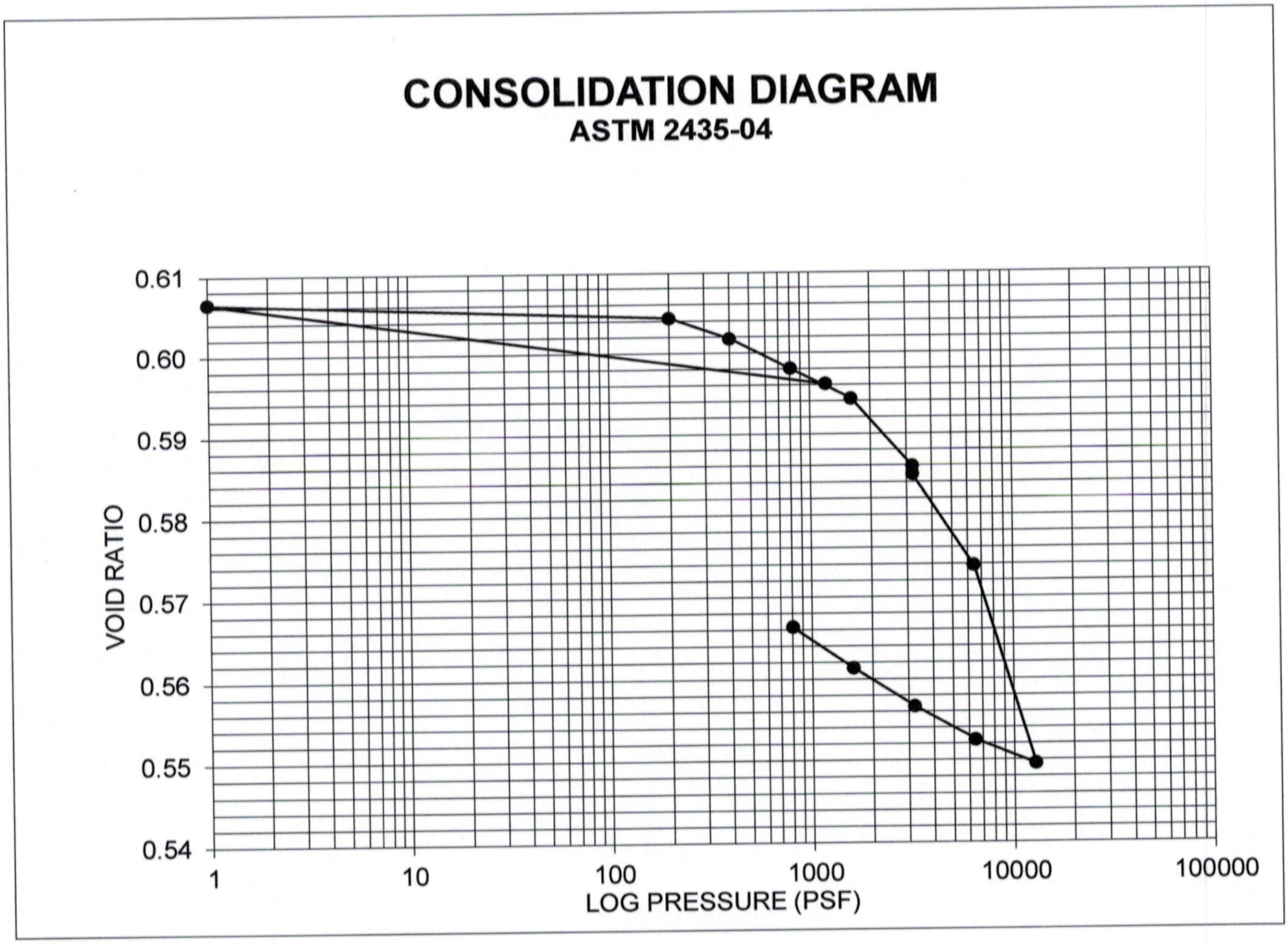
CONSOLIDATION DIAGRAM #3

IC: **21035** CONSULTANT: **JAI**

CLIENT: **6435 WILSHIRE PROJECT**

PLATE C-3

Earth Material:	Alluvium		
Sample Location:	B2@35'	Specific Gravity:	2.74
Dry Weight (pcf):	105.8	Initial Void Ratio:	0.617
Initial Moisture:	22.5%	Water Added At (psf):	3200
Initial Saturation:	99.9%	Consolidation Coef. (Cc):	0.0806
		Reloading Coef. (Cr):	0.0051



SOIL LABWORKS LLC

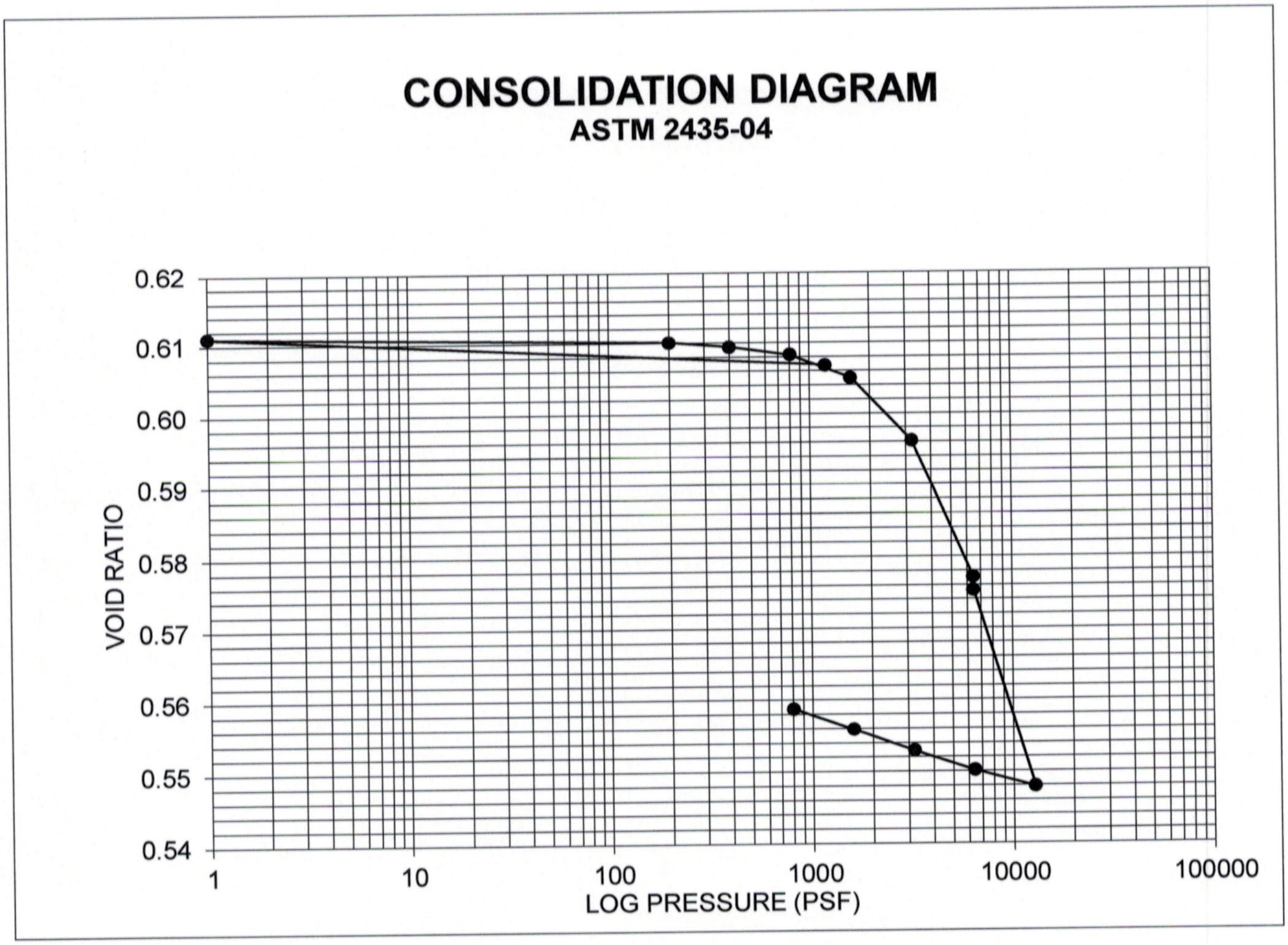
CONSOLIDATION DIAGRAM #4

IC: 21035 CONSULTANT: JAI

CLIENT: 6435 WILSHIRE PROJECT

PLATE C-4

Earth Material:	Alluvium		
Sample Location:	B2@40'	Specific Gravity:	2.70
Dry Weight (pcf):	103.3	Initial Void Ratio:	0.633
Initial Moisture:	23.4%	Water Added At (psf):	6400
Initial Saturation:	100.0%	Consolidation Coef. (Cc):	0.0911
		Reloading Coef. (Cr):	0.0062



SOIL LABWORKS LLC

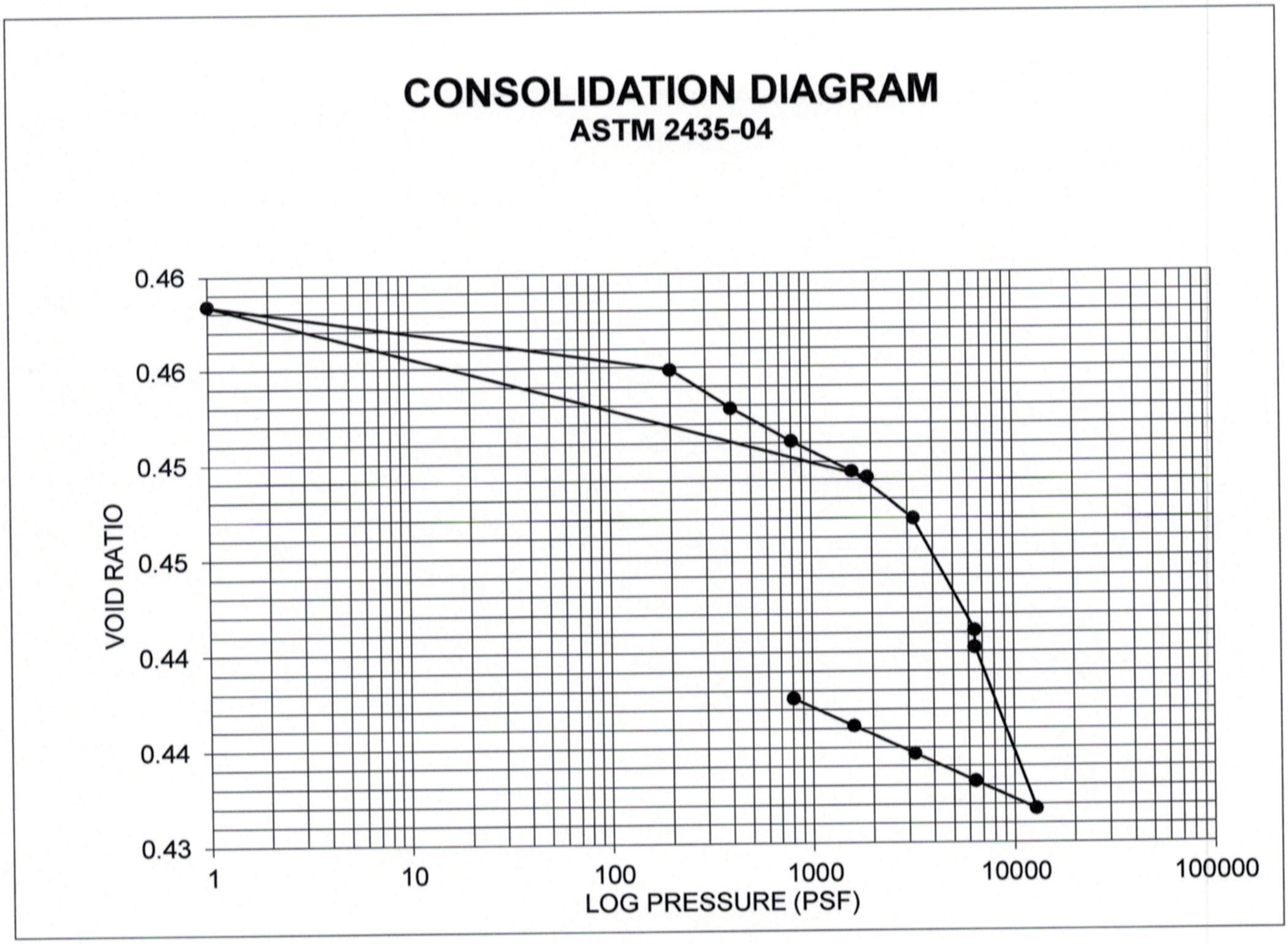
CONSOLIDATION DIAGRAM #5

IC: **21035** CONSULTANT: **JAI**

CLIENT: **6435 WILSHIRE PROJECT**

PLATE C-5

Earth Material:	Alluvium		
Sample Location:	B2@50'	Specific Gravity:	2.75
Dry Weight (pcf):	117.0	Initial Void Ratio:	0.464
Initial Moisture:	16.9%	Water Added At (psf):	6400
Initial Saturation:	100.0%	Consolidation Coef. (Cc):	0.0282
		Reloading Coef. (Cr):	0.0017



SOIL LABWORKS LLC

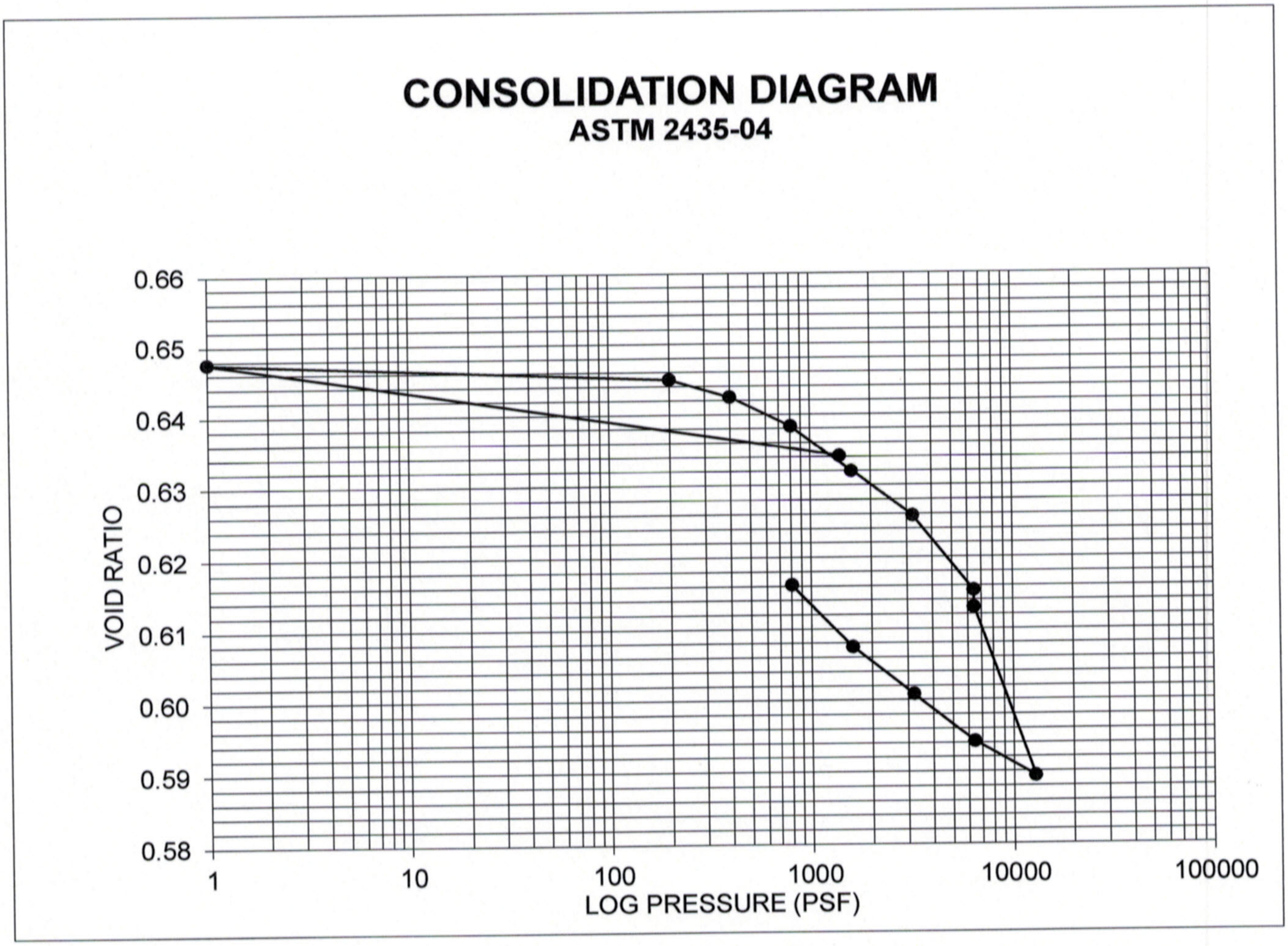
CONSOLIDATION DIAGRAM #6

IC: **21035** CONSULTANT: **JAI**

CLIENT: **6435 WILSHIRE PROJECT**

PLATE C-6

Earth Material:	Alluvium		
Sample Location:	B2@60'	Specific Gravity:	2.69
Dry Weight (pcf):	100.8	Initial Void Ratio:	0.665
Initial Moisture:	24.7%	Water Added At (psf):	6400
Initial Saturation:	99.9%	Consolidation Coef. (Cc):	0.0785
		Reloading Coef. (Cr):	0.0041

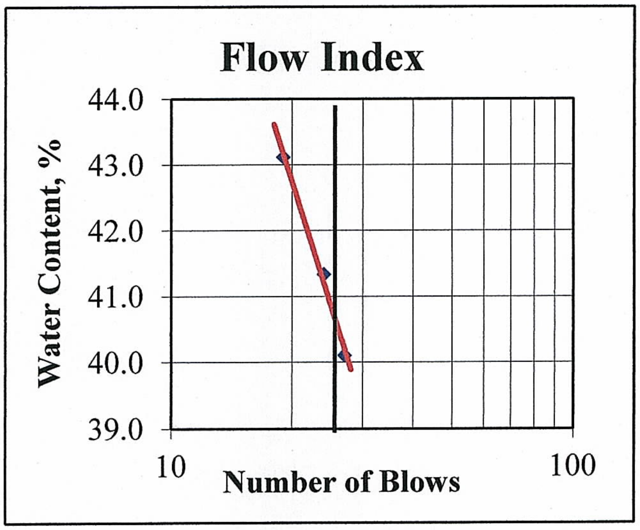


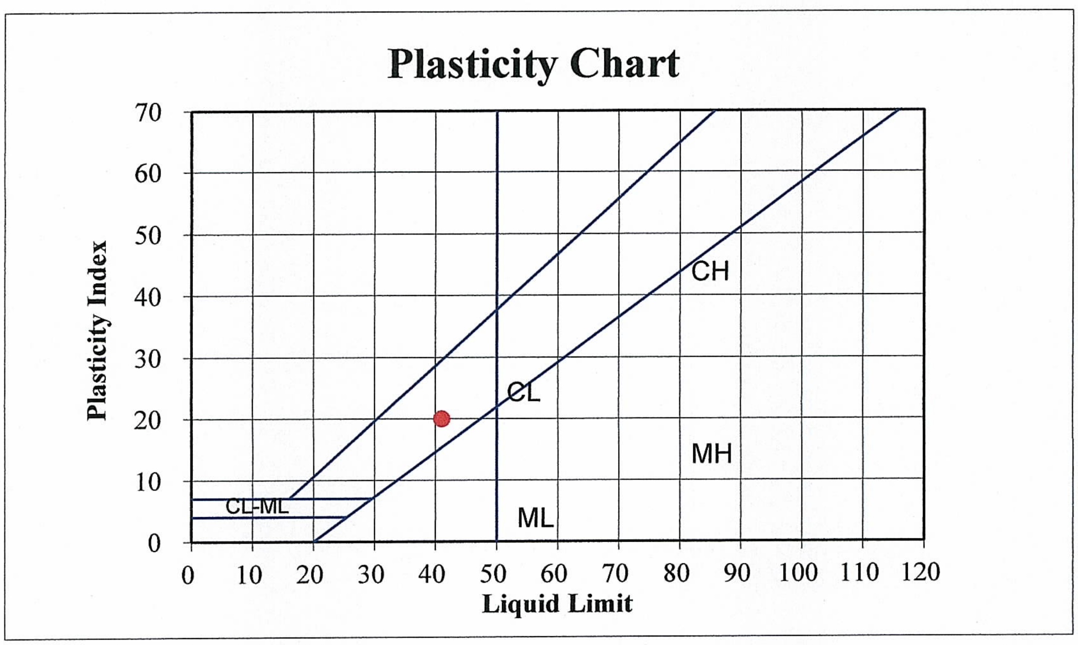
PLASTICITY INDEX

ASTM D-4318

Job Name: Irvine/Black Equities-6435 Wilshire Blvd
Sample ID: B1 @ 50'
Soil Description: CL

DATA SUMMARY				TEST RESULTS	
Number of Blows:	19	24	27	**LIQUID LIMIT**	**41**
Water Content, %	43.1	41.3	40.1	**PLASTIC LIMIT**	**21**
Plastic Limit:	20.9	20.7		**PLASTICITY INDEX**	**20**





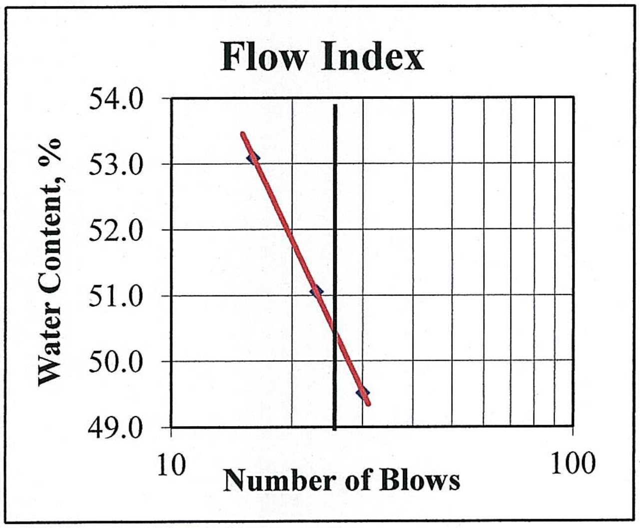
File No.: SL21.3607 April 16, 2021

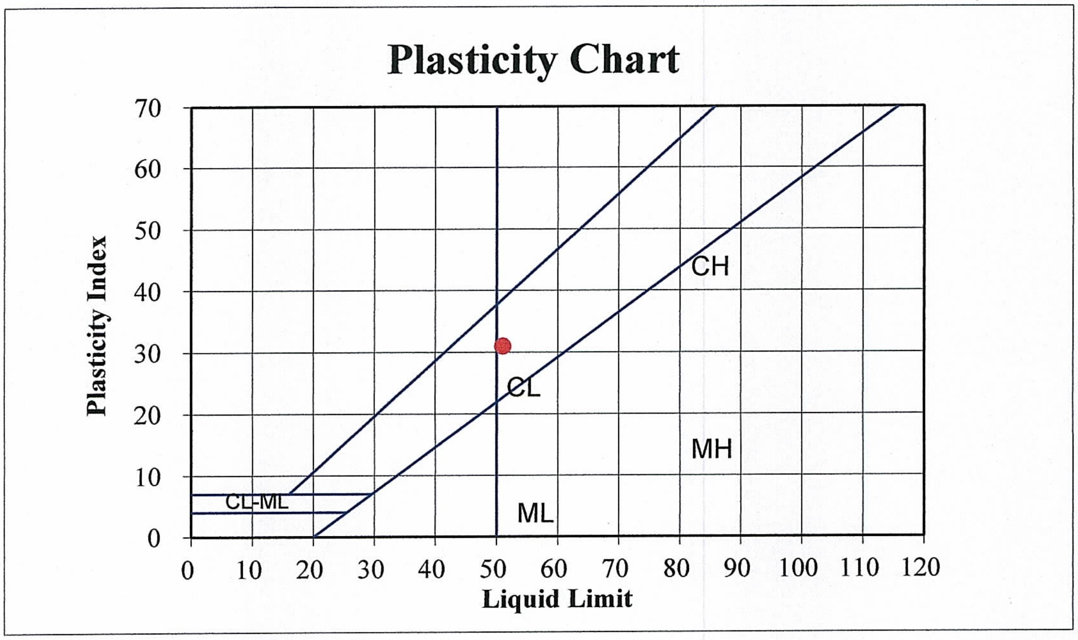
PLASTICITY INDEX

ASTM D-4318

Job Name: Irvine/Black Equities-6435 Wilshire Blvd
Sample ID: B1 @ 55'
Soil Description: CL

DATA SUMMARY				TEST RESULTS	
Number of Blows:	16	23	30	**LIQUID LIMIT**	**51**
Water Content, %	53.1	51.1	49.5	**PLASTIC LIMIT**	**20**
Plastic Limit:	20.2	20.5		**PLASTICITY INDEX**	**31**



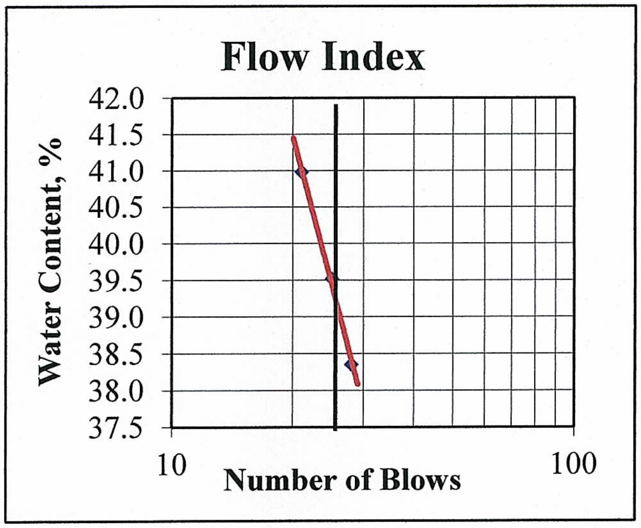


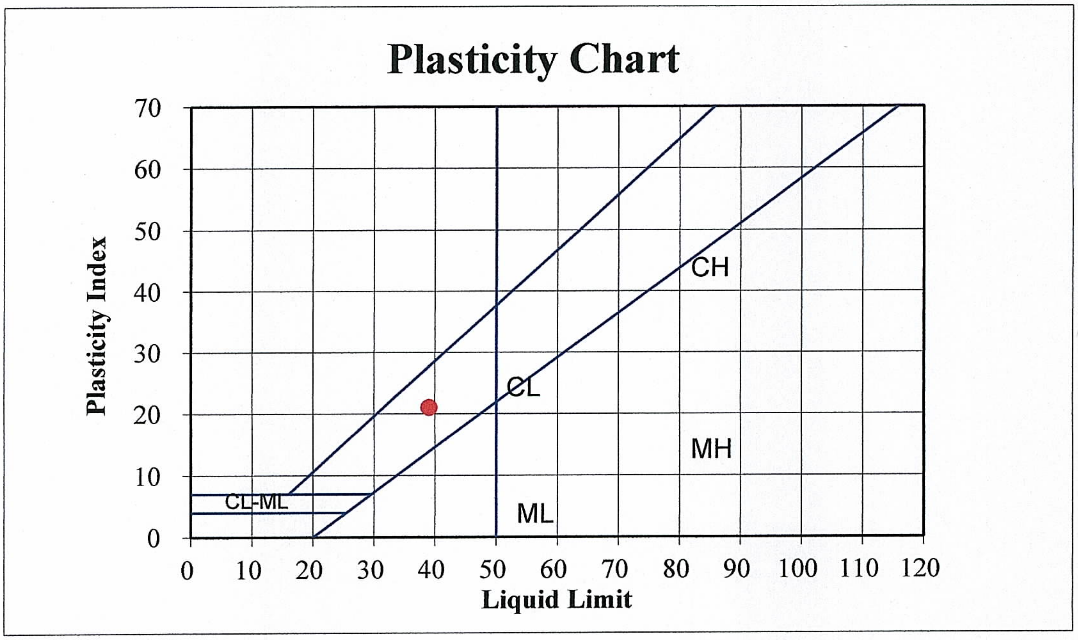
PLASTICITY INDEX

ASTM D-4318

Job Name: Irvine/Black Equities-6435 Wilshire Blvd
Sample ID: B1 @ 60'
Soil Description: CL

DATA SUMMARY				TEST RESULTS	
Number of Blows:	21	25	28	**LIQUID LIMIT**	**39**
Water Content, %	41.0	39.5	38.4	**PLASTIC LIMIT**	**18**
Plastic Limit:	17.6	17.8		**PLASTICITY INDEX**	**21**



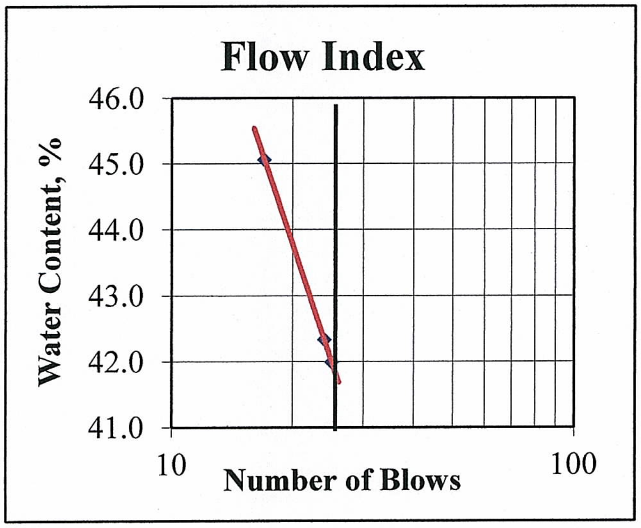


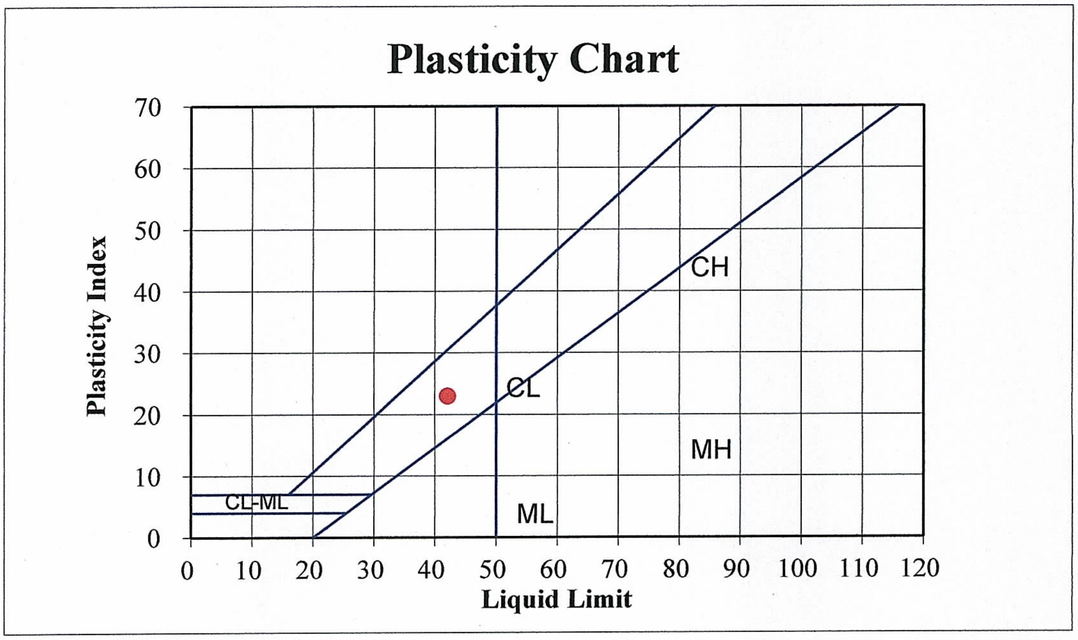
PLASTICITY INDEX

ASTM D-4318

Job Name: Irvine/Black Equities-6435 Wilshire Blvd
Sample ID: B1 @ 65'
Soil Description: CL

DATA SUMMARY				TEST RESULTS	
Number of Blows:	17	24	25	LIQUID LIMIT	42
Water Content, %	45.1	42.3	42.0	PLASTIC LIMIT	19
Plastic Limit:	18.8	18.8		PLASTICITY INDEX	23





File No.: SL21.3607 April 16, 2021

PLASTICITY INDEX

ASTM D-4318

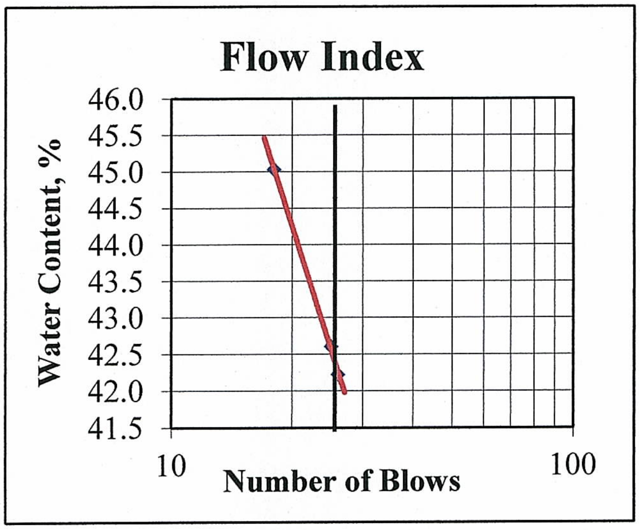
Job Name: Irvine/Black Equities-6435 Wilshire Blvd
Sample ID: B1 @ 70'
Soil Description: CL

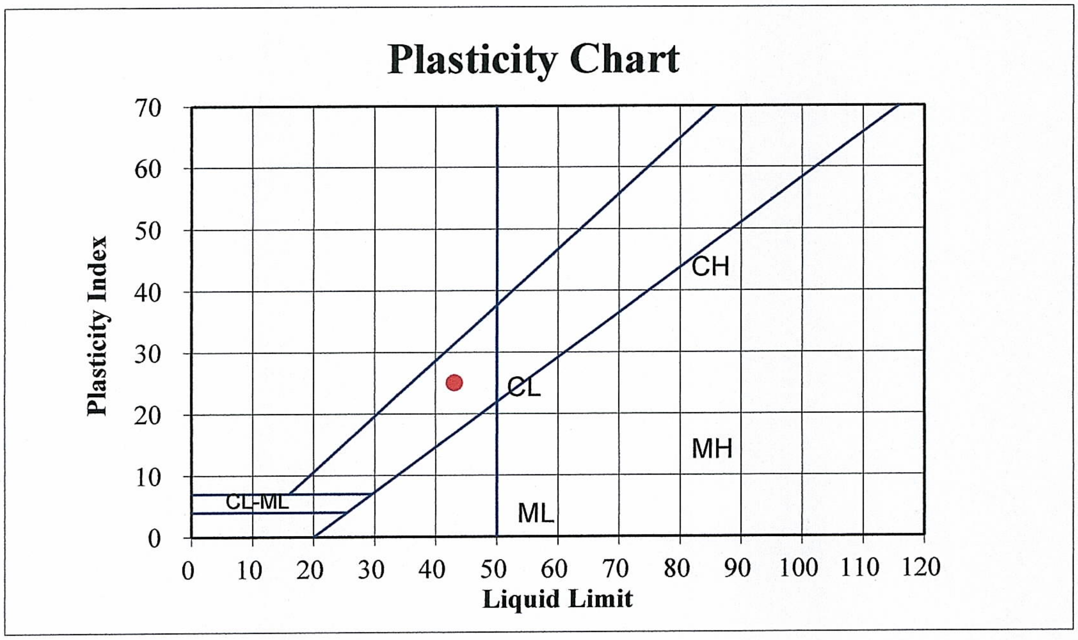
DATA SUMMARY

Number of Blows:	18	25	26
Water Content, %	45.0	42.6	42.2
Plastic Limit:	17.8	18.1	

TEST RESULTS

LIQUID LIMIT	**43**
PLASTIC LIMIT	**18**
PLASTICITY INDEX	**25**





IRVINE GEOTECHNICAL Inc

RETAINING WALL

IC: **21035** CONSULT: **JAI**
CLIENT: **BLACK EQUITIES**

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	COMPACTED FILL	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	B-3	BACKSLOPE ANGLE:	0 degrees
COHESION:	140 psf	SURCHARGE:	0 pounds
PHI ANGLE:	26.5 degrees	SURCHARGE TYPE:	P Point
DENSITY	136 pcf	INITIAL FAILURE ANGLE:	17 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	17.6 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	140.0 psf	FINAL TENSION CRACK:	25 feet

PHID = ATAN(TAN(PHI)/FS) = 26.5 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.325 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	40 degrees
AREA OF TRIAL FAILURE WEDGE	83.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	11367.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1296 trials
LENGTH OF FAILURE PLANE	15.7 feet
DEPTH OF TENSION CRACK	1.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	12.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**4151.0 pounds**

THE CALCULATION INDICATES THAT FOR THE DESIGN GROUND MOTION, THE UNBALANCED FORCE ON 12-FOOT RETAINING WALLS IS 4.152 KIPS.

IRVINE GEOTECHNICAL Inc

RETAINING WALL

IC: **21035** CONSULT: **JAI**
CLIENT: **BLACK EQUITIES**

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	ALLUVIUM	WALL HEIGHT	30 feet
SHEAR DIAGRAM:	B-2	BACKSLOPE ANGLE:	0 degrees
COHESION:	290 psf	SURCHARGE:	0 pounds
PHI ANGLE:	26 degrees	SURCHARGE TYPE:	P Point
DENSITY	136 pcf	INITIAL FAILURE ANGLE:	17 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	17.3 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	290.0 psf	FINAL TENSION CRACK:	30 feet

PHID = ATAN(TAN(PHI)/FS) = 26.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.325 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	40 degrees
AREA OF TRIAL FAILURE WEDGE	522.4 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	71047.1 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1566 trials
LENGTH OF FAILURE PLANE	39.2 feet
DEPTH OF TENSION CRACK	4.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	30.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**28364.0 pounds**

THE CALCULATION INDICATES THAT FOR THE DESIGN GROUND MOTION, THE UNBALANCED FORCE ON 30-FOOT RETAINING WALLS IS 28.365 KIPS.

IRVINE GEOTECHNICAL Inc	SHORING PILE
	IC: **21035** CONSULT: **JAI** CLIENT: **BLACK EQUITIES** CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	ALLUVIUM	RETAINED LENGTH	20 feet
SHEAR DIAGRAM:	B-2	BACKSLOPE ANGLE:	0 degrees
COHESION:	290 psf	SURCHARGE:	0 pounds
PHI ANGLE:	26 degrees	SURCHARGE TYPE:	P Point
DENSITY	136 pcf	INITIAL FAILURE ANGLE:	17 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	17.3 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	232.0 psf	FINAL TENSION CRACK:	30 feet

PHID = ATAN(TAN(PHI)/FS) = 21.3 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	54 degrees
AREA OF TRIAL FAILURE WEDGE	136.7 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	18595.1 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1566 trials
LENGTH OF FAILURE PLANE	18.7 feet
DEPTH OF TENSION CRACK	4.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	11.0 feet
CALCULATED THRUST ON PILE	**5865.2 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**29.3 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**30.0 pcf**

THE CALCULATION INDICATES THAT THE PROPOSED SHORING SUPPORITING EXCAVATION TO TO 20 FEET MAY MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 30 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

IRVINE GEOTECHNICAL Inc

SHORING PILE

IC: **21035** CONSULT: **JAI**
CLIENT: **BLACK EQUITIES**

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	ALLUVIUM	RETAINED LENGTH	27 feet
SHEAR DIAGRAM:	B-2	BACKSLOPE ANGLE:	0 degrees
COHESION:	290 psf	SURCHARGE:	0 pounds
PHI ANGLE:	26 degrees	SURCHARGE TYPE:	P Point
DENSITY	136 pcf	INITIAL FAILURE ANGLE:	17 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	17.3 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	232.0 psf	FINAL TENSION CRACK:	30 feet

PHID = ATAN(TAN(PHI)/FS) = 21.3 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	53 degrees
AREA OF TRIAL FAILURE WEDGE	267.2 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	36344.9 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1566 trials
LENGTH OF FAILURE PLANE	28.2 feet
DEPTH OF TENSION CRACK	4.4 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	17.0 feet
CALCULATED THRUST ON PILE	**12626.9 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**34.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**35.0 pcf**

THE CALCULATION INDICATES THAT THE PROPOSED SHORING SUPPORTING EXCAVATIONS UP TO 27 FEET MAY MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 35 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

IRVINE GEOTECHNICAL Inc

SHORING PILE

IC: **21035** CONSULT: **JAI**
CLIENT: **BLACK EQUITIES**

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	ALLUVIUM	RETAINED LENGTH	35 feet
SHEAR DIAGRAM:	B-2	BACKSLOPE ANGLE:	0 degrees
COHESION:	290 psf	SURCHARGE:	0 pounds
PHI ANGLE:	26 degrees	SURCHARGE TYPE:	P Point
DENSITY	136 pcf	INITIAL FAILURE ANGLE:	17 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	17.3 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	232.0 psf	FINAL TENSION CRACK:	30 feet

PHID = ATAN(TAN(PHI)/FS) = 21.3 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

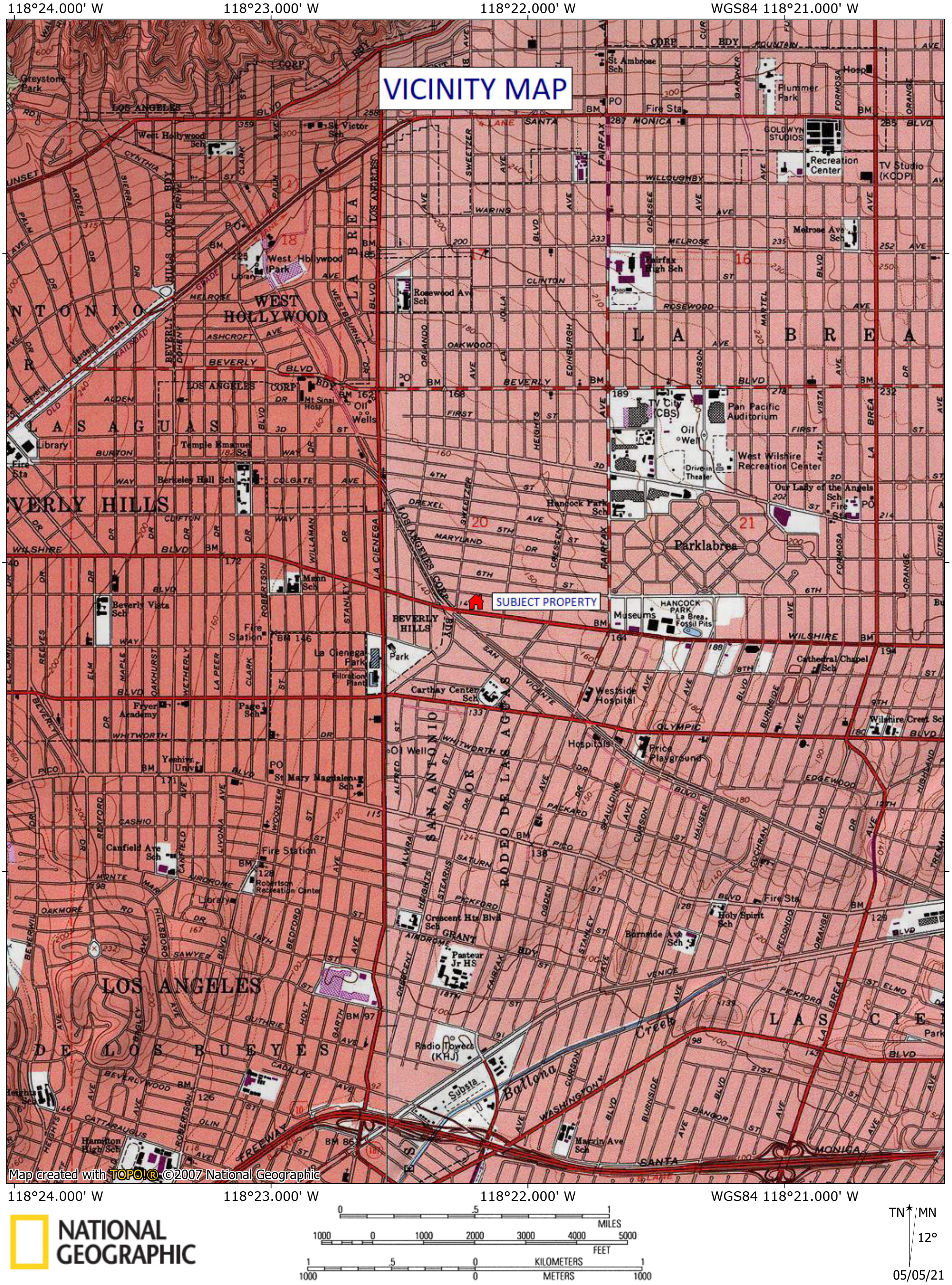
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	53 degrees
AREA OF TRIAL FAILURE WEDGE	454.0 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	61743.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1566 trials
LENGTH OF FAILURE PLANE	38.2 feet
DEPTH OF TENSION CRACK	4.5 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	23.0 feet
CALCULATED THRUST ON PILE	**23504.4 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**38.4 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**40.0 pcf**

THE CALCULATION INDICATES THAT THE PROPOSED SHORING SUPPORITING EXCAVATIONS UP TO 35 FEET MAY MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 40 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

TOPO! map printed on 05/05/21 from "IRVINE.tpo"

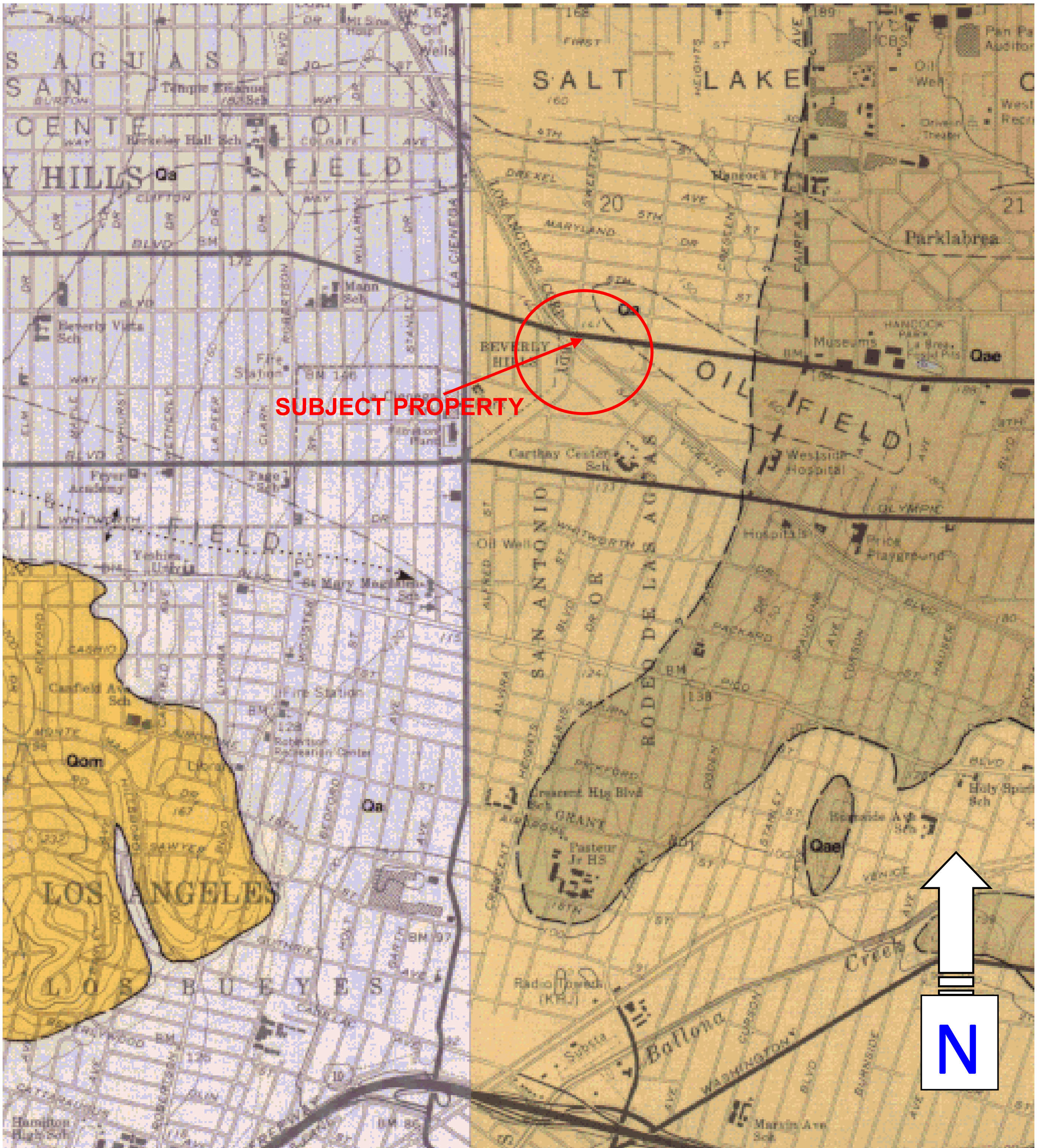


IRVINE GEOTECHNICAL Inc

REGIONAL GEOLOGIC MAP

IC:	**21035**	CONSULT:	**JAI**
CLIENT	**6435 WILSHIRE PROJECT**		
SCALE:	**1" = 2,000'**		

REFERENCE: Geologic Maps of the Santa Monica Mountains and Vicinity, CD Compilation T.W. Dibblee, 2001



LOG OF BORING

PROJECT	IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE	3/17/2021
LOG DATE	3/17/2021
LOGGED BY	RC
DRILL TYPE	HOLLOW-STEM
DIAMETER	8 INCHES

SURFACE ELEVATION	144.5 feet
DRILLING CONTRACTOR	Choice Drilling
SURFACE CONDITIONS	Driveway

BORING 1

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SC	144.5	0	**FILL:** 3" A/C over Sandy Clay, grey brown, moist, firm to stiff
							143.5	1	
							142.5	2	**ALLUVIUM:** Clayey Sand, orange-brown, moist to wet, dense
							141.5	3	
							140.5	4	
R	5	7/15/20	17.4	115.3	100	SC	139.5	5	Clayey Sand, yellow-brown, dense, very moist
							138.5	6	
							137.5	7	Silty Clay, orange-brown, very moist, porous
							136.5	8	
							135.5	9	
R	10	7/8/11	27.9	95.7	100	CL	134.5	10	
							133.5	11	
							132.5	12	
							131.5	13	Sand, medium to coarse, mottled yellow brown and orange brown, moist, dense
							130.5	14	
R	15	13/16/19	6.6	120.1	47	SW	129.5	15	
							128.5	16	Silty Sand, medium to coarse, orange brown, moist, medium dense
							127.5	17	
SPT	17.5	5/6/8				SM	126.5	18	
							125.5	19	
SPT	20	6/7/13				SM	124.5	20	

(continued next page...)

LOG OF BORING

PROJECT IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE 3/17/2021
LOG DATE 3/17/2021
LOGGED BY RC
DRILL TYPE HOLLOW-STEM
DIAMETER 8 INCHES

SURFACE ELEVATION 144.5 feet
DRILLING CONTRACTOR Choice Drilling
SURFACE CONDITIONS Driveway

BORING 1

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
SPT	20	6/7/13				SM	124.5	20	Silty Sand, yellow brown to grey brown, slightly moist, medium dense
							123.5	21	
							122.5	22	
SPT	22.5	8/9/10				SM	121.5	23	
							120.5	24	
SPT	25	4/7/9				SC	119.5	25	Clayey Sand with some Gravel, yellow olive green, moist, medium dense
							118.5	26	
							117.5	27	
SPT	27.5	7/11/13				SC	116.5	28	
							115.5	29	
SPT	30	7/9/12				SC	114.5	30	
							113.5	31	
							112.5	32	
SPT	32.5	5/7/9				SC	111.5	33	
							110.5	34	
SPT	35	9/12/16				SC	109.5	35	
							108.5	36	
							107.5	37	Clayey Sand with some Gravel, dark yellow grey, very moist to wet, dense
SPT	37.5	5/7/13				SC	106.5	38	
							105.5	39	groundwater
SPT	40	8/13/13				SM	104.5	40	

IRVINE GEOTECHNICAL Inc

LOG OF BORING

PROJECT IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE 3/17/2021
LOG DATE 3/17/2021
LOGGED BY RC
DRILL TYPE HOLLOW-STEM
DIAMETER 8 INCHES

SURFACE ELEVATION 144.5 feet
DRILLING CONTRACTOR Choice Drilling
SURFACE CONDITIONS Driveway

BORING 1

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
SPT	40	8/13/13				SM	104.5	40	Silty Sand, yellow grey, wet, dense
							103.5	41	
							102.5	42	
SPT	42.5	7/11/11				SM	101.5	43	
							100.5	44	
SPT	45	8/10/13				SM	99.5	45	
							98.5	46	
							97.5	47	
SPT	47.5	7/7/11				SM	96.5	48	
							95.5	49	Silty Clay, yellow dark grey, moist, stiff
SPT	50	5/7/8	25.9			CL	94.5	50	
							93.5	51	
							92.5	52	
SPT	52.5	4/5/5				CL	91.5	53	
							90.5	54	
SPT	55	5/6/7	26.8			CL	89.5	55	Sandy Clay, mottled yellow dark grey and light grey, moist, stiff
							88.5	56	
							87.5	57	
SPT	57.5	3/4/6				CL	86.5	58	
							85.5	59	Sandy Clay, yellow grey, moist, stiff
SPT	60	3/4/5	24.8			CL	84.5	60	

LOG OF BORING

IRVINE GEOTECHNICAL Inc

PROJECT	IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE	3/17/2021
LOG DATE	3/17/2021
LOGGED BY	RC
DRILL TYPE	HOLLOW-STEM
DIAMETER	8 INCHES

SURFACE ELEVATION	144.5 feet
DRILLING CONTRACTOR	Choice Drilling
SURFACE CONDITIONS	Driveway

BORING 1

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
SPT	60	3/4/5	24.8			CL	84.5	60	Sandy Clay, yellow grey, moist, stiff
							83.5	61	
							82.5	62	
SPT	62.5	3/5/6				CL	81.5	63	
							80.5	64	
SPT	65	4/5/6	24.4			CL	79.5	65	
							78.5	66	
							77.5	67	Sandy Clay, yellow grey, moist, stiff
SPT	67.5	4/7/12				CL	76.5	68	
							75.5	69	Silty Clay, yellow grey, moist, stiff
SPT	70	4/5/6	35.5			CL	74.5	70	
							73.5	71	
							72.5	72	Medium to coarse Sand, yellow grey, moist to wet, dense
SPT	72.5	25/28/50				SW	71.5	73	
							70.5	74	Silty Sand, yellow grey, moist, very dense
SPT	75	24/37/28				SW	69.5	75	
							68.5	76	
							67.5	77	
SPT	77.5	27/33/36				SW	66.5	78	
							65.5	79	
SPT	80	38/50-6"				SW	64.5	80	

LOG OF BORING

IRVINE GEOTECHNICAL Inc

PROJECT	IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE	3/17/2021
LOG DATE	3/17/2021
LOGGED BY	RC
DRILL TYPE	HOLLOW-STEM
DIAMETER	8 INCHES

SURFACE ELEVATION	144.5 feet
DRILLING CONTRACTOR	Choice Drilling
SURFACE CONDITIONS	Driveway

BORING 1

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
SPT	80	38/50-6"				SM	64.5	80	Silty Sand, yellow grey, moist, very dense
							63.5	81	
							62.5	82	
							61.5	83	
							60.5	84	
							59.5	85	
							58.5	86	
							57.5	87	
							56.5	88	
							55.5	89	
SPT	90	20/25/29				SM	54.5	90	
							53.5	91	
							52.5	92	
							51.5	93	Sandy Gravel, yellow brown, moist, very dense
						GW	50.5	94	
							49.5	95	
							48.5	96	Gravelly Sand, yellow grey, moist, very dense
							47.5	97	
							46.5	98	
							45.5	99	
SPT	100	31/50-3"				SW	44.5	100	**END BORING 1 @ 100':** Water @ 39 feet; Fill to 2.0 ft.

LOG OF BORING

IRVINE GEOTECHNICAL Inc

PROJECT IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE 3/18/2021
LOG DATE 3/18/2021
LOGGED BY RC
DRILL TYPE HOLLOW-STEM
DIAMETER 8 INCHES

SURFACE ELEVATION 146 feet
DRILLING CONTRACTOR Choice Drilling
SURFACE CONDITIONS Gently sloping Parking

BORING 2

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SC	146.0	0	**FILL:** 3" A/C over Sandy Clay, grey brown, moist, firm to stiff
							145.0	1	
						SC	144.0	2	**ALLUVIUM:** Clayey Sand, orange brown, moist, medium dense
							143.0	3	
							142.0	4	Clayey Sand, orange brown, moist, dense, porous
R	5	13/17/25	21.1	106.9	100	SC	141.0	5	
							140.0	6	Coarse Sand with Clay binder, orange brown, moist, medium dense
						SW	139.0	7	
							138.0	8	Sandy Clay, orange brown, moist, stiff
							137.0	9	
R	10	6/7/11	28.6	100.6	100	CL	136.0	10	
							135.0	11	
							134.0	12	
							133.0	13	
							132.0	14	Clayey Sand, orange brown, moist, medium dense
R	15	8/12/18	15.5	113.2	89	SC	131.0	15	
							130.0	16	
							129.0	17	
							128.0	18	
							127.0	19	Coarse Sand with Clay binder, yellow light brown, moist, medium dense
R	20	9/13/21	14.4	117.7	94	SW	126.0	20	

LOG OF BORING

IRVINE GEOTECHNICAL Inc

PROJECT	IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE	3/18/2021
LOG DATE	3/18/2021
LOGGED BY	RC
DRILL TYPE	HOLLOW-STEM
DIAMETER	8 INCHES
SURFACE ELEVATION	343 feet
DRILLING CONTRACTOR	Choice Drilling
SURFACE CONDITIONS	Gently sloping Parking

BORING 2

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
R	20	9/13/21	14.4	117.7	94	SW	323.0	20	Coarse Sand with Clay binder, yellow light brown, moist, dense
							322.0	21	
B	15-20						321.0	22	Sandy Clay, yellow light brown, moist, very stiff
							320.0	23	
							319.0	24	
R	25	9/15/21	31.2	94.8	100	CL	318.0	25	
							317.0	26	
							316.0	27	
							315.0	28	Clayey Sand, yellow olive green, slightly moist to moist, medium dense
							314.0	29	
R	30	11/16/18	15.8	114.6	95	SC	313.0	30	
							312.0	31	
							311.0	32	
							310.0	33	yellow dark grey, moist, dense
							309.0	34	
R	35	13/23/28	22.5	105.8	100	SC	308.0	35	
							307.0	36	
							306.0	37	
							305.0	38	
							304.0	39	Silty Sand, yellow dark grey, moist, dense
R	40	7/19/23	23.4	103.3	100	SM	303.0	40	

IRVINE GEOTECHNICAL Inc

LOG OF BORING

PROJECT	IC21035 - 6435 WILSHIRE PROJECT
DRILL DATE	3/18/2021
LOG DATE	3/18/2021
LOGGED BY	RC
DRILL TYPE	HOLLOW-STEM
DIAMETER	8 INCHES

SURFACE ELEVATION	343 feet
DRILLING CONTRACTOR	Choice Drilling
SURFACE CONDITIONS	Gently sloping Parking

BORING 2

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
R	40	7/29/23	23.4	103.3	100	SM	303.0	40	Silty Sand, yellow dark grey, moist, dense
							302.0	41	
							301.0	42	
							300.0	43	groundwater
							299.0	44	
R	45	7/15/21	28.0	97.4	100	SM	298.0	45	
							297.0	46	
							296.0	47	
							295.0	48	
							294.0	49	Coarse Sand, yellow dark grey, moist, very dense
R	50	15/34/50	16.9	117.0	100	SW	293.0	50	
							292.0	51	
							291.0	52	
							290.0	53	
							289.0	54	
R	55	34/50-3"	13.1	123.5	100	SW	288.0	55	
							287.0	56	
							286.0	57	
							285.0	58	
							284.0	59	
R	60	13/20/24	24.7	100.8	100	SW	283.0	60	**END BORING 2 @ 60':** Fill to 18 inches, Water at 43 feet.

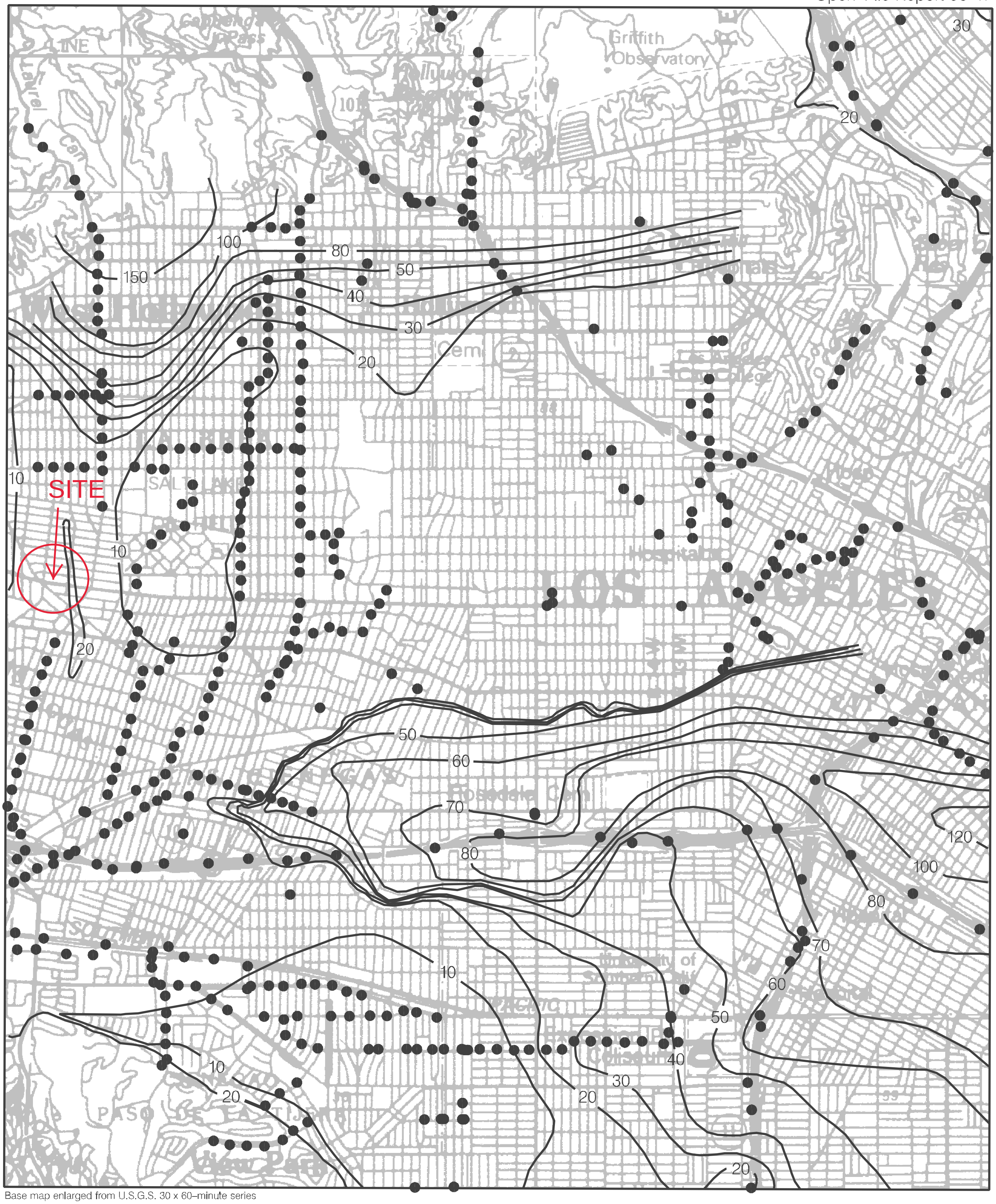


Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Hollywood Quadrangle.

● Borehole Site

30 Depth to ground water in feet

ONE MILE

SCALE

AVERAGE N60 FOR BORING 1

Energy Ratio C_E (Auto-hammer) 1.30

Boring	Depth (feet)	Lithology	Blow Count N80	Blow Count N60	Interval Thickness ft/N60
Boring 1					
1	5	Clayey Sand	22	28.4	0.176
1	10	Silty Clay	12	15.4	0.324
1	15	Sand	22	28.4	0.176
1	17.5	Silty Sand	14	18.2	0.137
1	20	Silty Sand	20	26.0	0.096
1	22.5	Silty Sand	19	24.7	0.101
1	25	Clayey Sand	16	20.8	0.120
1	27.7	Clayey Sand	24	31.2	0.080
1	30	Clayey Sand	21	27.3	0.092
1	32.5	Clayey Sand	16	20.8	0.120
1	35	Clayey Sand	28	36.4	0.069
1	37.5	Clayey Sand	20	26.0	0.096
1	40	Silty Sand	26	33.8	0.074
1	42.5	Silty Sand	22	28.6	0.087
1	45	Silty Sand	23	29.9	0.084
1	47.5	Silty Sand	18	23.4	0.107
1	50	Silty Clay	15	19.5	0.128
1	52.5	Silty Clay	10	13.0	0.192
1	55	Sandy Clay	13	16.9	0.148
1	57.5	Sandy Clay	10	13.0	0.192
1	60	Sandy Clay	9	11.7	0.214
1	62.5	Sandy Clay	11	14.3	0.175
1	65	Sandy Clay	11	14.3	0.175
1	67.5	Sandy Clay	19	24.7	0.101
1	70	Silty Clay	11	14.3	0.175
1	72.5	Sand	78	101.4	0.025
1	75	Silty Sand	65	84.5	0.030
1	77.5	Silty Sand	69	89.7	0.028
1	80	Silty Sand	88	114.4	0.022
1	90	Silty Sand	54	70.2	0.142
1	100	Gravelly Sand	81	105.3	0.095
				Sum	3.781

WEIGHTED AVERAGE N60 BLOW COUNT FOR UPPER 100 FEET (bl/ft) 26.5

LIQUEFACTION ANALYSIS USING SPT DATA

Use procedures established by T.L. Youd, et. al., 1996 NCEER-96-0022, SCEC SP117, CGS SP117A, 2008 Guidelines for Evaluating & Mitigating Seismic Hazards & I.M. Idriss & R.W. Boulanger, 2008, Soil Liquefaction During Earthquakes

Horizontal Ground Acceleration (% g)	0.975 PGA_M	Energy Ratio C_E (Auto-hammer)	1.30 *
Analyzed Groundwater Depth (feet)	18.0	Borehole Diameter C_B (6 - 8")	1.15
Average Wet Unit Weight (pcf)	120.0	Groundwater Depth in Boring (feet)	39.0
Design Magnitude Earthquake	6.36	$(N_1)_{60} = N_M C_N C_E C_B C_R C_S$	$(N_1)_{60CS} = K_S(N_1)_{60}$
Magnitude Scaling Factor (MSF)	1.5	C_S (for no sample liner) = $1+(N_1)_{60}/100$	
		* Energy Ratio certification provided by drilling company	

Boring	Depth (feet)	Lithology	Blow Count (N80)	Total Stress (tons/ft2)	Effective Stress (tons/ft2)	Fines Content FC(%)	Plasticity Index	C_R	C_N	C_S	rd	$(N_1)_{60}$	$(N_1)_{60cs}$	NCEER 1998 CSR/MSF	NCEER 1998 CRR	Liquefaction Safety Factor
Boring 1	5	Clayey Sand	22	0.300	0.300	0.0	0	0.75	1.47	1.30	0.99	47	47	0.4108	2.0000	No Water
1	10	Silty Clay	12	0.600	0.600	0.0	0	0.85	1.22	1.18	0.98	22	22	0.4060	0.2307	No Water
1	15	Sand	22	0.900	0.900	0.0	0	0.85	1.05	1.29	0.97	38	38	0.4011	2.0000	No Water
1	17.5	Silty Sand	14	1.050	1.050	59.7	0	0.85	0.98	1.17	0.96	20	30	0.3987	0.4557	No Water
1	20	Silty Sand	20	1.200	1.138	58.0	0	0.95	0.92	1.26	0.95	33	44	0.4180	2.0000	>1.3*
1	22.5	Silty Sand	19	1.350	1.210	52.7	0	0.95	0.86	1.23	0.95	29	39	0.4396	2.0000	>1.3*
1	25	Clayey Sand	16	1.500	1.282	48.9	0	0.95	0.81	1.19	0.94	22	31	0.4582	0.5824	1.27
1	27.7	Clayey Sand	24	1.662	1.359	44.0	0	0.95	0.77	1.26	0.94	33	45	0.4754	2.0000	>1.3*
1	30	Clayey Sand	21	1.800	1.426	43.9	0	0.95	0.73	1.22	0.93	27	37	0.4880	1.7439	3.57
1	32.5	Clayey Sand	16	1.950	1.498	0.0	0	0.95	0.70	1.16	0.91	18	18	0.4923	0.1877	0.38
1	35	Clayey Sand	28	2.100	1.570	0.0	0	1.00	0.67	1.28	0.89	36	36	0.4945	1.2865	2.60
1	37.5	Clayey Sand	20	2.250	1.642	33.6	0	1.00	0.64	1.19	0.87	23	32	0.4950	0.6254	1.26
1	40	Silty Sand	26	2.400	1.714	28.4	0	1.00	0.62	1.24	0.85	30	38	0.4940	2.0000	>1.3*
1	42.5	Silty Sand	22	2.550	1.786	29.7	0	1.00	0.60	1.20	0.83	24	32	0.4916	0.6573	1.34
1	45	Silty Sand	23	2.700	1.858	30.1	0	1.00	0.59	1.20	0.81	25	33	0.4881	0.7647	1.57
1	47.5	Silty Sand	18	2.850	1.930	22.4	0	1.00	0.58	1.16	0.79	18	24	0.4835	0.2643	0.55
1	50	Silty Clay	15	3.000	2.002	0.0	20	1.00	0.57	1.13	0.77	14	14	0.4779	0.1514	PI > 18
1	52.5	Silty Clay	10	3.150	2.074	0.0	20	1.00	0.56	1.10	0.75	9	9	0.4715	0.1126	PI > 18
1	55	Sandy Clay	13	3.300	2.146	0.0	31	1.00	0.55	1.11	0.73	12	12	0.4644	0.1312	PI > 18
1	57.5	Sandy Clay	10	3.450	2.218	0.0	31	1.00	0.54	1.10	0.71	9	9	0.4566	0.1104	PI > 18
1	60	Sandy Clay	9	3.600	2.290	0.0	21	1.00	0.53	1.10	0.69	8	8	0.4482	0.1037	PI > 18
1	62.5	Sandy Clay	11	3.750	2.362	0.0	21	1.00	0.52	1.10	0.67	9	9	0.4392	0.1142	PI > 18
1	65	Sandy Clay	11	3.900	2.434	0.0	23	1.00	0.51	1.10	0.65	9	9	0.4297	0.1131	PI > 18
1	67.5	Sandy Clay	19	4.050	2.506	0.0	23	1.00	0.50	1.14	0.62	16	16	0.4197	0.1682	PI > 18
1	70	Silty Clay	11	4.2	2.6	0.0	25	1.00	0.50	1.10	0.60	9	9	0.4093	0.1111	PI > 18
1	72.5	Sand	78	4.4	2.6	0.0	0	1.00	0.49	1.30	0.58	74	74	0.3985	2.0000	>1.3*
1	75	Silty Sand	65	4.5	2.7	0.0	0	1.00	0.48	1.30	0.56	61	61	0.3874	2.0000	>1.3*
1	77.5	Silty Sand	69	4.65	2.7936	0.0	0	1.00	0.47	1.30	0.54	63	63	0.3759	2.0000	>1.3*
1	80	Silty Sand	88	4.8	2.8656	0.0	0	1.00	0.47	1.30	0.52	80	80	0.3641	2.0000	>1.3*

LIQUEFACTION ANALYSIS USING SPT DATA

Use procedures established by T.L. Youd, et. al., 1996 NCEER-96-0022, SCEC SP117, CGS SP117A, 2008 Guidelines for Evaluating & Mitigating Seismic Hazards & I.M. Idriss & R.W. Boulanger, 2008, Soil Liquefaction During Earthquakes

Horizontal Ground Acceleration (% g)	0.975 PGA_M	Energy Ratio C_E (Auto-hammer)	1.30 *
Analyzed Groundwater Depth (feet)	18.0	Borehole Diameter C_B (6 - 8")	1.15
Average Wet Unit Weight (pcf)	120.0	Groundwater Depth in Boring (feet)	39.0
Design Magnitude Earthquake	6.36	$(N_1)_{60} = N_M C_N C_E C_B C_R C_S$	$(N_1)_{60CS} = K_S(N_1)_{60}$
Magnitude Scaling Factor (MSF)	1.5	C_S (for no sample liner) = $1+(N_1)_{60}/100$	
		* Energy Ratio certification provided by drilling company	

Boring	Depth (feet)	Lithology	Blow Count (N80)	Total Stress (tons/ft2)	Effective Stress (tons/ft2)	Fines Content FC(%)	Plasticity Index	C_R	C_N	C_S	rd	$(N_1)_{60}$	$(N_1)_{60cs}$	NCEER 1998 CSR/MSF	NCEER 1998 CRR	Liquefaction Safety Factor
1	90	Silty Sand	54	5.4	3.1536	0.0	0	1.00	0.44	1.30	0.44	46	46	0.3143	2.0000	>1.3*
1	100	Gravelly Sand	81	6	3.4416	0.0	0	1.00	0.42	1.30	0.36	65	65	0.2610	2.0000	>1.3*

DYNAMIC SETTLEMENT ANALYSIS USING SPT DATA
"SATURATED SAND SETTLEMENT"

Procedure by Tokimatsu & Seed, 1987, Modified for Spreadsheet by Idriss & Boulanger, 2008

Horizontal Ground Acceleration (% g)	0.975 PGA_M	Energy Ratio C_E (Auto-hammer)	1.30
Analyzed Groundwater Depth (feet)	18.0	Borehole Diameter C_B (6 - 8")	1.15
Average Wet Unit Weight (pcf)	120	Groundwater Depth in Boring (feet)	39.0
Design Magnitude Earthquake	6.36	Foundation Depth (feet)	1.5
Magnitude Scaling Factor (MSF)	1.52		

Boring	Depth (feet)	Interval Thickness (feet)	Lithology	Blow Count (N80)	Total Stress (tons/ft2)	Effective Stress (tons/ft2)	Blow Count N_{60}	SPT $(N_1)_{60}$ (blow/ft)	Limiting Shear Strain (%)	Max. Shear Strain (%)	Liquefaction Safety Factor	Vertical Strain (%)	Calculated Settlement (inches)	Cumulative Settlement (inches)
Boring 1	5	2.5	Clayey Sand	22	0.300	0.300	28.4	47	0.0000	0.0000	No Water	0.0000	0.00	0.00
1	10	2.5	Silty Clay	12	0.600	0.600	15.4	22	0.0000	0.0000	No Water	0.0000	0.00	0.00
1	15	2.5	Sand	22	0.900	0.900	28.4	38	0.0000	0.0000	No Water	0.0000	0.00	0.00
1	17.5	2.5	Silty Sand	14	1.050	1.050	18.2	30	0.0000	0.0000	No Water	0.0000	0.00	0.00
1	20	2.5	Silty Sand	20	1.200	1.200	26.0	44	0.0000	0.0000	>1.3*	0.0000	0.00	0.00
1	22.5	2.5	Silty Sand	19	1.350	1.350	24.7	39	0.0000	0.0000	>1.3*	0.0000	0.00	0.00
1	25	2.5	Clayey Sand	16	1.500	1.500	20.8	31	0.0385	0.0000	1.2712	0.0000	0.00	0.00
1	27.7	2.5	Clayey Sand	24	1.662	1.662	31.2	45	0.0000	0.0000	>1.3*	0.0000	0.00	0.00
1	30	2.5	Clayey Sand	21	1.800	1.800	27.3	37	0.0156	0.0000	3.5732	0.0000	0.00	0.00
1	32.5	2.5	Clayey Sand	16	1.950	1.950	20.8	18	0.1902	0.1902	0.3813	0.0247	0.74	0.74
1	35	2.5	Clayey Sand	28	2.100	2.100	36.4	36	0.0196	0.0000	2.6014	0.0000	0.00	0.74
1	37.5	2.5	Clayey Sand	20	2.250	2.250	26.0	32	0.0360	0.0000	1.2634	0.0000	0.00	0.74
1	40	2.5	Silty Sand	26	2.400	2.369	33.8	38	0.0000	0.0000	>1.3*	0.0000	0.00	0.74
1	42.5	2.5	Silty Sand	22	2.550	2.441	28.6	32	0.0343	0.0000	1.3370	0.0000	0.00	0.74
1	45	2.5	Silty Sand	23	2.700	2.513	29.9	33	0.0299	0.0000	1.5667	0.0000	0.00	0.74
1	47.5	2.5	Silty Sand	18	2.850	2.585	23.4	24	0.1025	0.1025	0.5466	0.0198	0.59	1.33
1	50	2.5	Silty Clay	15	3.000	2.657	19.5	14	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	52.5	2.5	Silty Clay	10	3.150	2.729	13.0	9	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	55	2.5	Sandy Clay	13	3.300	2.801	16.9	12	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	57.5	2.5	Sandy Clay	10	3.450	2.873	13.0	9	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	60	2.5	Sandy Clay	9	3.600	2.945	11.7	8	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	62.5	2.5	Sandy Clay	11	3.750	3.017	14.3	9	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	65	2.5	Sandy Clay	11	3.900	3.089	14.3	9	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	67.5	2.5	Sandy Clay	19	4.050	3.161	24.7	16	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	70	2.5	Silty Clay	11	4.200	3.233	14.3	9	0.0000	0.0000	PI > 18	0.0000	0.00	1.33
1	72.5	2.5	Sand	78	4.350	3.305	101.4	74	0.0000	0.0000	>1.3*	0.0000	0.00	1.33
1	75	2.5	Silty Sand	65	4.500	3.377	84.5	61	0.0000	0.0000	>1.3*	0.0000	0.00	1.33
1	77.5	2.5	Silty Sand	69	4.650	3.449	89.7	63	0.0000	0.0000	>1.3*	0.0000	0.00	1.33
1	80	2.5	Silty Sand	88	4.800	3.521	114.4	80	0.0000	0.0000	>1.3*	0.0000	0.00	1.33
1	90	2.5	Silty Sand	54	5.400	3.809	70.2	46	0.0000	0.0000	>1.3*	0.0000	0.00	1.33
1	100	2.5	Gravelly Sand	81	6.000	4.097	105.3	65	0.0000	0.0000	>1.3*	0.0000	0.00	1.33

IRVINE GEOTECHNICAL Inc

SITE PLAN

PROJECT: IC21035 BLACK EQUITIES

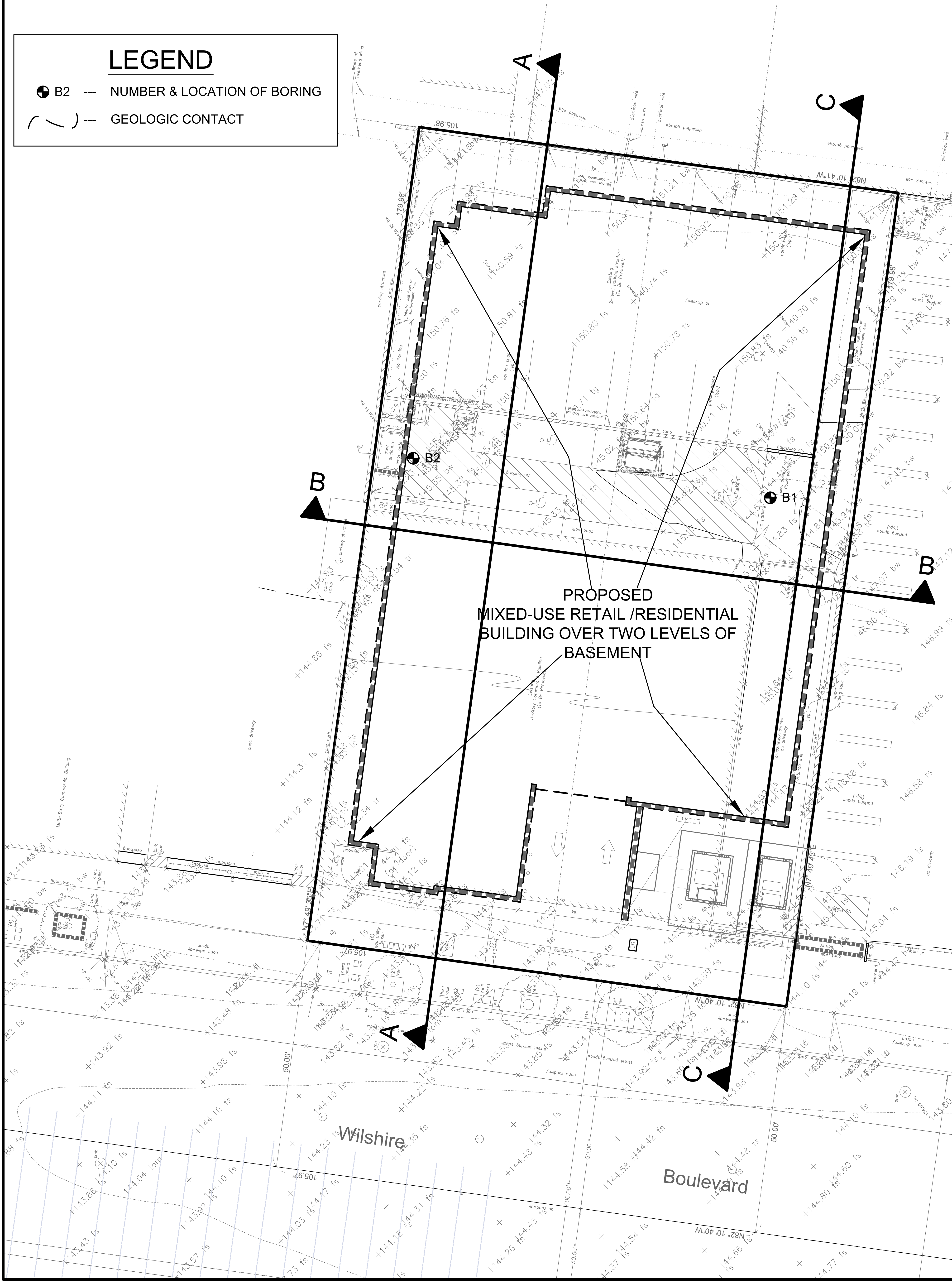
CONSULTANT: JAI

SCALE: 1" = 20'

LEGEND

B2 --- NUMBER & LOCATION OF BORING

--- GEOLOGIC CONTACT

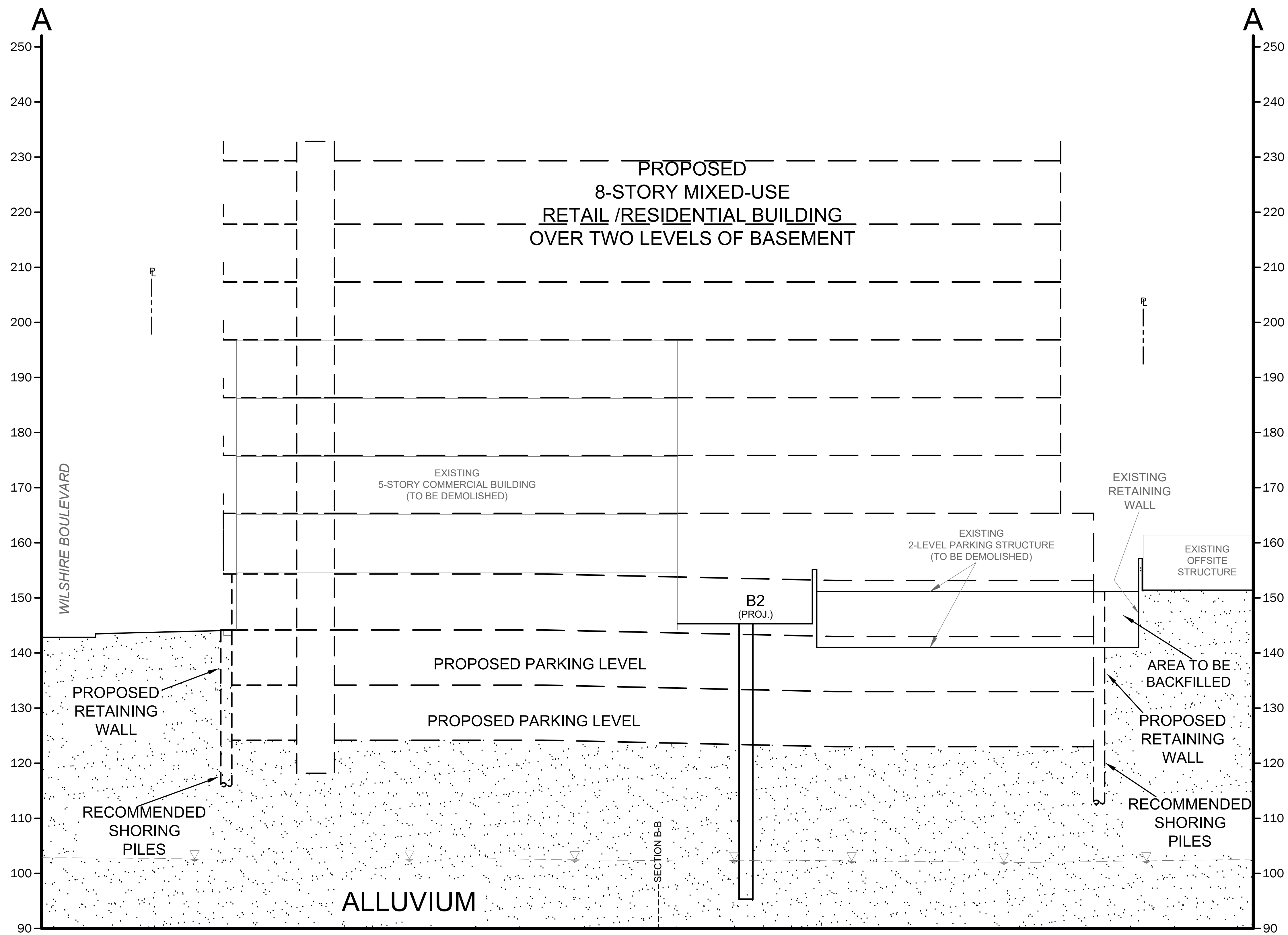


IRVINE GEOTECHNICAL Inc

SECTION A - A

PROJECT: IC21035 BLACK EQUITIES

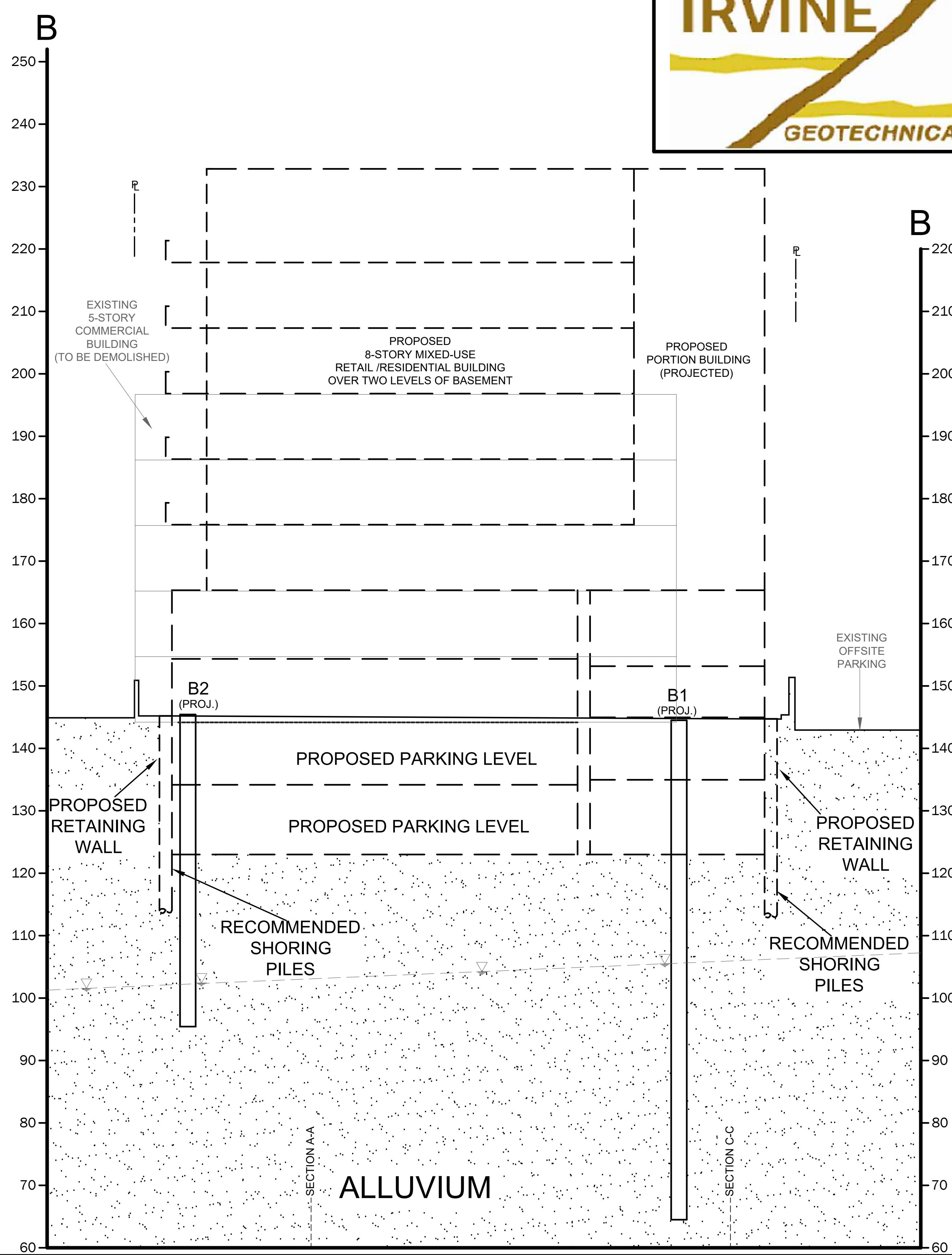
CONSULTANT: JAI | SCALE: 1" = 20'



IRVINE GEOTECHNICAL Inc

SECTION B - B

PROJECT: IC21035 BLACK EQUITIES	
CONSULTANT: JAI	SCALE: 1" = 30'



IRVINE GEOTECHNICAL Inc

SECTION C - C

PROJECT: IC21035 BLACK EQUITIES

CONSULTANT: JAI | SCALE: 1" = 30'