GEOLOGIC AND SOILS ENGINEERING EXPLORATION

PROPOSED RESIDENCE

LOT 100, ARB.1, LE MOYNE TERRACES TRACT NO.2

NORTH OF 481 FRONTENAC AVENUE

LOS ANGELES, CALIFORNIA

FOR OWENSBY RESIDENCE

IRVINE GEOTECHNICAL, INC. PROJECT NUMBER IC 21198-I

DECEMBER 26, 2021

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INTRODUCTION

This report has been prepared per our agreement and summarizes findings of Irvine Geotechnical's geologic and soils engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution, 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 Negative Studio 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 Geologic Map and cross section. Conditions affecting portions of the property outside the area explored, are beyond the scope of this report.

Exploration was conducted on November 12, 2021, with the aid of hand labor. It included excavating five test pits to a maximum depth of 7 feet. Samples of the earth materials were obtained and delivered to the soils engineering laboratory of Soil Labworks, LLC for testing and analysis. Downhole observation of the earth materials was performed by the engineering geologist.

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 Geologic Map and cross section, and performing engineering analysis. Earth materials exposed in the test pits are described on the enclosed Log of Test Pits. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project, surface geologic conditions, and the location of the test pits are shown on the Geologic Map. Subsurface distribution of the earth materials projected geologic structure, and the proposed project are shown on Section A, which also forms the basis for the enclosed stability calculations.

PROPOSED PROJECT

Information concerning the proposed project was provided by the client. The preliminary plans prepared by Negative Design were a guide for exploring the site and preparing this report. It is proposed to develop the vacant lot with a single-family residence with a pool. The structure will be two-story, with the lower level partially tucked in the descending slope below Frontenac Avenue. Retaining walls up to 18 feet high are planned to support excavations for the basement level. Grading will include improving site conditions, exporting soils from the basement, backfilling retaining walls, and improving Frontenac Avenue at the residence. The pool will be elevated above the structural deck at the upper level of the structure.

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

SITE DESCRIPTION

The subject property consists of a partially graded and vacant hillside lot, on the southeast flank of Mount Washington, in the City of Los Angeles, California. It is located on the east side of

Frontenac Avenue, just north of the intersection of Frontenac Avenue with Canyon Vista Drive, and ½ mile northwest of the intersection of Avenue 43 and Figueroa Street. The eastern property line fronts Avenue 44. The site and the adjoining lots are vacant. The surrounding area is developed with a few single-family residences.

Topographically, the site is situated on the east flank of a south-southeast trending ridge. Mount Washington Drive and Canyon View Drive are located at the crest of the broad ridge and Avenue 44 is within a southeast-draining secondary canyon. Past grading on the site has been associated with grading of Frontenac Avenue and Avenue 44. Fill was spilled over the shoulder of the road along the western property line and cuts were made into the toe of slope near the eastern property line. Physical relief across the property is about 45 feet with slope gradients ranging 0.9:1 to 3:1.

Vegetation on the site consists of trees and a thick assemblage of grasses and native ground cover. Surface drainage is by sheet flow runoff down the contours of the land toward the east.

GROUNDWATER

Groundwater was not encountered during exploration. Seasonal fluctuations in groundwater levels may occur due to variations in climate, irrigation, and other factors not evident at the time of the exploration. Fluctuations in groundwater levels may also occur across the site.

EARTH MATERIALS

Fill

Fill, associated with previous site grading, underlies portions of the site to a maximum observed thickness of 3½ feet in the vicinity of Test Pit 5. The fill consists of silty sand and clayey sand that is grey brown to yellow grey, slightly moist, and medium dense.

Soil

Natural residual soils were encountered in all of the test pit. The soil consists of clayey sand to silty sand that is grey brown, slightly moist, medium dense and contains roots. The soil layer ranges from 1 to 3½ feet thick.

Bedrock

Bedrock underlying the site and encountered in the test pits consist of sandstone and siltstone of the Puente Formation as mapped by D. L Lamar, 1970, (*Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California,* USGS Special Report 101). The bedrock is yellow brown to orange brown, moderately hard to hard, massive to bedded, and moderately weathered.

GEOLOGIC STRUCTURE

The bedrock described is common to this area of Mount Washington and the geologic structure is consistent with regional trends. The sandstone bedrock is massive to thickly bedded and lacks significant structural planes. Bedding planes mapped strike east-west to northeast and dip shallowly toward the south.

The massive nature of the sandstone and geologic structure of the bedrock is favorably oriented for stability of the site and proposed project. Recommendations to eliminate or support any unfavorably oriented bedding are presented in the CONCLUSIONS AND RECOMMENDATIONS section of this report.

GENERAL SEISMIC CONSIDERATIONS

Southern California is 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 can cause 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 following table lists the applicable seismic coefficients for the 2020 Los Angeles Building Code.

SEISMIC COEFFICIENTS (2020 Los Angeles Building Code)		
Latitude = 34.09991°N Longitude = 118.21329°W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class Chapter 20 - ASCE 7	Bedrock - C	
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)	$S_{s(g)}$ = 2.105	$S_{1(g)}$ = 0.745
Site Coefficients from Tables 1613.2.3 (1) and 1613.2.3 (2)	F_A = 1.2	F_V = 1.4
Spectral Response Accelerations from Equations 16-36 and 16-37	$S_{MS(g)}$ = 2.527	$S_{M1(g)}$ = 1.043
Design Accelerations	$S_{DS(g)}$ = 1.684	$S_{D1(g)}$ = 0.694

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 private 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. However, secondary effects such as surface rupture, lurching, liquefaction, consolidation, ridge shattering, and landsliding should not occur at the subject property.

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, Los Angeles Quadrangle. Seismic hazards within the Los Angeles Quadrangle were evaluated by the CGS in their report, "*Seismic Hazard Zone Report for the Los Angeles 7.5-minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 029."* According to the Seismic Hazard Zones Map, the subject property is not within an area that has been subject to or may be subject to liquefaction or earthquake induced ground deformation.

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_M for this site is 1.096g. 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 0.511g and 6.9, 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 1.009g and 6.9, respectively. The modal distance to the ground motion source is 5.02 km. It is our understanding that the fundamental period of the proposed building will be less than 0.5 seconds.

SLOPE STABILITY

Gross Stability

Slopes affecting the subject property include a 45-foot-high slope between Frontenac Avenue and Avenue 44. Section A was prepared to show the highest and steepest topography on the site. The gross stability of the slope shown in Section A was calculated using a computerized version of Simplified Bishop's method (SLIDE2 Version 9.020 developed by ROCSCIENCE, Inc.).

The analysis shows that the subject property and existing slopes are grossly stable with a factor of safety more than 1.5 under static conditions. The calculations use the shear tests of samples believed to represent the weakest bedrock encountered during exploration. The cross section used is the most critical for the slopes analyzed.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon five test pits, field geologic mapping, research of available records, consultation, years of experience observing

comparable 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 geologic and soils 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 bedrock, which was encountered at depths of 2 to 5 feet. The existing fill and soil are not recommended foundation or slab support. Conventional foundations may be utilized to support portions of the proposed structure tucked into the slope. Deepened foundations are recommended to support portions of the proposed structure located adjacent to the slope.

Geotechnical Issues

Geotechnical issues affecting the site include temporary excavations adjacent to property lines required to construct the basement walls. Shoring will be required to construct portions of the project.

A small portion of the road cut along Avenue 44 is steeper than 1:1. It is recommended that any existing road cuts steeper than 1:1 be trimmed to 1:1. A typical trim is shown on Section A.

SITE PREPARATION

Surficial materials consisting of fill and soil are present on the site. Remedial grading is recommended to improve site conditions for improving the road in front of the property.

General Grading Specifications

The following guidelines may be used in preparation of the grading 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 soil. The exposed excavated area should be observed by the soils engineer or geologist 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.

Cut/Trim Slopes

Cut/trim slopes in bedrock along Avenue 44 may be created at a 1:1 gradient. Cut trim slopes in fill and soil may be created at a 2:1 gradient.

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 (%)
Bedrock	90.9	80	90.9	13.6	0.0
Fill/Soil	78.4	75	99.0	4.5	20.8

Excavation Characteristics

The test pits did encounter hard, cemented bedrock. Excavation difficulty is a function of the degree of weathering and amount of fracturing within the bedrock. The bedrock becomes harder and more difficult to excavate with increasing depth. Hard cemented layers are also known to occur at random locations and depths and may be encountered during foundation excavation. Should a hard cemented layer be encountered, coring or the use of jackhammers may be necessary.

SWIMMING POOL

The proposed swimming pool will be supported by the building superstructure and be designed as free-standing.

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.

Spread Footings

Continuous and/or pad footings may be used to support the proposed structures provided they are founded in bedrock. 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)
Bedrock	18	3,000	0.35	300	5,000

Increases in the bearing value are allowable at a rate of 600 pounds per square foot for each additional foot of footing width or depth to a maximum of 5,000 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.

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.

Deepened Foundations - Friction Piles

Drilled, cast-in-place concrete friction piles are recommended to support portions of the proposed structures located over thick fill and soil over on slopes. Piles should be a minimum of 24 inches in diameter and a minimum of 8 feet into bedrock. Piles may be assumed fixed at 3 feet into bedrock. For the vertical forces, the piles may be designed for a skin friction of 800 pounds per square foot for that portion of pile in contact with the bedrock. 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.

Lateral Design - Isolated Piles and Loading Normal to Row of Piles

The existing fill and soil on the site are subject to downhill creep. Pile shafts are subject to lateral loads due to the creep forces. Pile shafts should be designed for a lateral load of 1,000 pounds per linear foot for each foot of shaft exposed to the existing fill and soil.

Resistance to lateral loading may be provided by passive earth pressure within the bedrock. 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 5,000 pounds per square foot. For design of isolated piles and the direction of loading does not coincide with a row of 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.

All piles should be designed to resist lateral forces in conformance with Section 1810.2 of the Building Code. This may require tying the piles in two horizontal directions with grade beams or structural slabs.

Lateral Design - Pile Groups or Loading Parallel to Row of Piles

In conformance with Section 1810.2.5 of the Building Code, piles in groups may be subject to reduced lateral and axial capacities to account for group effects.

For piles in groups, the following reduction factors may be assumed for group action. The lateral resistance should be reduced by the factor.

LATERAL RESISTANCE REDUCTION FACTORS FOR PILE GROUPS			
Pile Spacing in Direction of Loading D = Pile Diameter	Row 1 Leading Row	Row 2	Row 3 & above (Trailing Row)
7D	1.00	1.00	0.90
6.5D	1.00	0.97	0.87
5D	0.86	0.83	0.77
3D	0.75	0.55	0.40

* Recommended by Caltrans GeoResearch Group

For piles in groups, the following reduction factors may be assumed for group action. The skin friction axial capacity should be reduced by the group efficiency factor.

AXIAL CAPACITY REDUCTION FACTORS FOR PILE GROUPS	
Pile Spacing D = Pile Diameter	Group Efficiency
4.5D	0.80
3D	0.70
2D	0.55
1.5D	0.30

* Figure 3 NAVFAC DM 702-206

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A settlement of ¼ to ½ inch may be anticipated. Differential settlement should not exceed ½ inch.

Foundation Setback

The Building Code requires that foundations be a sufficient depth to provide horizontal setback from a descending slope steeper than 3:1. The required setback is 1/3 the height of the slope with a minimum of five feet and a maximum of 40 feet measured horizontally from the base of the foundation to the slope face. On the subject property, the slope descends below the building area 27 to 45 feet. The code required clearance ranges from 15 to 9 feet.

RETAINING WALLS

General Design - Static Loading

Cantilevered retaining walls up to 18 feet high that support bedrock and approved retaining wall backfill, may be designed for an equivalent fluid pressure of 45 pounds per cubic foot. South-facing retaining walls up to 18 feet high that support bedding may be designed for an equivalent fluid pressure of 55 pounds per cubic foot per the enclosed calculations. 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 67 pcf.

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 1.096g. The horizontal coefficient of seismic increment (K_E) was assumed to be a(PGA_M) = 0.365g.

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 0.365g.

The recommended static and seismic forces for 12-foot-high 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	18ft^2 * 45 pcf /2= 7.290 kips	7.109 kips	0 pcf
Level**	18ft^2 * 55 pcf /2= 8.910 kips	8.193 kips	0 pcf
Restrained	18ft^2 * 67 pcf /2= 10.854 kips	7.109 kips	0 pcf

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

** Bedding Plane Condition

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 retaining walls. The excavations could be up to 18 feet in height and will expose fill and soil over bedrock. The fill should be trimmed to 1:1 for wall excavations. Where not surcharged by existing footings or structures, the bedrock can maintain vertical excavations up to 8 feet per the enclosed calculations. Where vertical excavations in the bedrock exceed 8 feet in height, the upper portion should be trimmed to 1:1 (45 degrees).

South-facing excavations into bedrock, may unsupport bedding in the down-dip direction. South-facing excavations should be trimmed to 2:1 or supported by shoring.

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 property lines will require the use of temporary shoring.

Shoring

Temporary shoring supporting favorable bedrock should be designed for an equivalent fluid pressure of 30 pounds per cubic foot. Shoring that supports south-facing excavations and bedding planes should be designed for an EFP of 35 pcf per the enclosed calculations. Shoring that is integrated into the permanent retaining walls should be designed for earth pressures conforming to the RETAINING WALL section of this report.

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 8 feet into bedrock below the base of the excavation. Piles may be assumed fixed 3 feet into bedrock below the base of the excavation. For the vertical forces, piles may be designed for a skin friction of 800 pounds per square foot for that portion of pile in contact with the bedrock. 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 bedrock 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 5,000 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.

A representative of the geotechnical engineer or geologist 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 corrosive to ferrous metals.

FLOOR SLABS, CONCRETE DECKING AND PAVING

Floor slabs and concrete decking should be cast over bedrock or approved compacted fill. In areas of existing fill and soil, the ground should be prepared, and the fill placed in conformance with the SITE PREPARATION section of this report.

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 if 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 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 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.

The paving section should be cast over bedrock or an approved compacted fill cap. In areas of existing fill and soil, the ground should be prepared, and the fill placed in conformance with the SITE PREPARATION section of this report.

Trench backfill below paving, should be compacted to 90 percent of the maximum dry density. Irrigation water should be prevented from migrating under paving. Base course should be compacted to at least 95 percent of the maximum dry density. The following table shows the recommended pavement sections:

Service	Pavement Thickness (Inches)	Base Course (Inches)
Light Passenger Cars	3	4
Street	3	8

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 location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. 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

Because the site is within a designated hillside area and due to nearby slopes, 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, several 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 geologist/engineer will recommend treatment.

Pre-construction meeting	Advised
Temporary excavations	Required
Shoring pile 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 Irvine Geotechnical, and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein and shown on the enclosed cross section 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 engineering geologist and geotechnical engineer during 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-5)
Vicinity Map
Regional Geologic Map
Log of Test Pits (3 Pages)
Calculation Sheets (15)
Section A
Local Topo Map
Geologic Map

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 December 7, 2021, and the results are reasonable for this area of the Mt. Washington. Irvine Geotechnical and the undersigned engineer concurs with the findings of Soil Labworks, LLC and accepts professional responsibility for utilizing the data.

SL21.3819
December 7, 2021

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

Subject: Laboratory Testing

Site: N of 486 W. Avenue 44
Los Angeles, California

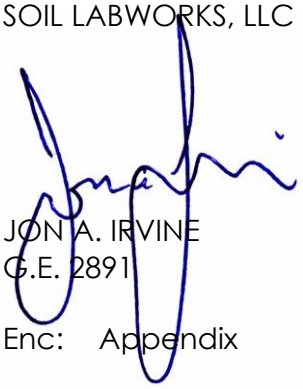
Job: IRVINE/KAVASCH

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



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 - Hand Labor

Samples of earth materials were obtained by driving a thin-walled steel sampler with successive blows of a drop hammer. The earth material was retained in brass rings of 2.416 inches inside diameter and 1.00 inch height. The samples were stored in closefitting, water-tight containers for transportation to the laboratory.

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)
TP1	1	Soil	76.9	9.4	22
TP1	3	Bedrock	97.5	11.2	43
TP2	3	Bedrock	101.8	8.7	37
TP3	2½	Soil	79.8	12.7	31
TP3	7	Bedrock	83.8	22.8	62
TP4	1	Fill	76.1	7.0	16
TP4	2½	Soil	80.7	10.4	26
TP4	6	Bedrock	80.4	22.5	57

Compaction Character

Compaction tests were performed on bulk samples of the earth materials in accordance with ASTM D1557-12(2021). The results of the tests are provided on the table below and on the "Moisture-Density Relationship", A-Plates. The specific gravity of the soil/bedrock was estimated from the compaction curves.

Compaction Character (continued)

Test Pit/Boring No.	Sample Depth (Feet)	Soil Type	Maximum Dry Density (pcf)	Optimum Moisture Content (Percent)
TP3	0-6	Soil/Bedrock	107.4	16.3

Shear Strength

The peak and ultimate shear strengths of the fill, soil and bedrock 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 residual shear strength of the soil and bedrock was determined by repeatedly shearing a sample under varying confining pressures in the direct shear machine. The rate of deformation for the last test at each confining pressure was 0.005 inches per minute. 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)
TP4	1	76.1	40.2
TP3	7	83.8	41.7
TP3*	0-6	96.7	29.7
TP1**	1	76.9	39.6
TP4	6	80.4	36.7

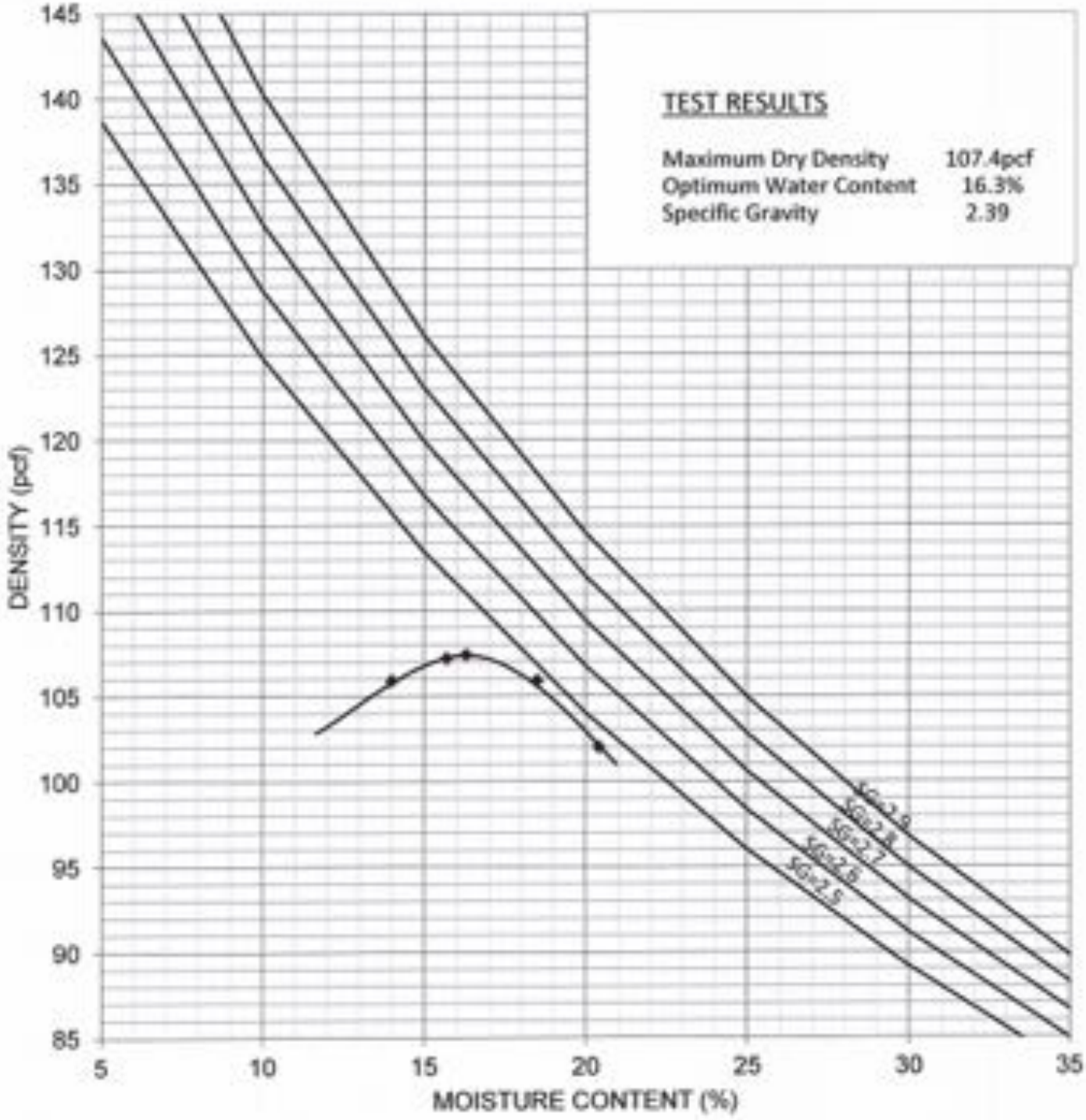
* Sample remolded to 90 % of the laboratory maximum density.
** Sample repeatedly sheared to determine residual strength.

SOIL LABWORKS LLC

MOISTURE-DENSITY RELATIONSHIP A-1

JN: SL21.3819 CONSULTANT: JAI
CLIENT: IRVINE/Kavasch-n of 486 W Avenue 44
TP3 @ 0-6'
EARTH MATERIAL: SOIL/BEDROCK

NOTE: ASTM Test Method D-1557-12

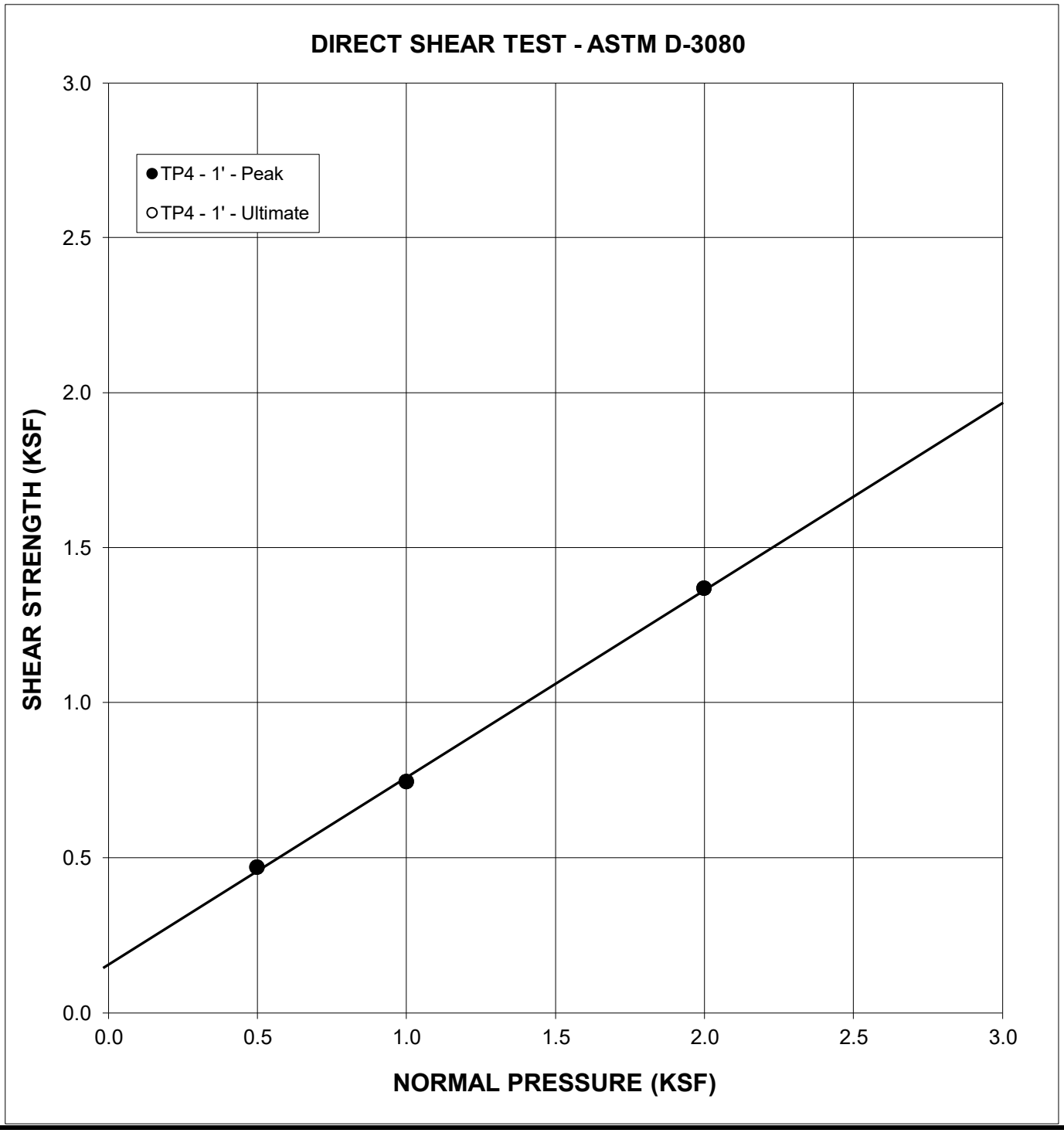


SHEAR DIAGRAM B-1

JN: **SL21.3819** CONSULTANT **JAI**
CLIENT: **Irvine/Kavasch-N of 486 W Avenue 44**

EARTH MATERIAL: **FILL**

	PEAK	ULTIMATE			
Phi Angle	**31**	**31**	**degrees**	**Average Moisture Content**	**40.2%**
Cohesion	**160**	**160**	**psf**	**Average Dry Density (pcf)**	**76.1**
				Percent Saturation	**95.7%**

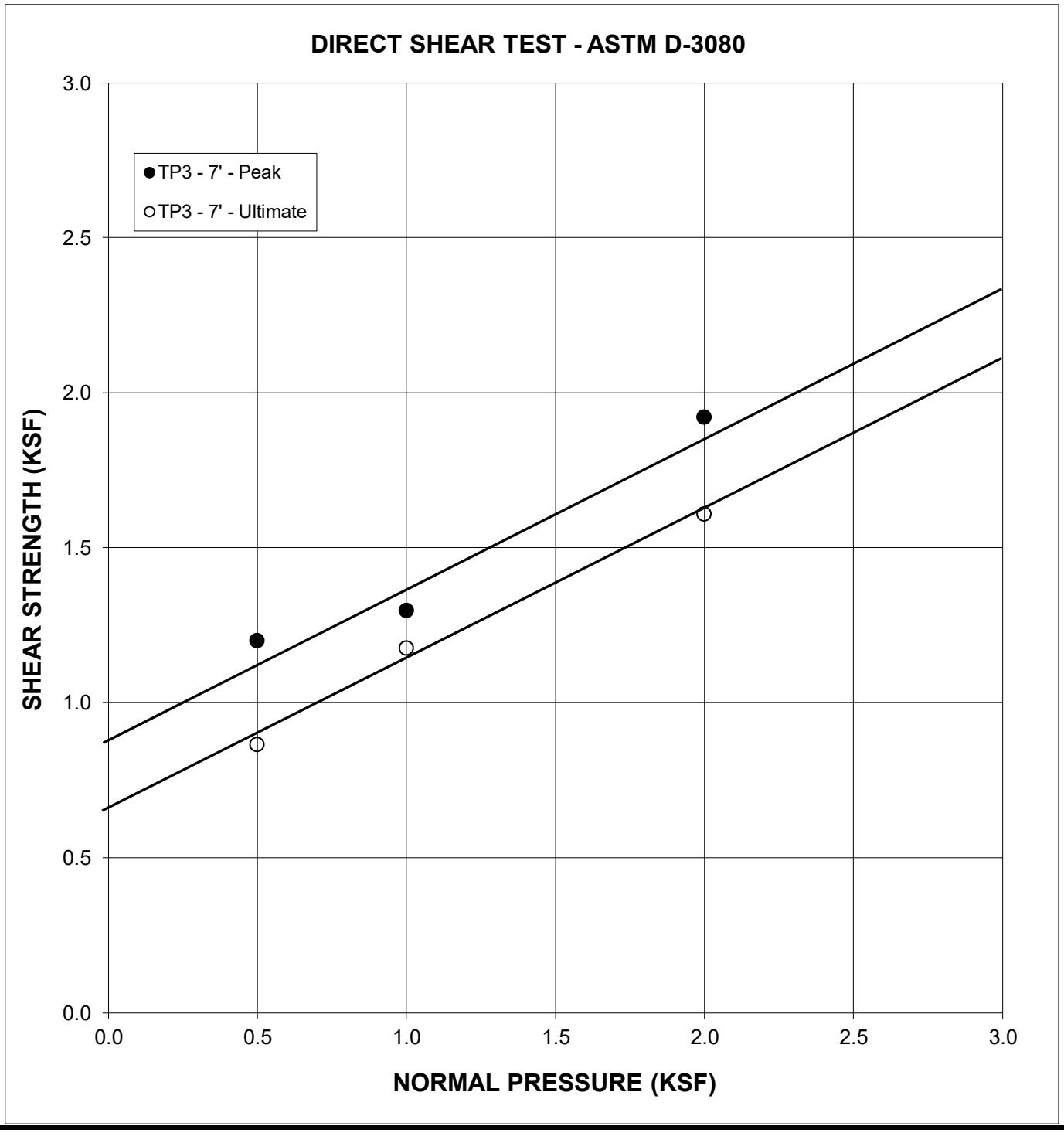


SHEAR DIAGRAM B-2

JN: **SL21.3819** CONSULTANT **JAI**
CLIENT: **Irvine/Kavasch-N of 486 W Avenue 44**

EARTH MATERIAL: **BEDROCK**

	PEAK	ULTIMATE			
Phi Angle	**26**	**26**	**degrees**	**Average Moisture Content**	**41.7%**
Cohesion	**880**	**655**	**psf**	**Average Dry Density (pcf)**	**83.8**
				Percent Saturation	**100.0%**



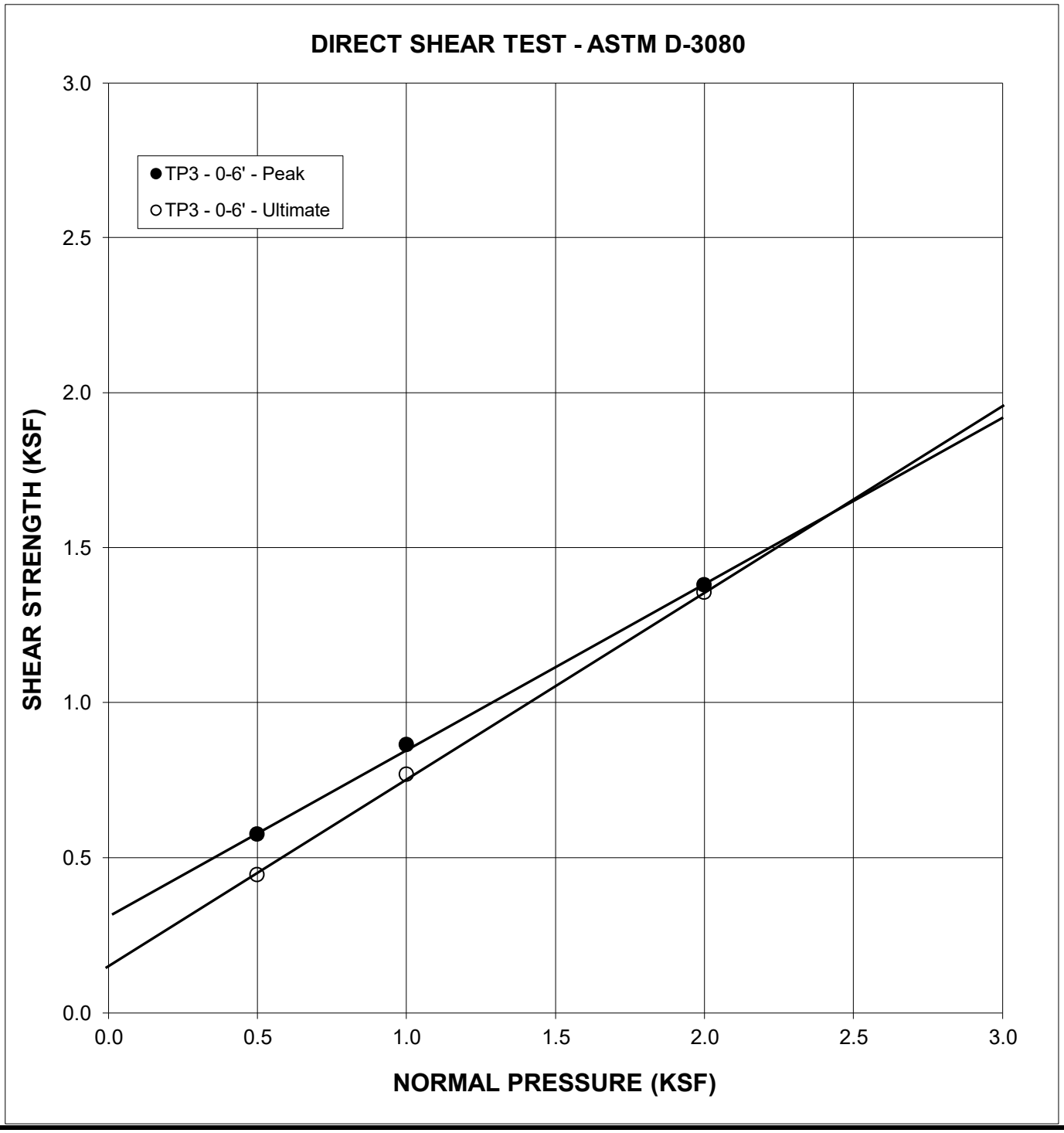
SHEAR DIAGRAM B-3

JN: **SL21.3819** CONSULTANT **JAI**
CLIENT: **Irvine/Kavasch-N of 486 W Avenue 44**

EARTH MATERIAL: **SOIL/BEDROCK**

Sample remolded to 90 % of the laboratory maximum density

	PEAK	ULTIMATE			
Phi Angle	**28**	**31**	**degrees**	**Average Moisture Content**	**29.7%**
Cohesion	**320**	**160**	**psf**	**Average Dry Density (pcf)**	**96.7**
				Percent Saturation	**100.0%**



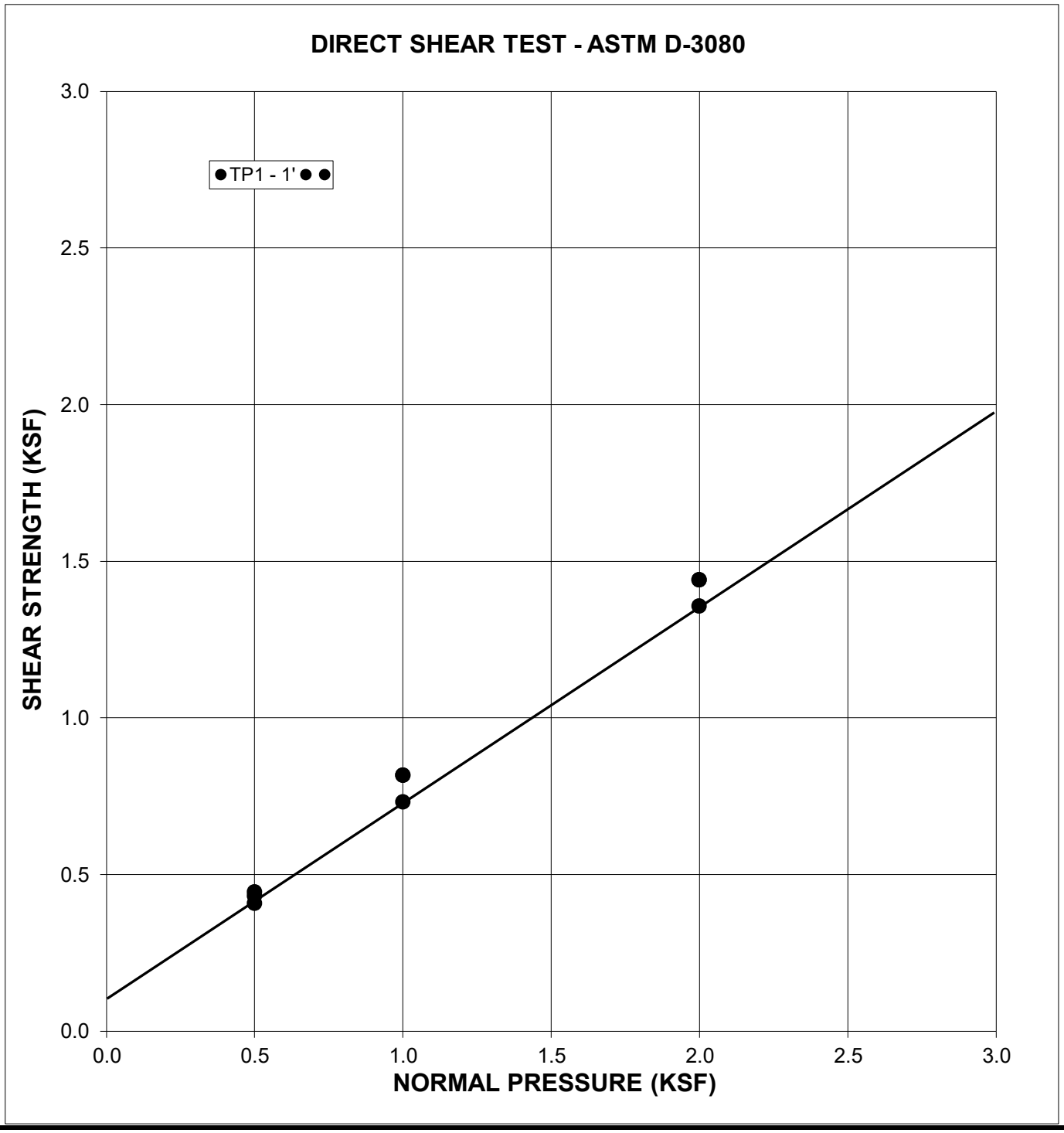
SHEAR DIAGRAM B-4

JN: **SL21.3819** CONSULTANT **JAI**
CLIENT: **Irvine/Kavasch-N of 486 W Avenue 44**

EARTH MATERIAL: **SOIL**

Sample repeatedly sheared to determine residual strength.

	RESIDUAL			
Phi Angle	**32**	**degrees**	**Average Moisture Content**	**39.6%**
Cohesion	**100**	**psf**	**Average Dry Density (pcf)**	**76.9**
			Percent Saturation	**96.2%**



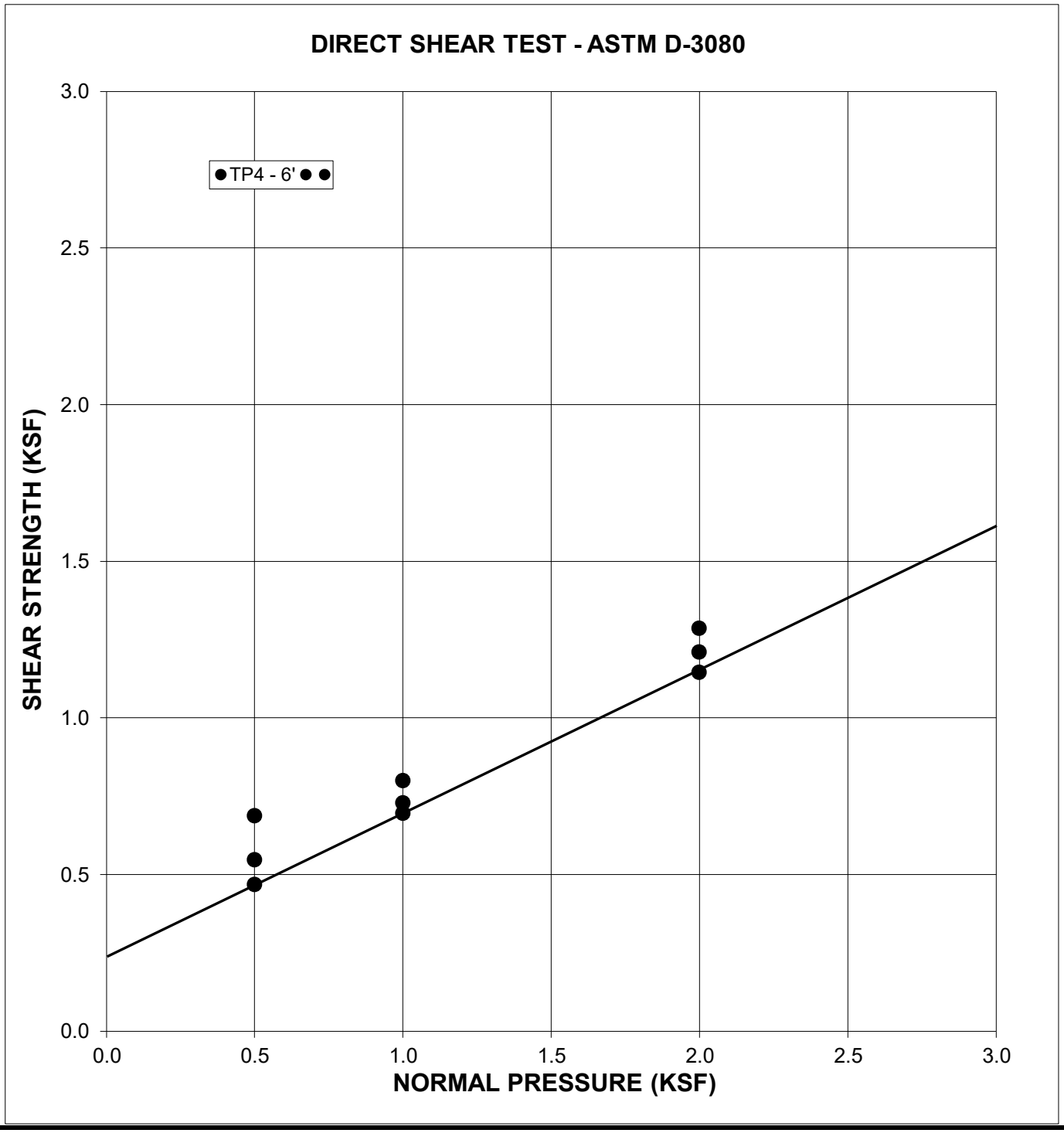
SHEAR DIAGRAM B-5

JN: **SL21.3819** CONSULTANT **JAI**
CLIENT: **Irvine/Kavasch-N of 486 W Avenue 44**

EARTH MATERIAL: **BEDROCK**

Sample repeatedly sheared to determine residual strength.

	RESIDUAL			
Phi Angle	**24**	**degrees**	**Average Moisture Content**	**36.7%**
Cohesion	**240**	**psf**	**Average Dry Density (pcf)**	**80.4**
			Percent Saturation	**97.6%**



IRVINE GEOTECHNICAL Inc

RETAINING WALL

IC: **21198-I** CONSULT: **JAI**
CLIENT: **KAVASCH**

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:	BEDDING	WALL HEIGHT	18 feet
SHEAR DIAGRAM:	B-5	BACKSLOPE ANGLE:	0 degrees
COHESION:	240 psf	SURCHARGE:	0 pounds
PHI ANGLE:	24 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	110 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	160.0 psf	FINAL TENSION CRACK:	100 feet

PHID = ATAN(TAN(PHI)/FS) = 16.5 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	111.2 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	12229.9 pounds
NUMBER OF TRIAL WEDGES ANALYZED	4059 trials
LENGTH OF FAILURE PLANE	17.0 feet
DEPTH OF TENSION CRACK	4.2 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	10.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**6085.7 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**37.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**55.0 pcf**

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALLS SUPPORTING BEDDING MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 55 POUNDS PER CUBIC FOOT.

IRVINE GEOTECHNICAL Inc

RETAINING WALL

IC: **21198-I** CONSULT: **JAI**
CLIENT: **KAVASCH**

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	RETAINED LENGTH	18 feet
SHEAR DIAGRAM:	B-3	BACKSLOPE ANGLE:	0 degrees
COHESION:	320 psf	SURCHARGE:	0 pounds
PHI ANGLE:	28 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	125 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	320.0 psf	FINAL TENSION CRACK:	100 feet

PHID = ATAN(TAN(PHI)/FS) = 28.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.365 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	43 degrees
AREA OF TRIAL FAILURE WEDGE	160.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	20076.7 pounds
NUMBER OF TRIAL WEDGES ANALYZED	4059 trials
LENGTH OF FAILURE PLANE	19.1 feet
DEPTH OF TENSION CRACK	4.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	14.0 feet
CALCULATED THRUST ON PILE	**7108.1 pounds**

THE CALCULATION INDICATES THAT FOR THE DESIGN GROUND MOTION, THE UNBALANCED FORCE ON RETAINING WALLS SUPPORTING BEDDING IS 7.109 KIPS.

IRVINE GEOTECHNICAL Inc

RETAINING WALL

IC: **21198-I** CONSULT: **JAI**
CLIENT: **KAVASCH**

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:	BEDDING	WALL HEIGHT	18 feet
SHEAR DIAGRAM:	B-5	BACKSLOPE ANGLE:	0 degrees
COHESION:	240 psf	SURCHARGE:	0 pounds
PHI ANGLE:	24 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	110 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	240.0 psf	FINAL TENSION CRACK:	100 feet

PHID = ATAN(TAN(PHI)/FS) = 24.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.365 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	39	degrees
AREA OF TRIAL FAILURE WEDGE	189.0	square feet
TOTAL EXTERNAL SURCHARGE	0.0	pounds
WEIGHT OF TRIAL FAILURE WEDGE	20788.5	pounds
NUMBER OF TRIAL WEDGES ANALYZED	4059	trials
LENGTH OF FAILURE PLANE	21.9	feet
DEPTH OF TENSION CRACK	4.2	feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	17.0	feet
CALCULATED HORIZONTAL THRUST ON WALL	**8192.8**	**pounds**

THE CALCULATION INDICATES THAT FOR THE DESIGN GROUND MOTION, THE UNBALANCED FORCE ON RETAINING WALLS SUPPORTING BEDDING IS 8.193 KIPS.

IRVINE GEOTECHNICAL Inc

TEMPORARY EXCAVATION HEIGHT

IC: **21198-I** CONSULT: **JAI**
CLIENT: **KAVASCH**

CALCULATION SHEET #

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST). THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT:	8 feet
SHEAR DIAGRAM:	B-2	BACKSLOPE ANGLE:	45 degrees
COHESION:	880 psf	SURCHARGE:	0 pounds
PHI ANGLE:	26 degrees	SURCHARGE TYPE:	P Point
DENSITY:	110 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	3 feet
CD (C/FS):	704.0 psf	FINAL TENSION CRACK:	30 feet
PHID = ATAN(TAN(PHI)/FS) =	21.3 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	38 degrees
AREA OF TRIAL FAILURE WEDGE	25.0 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	2748.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1148 trials
LENGTH OF FAILURE PLANE	3.8 feet
DEPTH OF TENSION CRACK	8.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	3.0 feet
CALCULATED HORIZONTAL THRUST	**-1782.8 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-55.7 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**8.0 feet**

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE TEMPORARY EXCAVATIONS IN BEDROCK UP TO 8 FEET HIGH HAVE A NEGATIVE THRUST AND ARE TEMPORARILY STABLE.

IRVINE GEOTECHNICAL Inc

SHORING PILE

IC: **21198-I** CONSULT: **JAI**
CLIENT: **KAVASCH**

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:	BEDDING	RETAINED LENGTH	18 feet
SHEAR DIAGRAM:	B-5	BACKSLOPE ANGLE:	0 degrees
COHESION:	240 psf	SURCHARGE:	0 pounds
PHI ANGLE:	24 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	110 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	2 feet
CD (C/FS):	192.0 psf	FINAL TENSION CRACK:	100 feet

PHID = ATAN(TAN(PHI)/FS) = 19.6 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

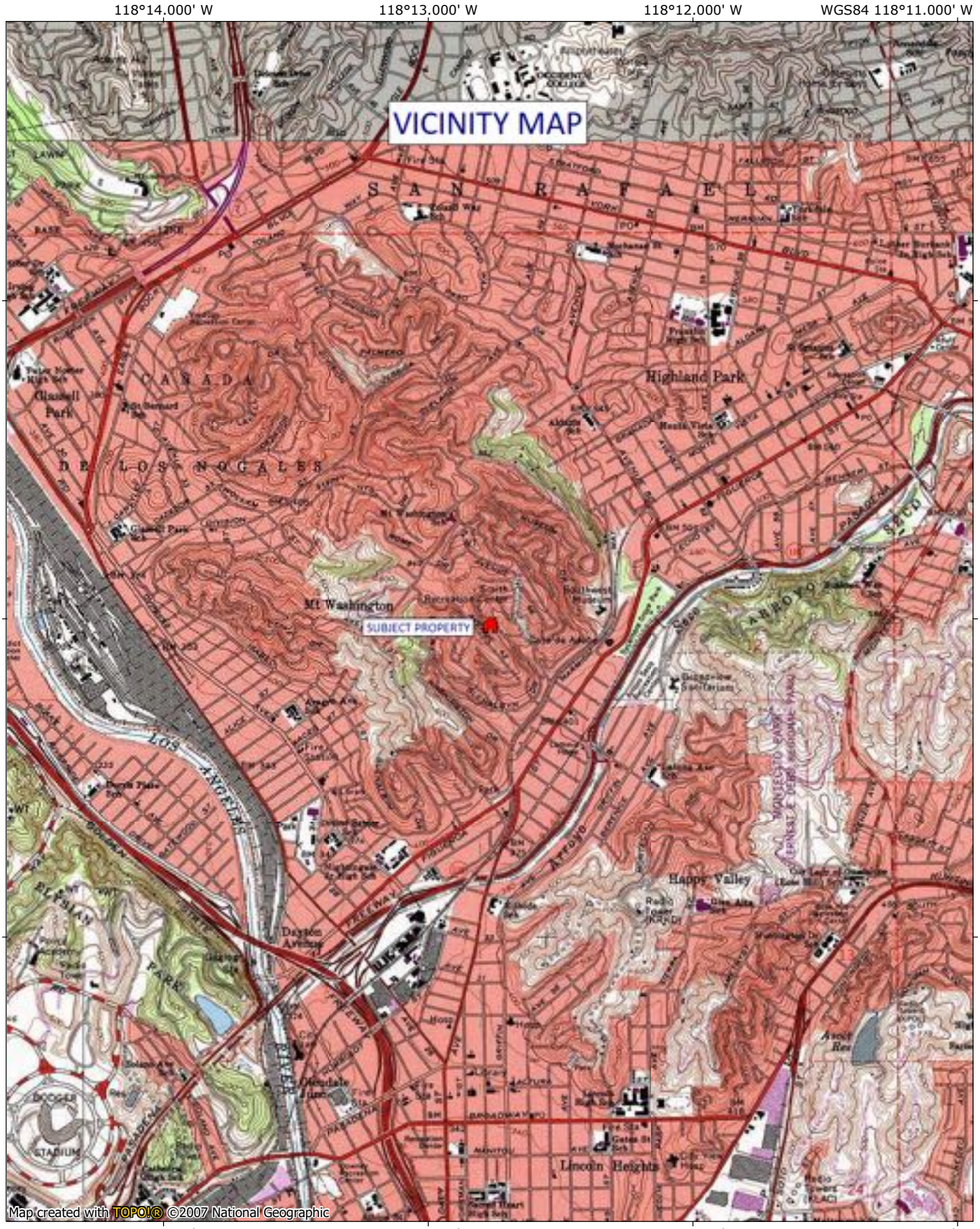
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	55 degrees
AREA OF TRIAL FAILURE WEDGE	104.2 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	11457.6 pounds
NUMBER OF TRIAL WEDGES ANALYZED	4059 trials
LENGTH OF FAILURE PLANE	15.7 feet
DEPTH OF TENSION CRACK	5.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	9.0 feet
CALCULATED THRUST ON PILE	**4659.5 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**28.8 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**35.0 pcf**

THE CALCULATION INDICATES THAT THE PROPOSED SHORING PILES MAY SUPPORTING BEDDING PLANES MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 35 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

TOPO! map printed on 12/23/21 from "IRVINE.tpo"



118°14.000' W 118°13.000' W 118°12.000' W WGS84 118°11.000' W

34°07.000' N
34°06.000' N
34°05.000' N

NATIONAL GEOGRAPHIC

0 .5 1 MILES
1000 0 1000 2000 3000 4000 5000 FEET
1 .5 0 KILOMETERS 1
1000 0 METERS 1000

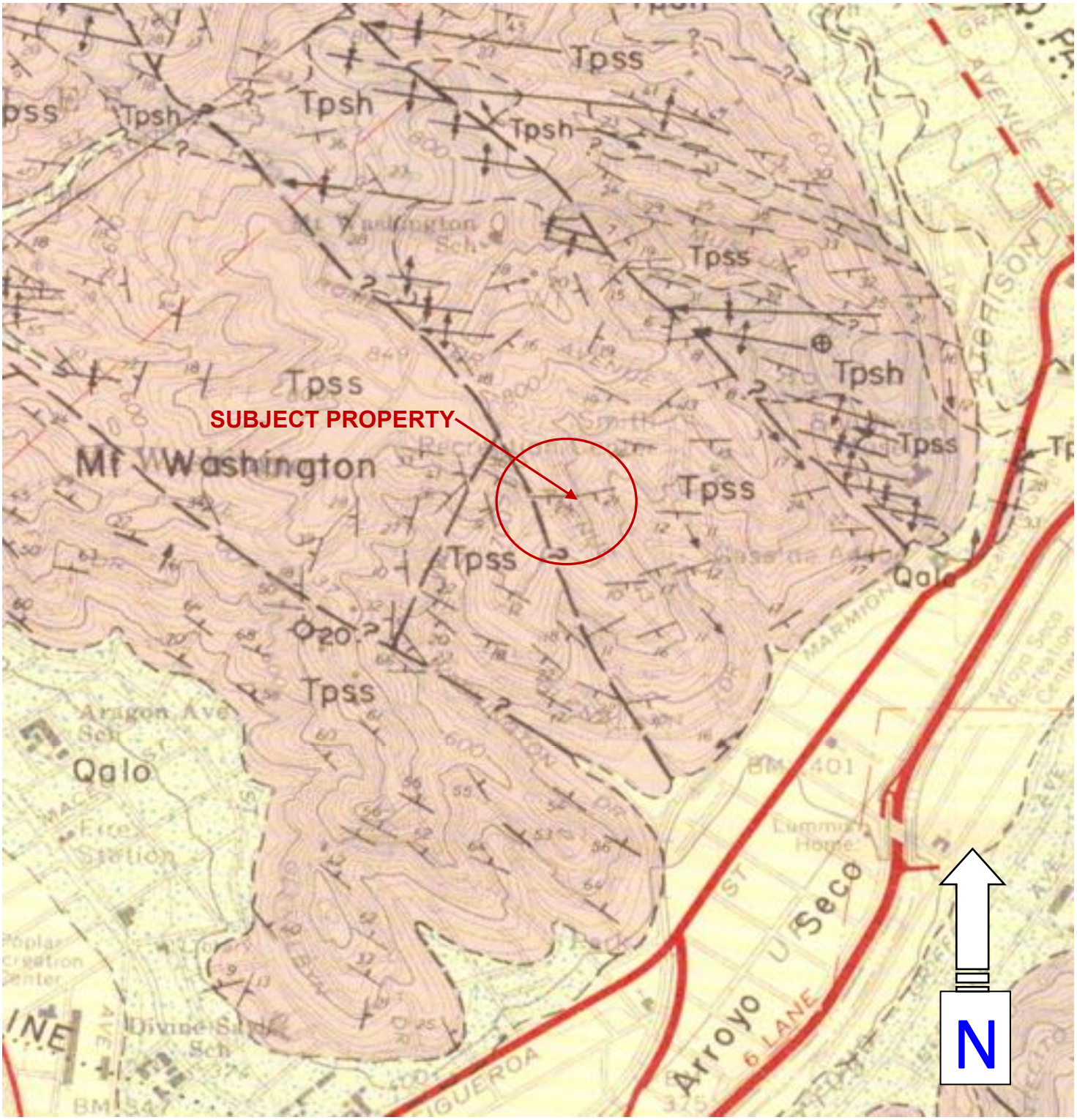
TN MN
11½°
12/23/21

IRVINE GEOTECHNICAL Inc

REGIONAL GEOLOGIC MAP

IC:	**21198**	CONSULT:	**JAI**
CLIENT	**KAVASCH**		
SCALE:	**1" = 1,000'**		

REFERENCE: *Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California* , by Donald L. Lamar, USGS Special Report 101, 1970.



LOG OF TEST PITS

IRVINE GEOTECHNICAL Inc

PROJECT	IC21198 KAVASCH
DRILL DATE	11/12/2021
LOG DATE	11/12/2021
LOGGED BY	MH
DRILL TYPE	Hand Labor
DIAMETER	30 Inches

SURFACE ELEVATION 676 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS slope

TEST PIT 1

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
							676.0	0	**SOIL:** Silty to Clayey Sand, dark grey, brown, slightly moist, medium dense, porous
R	1	n/a	9.4	76.9	22	SM/SC	675.0	1	
							674.0	2	**BEDROCK:** Sandstone and Siltstone, light yellow-brown to orange-brown, tight, very hard, massive, moderately weathered
R	3	n/a	11.2	97.5	43		673.0	3	
							672.0	4	**END TP1 @ 4':** No Water; No Caving; No Fill Joints: N-S; 38W & N3W; 90

SURFACE ELEVATION 681 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS slope

TEST PIT 2

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SC	681.0	0	**SOIL:** Clayey Sand, grey brown, slightly moist, medium
							680.0	1	**BEDROCK:** Sandstone and Siltstone, yellow-brown to orange-brown, very hard, massive, moderately weathered
							679.0	2	
R	3	n/a	8.7	101.8	37		678.0	3	**END TP2 @ 3':** No Water; No Caving; No Fill Joints: N10W; 85E & N15W; 75E

LOG OF TEST PITS

IRVINE GEOTECHNICAL Inc

PROJECT	IC21198 KAVASCH
DRILL DATE	11/12/2021
LOG DATE	11/12/2021
LOGGED BY	MH
DRILL TYPE	Hand Labor
DIAMETER	30 Inches

SURFACE ELEVATION 680 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS slope

TEST PIT 3

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SM	680.0	0	**SOIL:** Silty Sand, grey brown, slightly moist, medium dense, porous
							679.0	1	
R	2.5	n/a	12.7	79.8	31		678.0	2	
							677.0	3	
							676.0	4	**BEDROCK:** Sandstone and Siltstone, yellow-brown to orange-brown, bedded, hard, moderately weathered
							675.0	5	
							674.0	6	
R	7	n/a	22.8	83.8	62		673.0	7	**END TP3 @ 7':** No Water; No Caving; No Fill Bedding N47E; 16SE

SURFACE ELEVATION 702 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS slope

TEST PIT 4

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SC	702.0	0	**FILL:** Clayey Sand, grey, brown, slightly moist, medium dense
R	1	n/a	7.0	76.1	16		701.0	1	
R	2.5	n/a	10.4	80.7	26		700.0	2	
						SC	699.0	3	**SOIL:** Clayey Sand, grey brown, slightly moist, medium dense, porous
							698.0	4	**BEDROCK:** Sandstone, orange-brown, massive, very hard, moderately weathered
							697.0	5	
R	6	n/a	22.5	80.4	57		696.0	6	**END TP4 @ 6':** No Water; No Caving; Fill to 2 feet

LOG OF TEST PITS

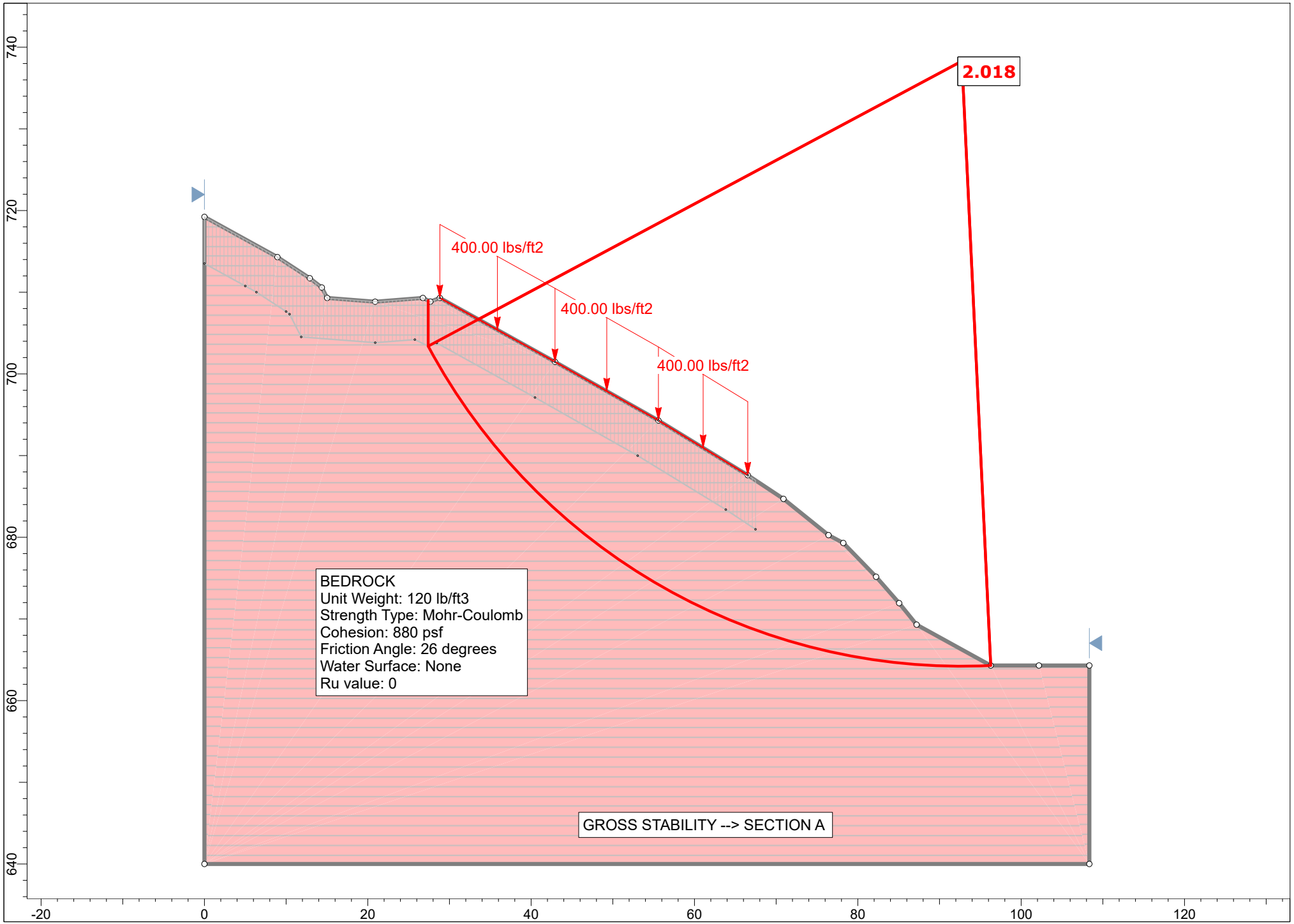
IRVINE GEOTECHNICAL Inc

PROJECT	IC21198 KAVASCH
DRILL DATE	11/12/2021
LOG DATE	11/12/2021
LOGGED BY	MH
DRILL TYPE	Hand Labor
DIAMETER	30 Inches

SURFACE ELEVATION	704 feet
DRILLING CONTRACTOR	Mike's Excavating Service
SURFACE CONDITIONS	slope

TEST PIT 5

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SM	704.0	0	**FILL:** Silty Sand, yellow grey, slightly moist, medium dense
							703.0	1	
							702.0	2	
							701.0	3	
						SC	700.0	4	**SOIL:** Clayey Sand, grey-brown, moist, medium dense, roots
							699.0	5	
							698.0	6	**BEDROCK:** Sandstone. orange, hard, massive, moderately weathered
							697.0	7	**END TP7 @ 7':** No Water; No Caving; Fill to 3.5


2.018
400.00 lbs/ft2
400.00 lbs/ft2
400.00 lbs/ft2
BEDROCK
Unit Weight: 120 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 880 psf
Friction Angle: 26 degrees
Water Surface: None
Ru value: 0
GROSS STABILITY --> SECTION A
740
720
700
680
660
640
-20
0
20
40
60
80
100
120

Slide Analysis Information

KAVASCH --> FRONTENAC

Materials

BEDROCK	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	880
Friction Angle [deg]	26
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	2.017950
Center:	92.746, 738.302
Radius:	74.086
Left Slip Surface Endpoint:	27.393, 703.405
Right Slip Surface Endpoint:	96.278, 664.300
Left Slope Intercept:	27.393 708.981
Right Slope Intercept:	96.278 664.300
Resisting Moment:	9.84956e+06 lb-ft
Driving Moment:	4.88097e+06 lb-ft
Total Slice Area:	967.127 ft2
Surface Horizontal Width:	68.885 ft
Surface Average Height:	14.0397 ft

Global Minimum Support Data

No Supports Present

Valid and Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces:	8241
Number of Invalid Surfaces:	0

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 2.01795

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [deg]	Base Material	Base Cohesion [psf]	Base Friction Angle [deg]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	1.3777	1134.54	-60.8064	BEDROCK	880	26	443.567	895.095	30.9497	0	30.9497	824.827	824.827
2	1.3777	1511.13	-58.689	BEDROCK	880	26	578.051	1166.48	587.367	0	587.367	1537.68	1537.68
3	1.3777	1745.42	-56.6936	BEDROCK	880	26	623.788	1258.77	776.599	0	776.599	1725.99	1725.99
4	1.3777	1953.75	-54.7992	BEDROCK	880	26	662.734	1337.36	937.734	0	937.734	1877.19	1877.19
5	1.3777	2139.83	-52.9899	BEDROCK	880	26	698.485	1409.51	1085.65	0	1085.65	2012.23	2012.23
6	1.3777	2306.38	-51.2537	BEDROCK	880	26	731.37	1475.87	1221.71	0	1221.71	2133.1	2133.1
7	1.3777	2455.58	-49.5808	BEDROCK	880	26	761.659	1536.99	1347.03	0	1347.03	2241.37	2241.37
8	1.3777	2589.19	-47.9635	BEDROCK	880	26	789.574	1593.32	1462.52	0	1462.52	2338.31	2338.31
9	1.3777	2708.64	-46.3954	BEDROCK	880	26	815.302	1645.24	1568.97	0	1568.97	2424.99	2424.99
10	1.3777	2815.15	-44.8712	BEDROCK	880	26	839	1693.06	1667.03	0	1667.03	2502.26	2502.26
11	1.3777	2909.73	-43.3864	BEDROCK	880	26	860.806	1737.06	1757.24	0	1757.24	2570.87	2570.87
12	1.3777	2992.57	-41.9371	BEDROCK	880	26	881.058	1777.93	1841.03	0	1841.03	2632.59	2632.59
13	1.3777	3063.28	-40.5202	BEDROCK	880	26	899.171	1814.48	1915.98	0	1915.98	2684.49	2684.49
14	1.3777	3124.24	-39.1325	BEDROCK	880	26	915.559	1847.55	1983.77	0	1983.77	2728.69	2728.69
15	1.3777	3176.11	-37.7717	BEDROCK	880	26	930.434	1877.57	2045.32	0	2045.32	2766.31	2766.31
16	1.3777	3219.4	-36.4356	BEDROCK	880	26	943.869	1904.68	2100.9	0	2100.9	2797.68	2797.68
17	1.3777	3254.54	-35.1221	BEDROCK	880	26	955.921	1929	2150.77	0	2150.77	2823.15	2823.15
18	1.3777	3281.94	-33.8294	BEDROCK	880	26	966.649	1950.65	2195.16	0	2195.16	2843	2843
19	1.3777	3301.93	-32.556	BEDROCK	880	26	976.104	1969.73	2234.28	0	2234.28	2857.47	2857.47
20	1.3777	3314.85	-31.3005	BEDROCK	880	26	984.331	1986.33	2268.32	0	2268.32	2866.81	2866.81
21	1.3777	3319.49	-30.0614	BEDROCK	880	26	992.19	2002.19	2300.82	0	2300.82	2875.08	2875.08
22	1.3777	3309.81	-28.8377	BEDROCK	880	26	997.562	2013.03	2323.05	0	2323.05	2872.32	2872.32
23	1.3777	3292.71	-27.6283	BEDROCK	880	26	1000.71	2019.38	2336.08	0	2336.08	2859.87	2859.87
24	1.3777	3269.51	-26.432	BEDROCK	880	26	1002.75	2023.5	2344.52	0	2344.52	2842.99	2842.99
25	1.3777	3240.41	-25.2481	BEDROCK	880	26	1003.71	2025.43	2348.48	0	2348.48	2821.82	2821.82
26	1.3777	3205.59	-24.0756	BEDROCK	880	26	1003.61	2025.24	2348.08	0	2348.08	2796.51	2796.51
27	1.3777	3165.19	-22.9137	BEDROCK	880	26	1002.48	2022.95	2343.39	0	2343.39	2767.14	2767.14
28	1.3777	3119.38	-21.7617	BEDROCK	880	26	1000.33	2018.62	2334.52	0	2334.52	2733.84	2733.84
29	1.3777	3066.17	-20.6188	BEDROCK	880	26	935.231	1887.25	2065.16	0	2065.16	2417.04	2417.04
30	1.3777	2998.94	-19.4845	BEDROCK	880	26	886.52	1788.95	1863.63	0	1863.63	2177.29	2177.29

31	1.3777	2925.66	-18.3581	BEDROCK	880	26	878.969	1773.72	1832.39	0	1832.39	2124.07	2124.07
32	1.3777	2844.63	-17.239	BEDROCK	880	26	869.995	1755.61	1795.26	0	1795.26	2065.21	2065.21
33	1.3777	2735.09	-16.1266	BEDROCK	880	26	856.184	1727.74	1738.12	0	1738.12	1985.67	1985.67
34	1.3777	2615.43	-15.0205	BEDROCK	880	26	840.504	1696.1	1673.25	0	1673.25	1898.78	1898.78
35	1.3777	2491.06	-13.92	BEDROCK	880	26	823.838	1662.46	1604.29	0	1604.29	1808.47	1808.47
36	1.3777	2367.51	-12.8248	BEDROCK	880	26	807.096	1628.68	1535.02	0	1535.02	1718.76	1718.76
37	1.3777	2284.26	-11.7343	BEDROCK	880	26	796.885	1608.08	1492.77	0	1492.77	1658.3	1658.3
38	1.3777	2139.74	-10.6481	BEDROCK	880	26	776.256	1566.45	1407.42	0	1407.42	1553.37	1553.37
39	1.3777	1946.25	-9.5658	BEDROCK	880	26	747.145	1507.7	1286.98	0	1286.98	1412.89	1412.89
40	1.3777	1747.86	-8.48691	BEDROCK	880	26	716.91	1446.69	1161.88	0	1161.88	1268.86	1268.86
41	1.3777	1527.74	-7.41104	BEDROCK	880	26	682.679	1377.61	1020.26	0	1020.26	1109.06	1109.06
42	1.3777	1294.3	-6.33779	BEDROCK	880	26	645.844	1303.28	867.851	0	867.851	939.584	939.584
43	1.3777	1043.72	-5.26677	BEDROCK	880	26	605.718	1222.31	701.837	0	701.837	757.674	757.674
44	1.3777	808.565	-4.1976	BEDROCK	880	26	567.88	1145.95	545.286	0	545.286	586.965	586.965
45	1.3777	684.792	-3.12988	BEDROCK	880	26	548.979	1107.81	467.086	0	467.086	497.105	497.105
46	1.3777	569.729	-2.06326	BEDROCK	880	26	531.417	1072.37	394.423	0	394.423	413.568	413.568
47	1.3777	450.421	-0.99735	BEDROCK	880	26	512.951	1035.11	318.023	0	318.023	326.953	326.953
48	1.3777	326.876	0.0682143	BEDROCK	880	26	493.574	996.007	237.849	0	237.849	237.261	237.261
49	1.3777	199.093	1.1338	BEDROCK	880	26	473.274	955.044	153.863	0	153.863	144.496	144.496
50	1.3777	67.0716	2.20014	BEDROCK	880	26	452.043	912.201	66.0216	0	66.0216	48.6547	48.6547

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 2.01795

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [deg]
1	27.3931	703.405	0	0	0
2	28.7708	700.939	-533.767	0	0
3	30.1485	698.674	1.53341	0	0
4	31.5262	696.577	771.976	0	0
5	32.9039	694.624	1691.81	0	0
6	34.2816	692.797	2715.24	0	0
7	35.6593	691.08	3806.75	0	0
8	37.037	689.462	4938.25	0	0
9	38.4147	687.934	6087.2	0	0
10	39.7924	686.488	7235.34	0	0
11	41.1701	685.116	8367.73	0	0
12	42.5478	683.814	9472.07	0	0
13	43.9255	682.576	10539	0	0
14	45.3032	681.399	11558.4	0	0
15	46.6809	680.278	12522.8	0	0
16	48.0586	679.21	13426.6	0	0
17	49.4363	678.193	14265.1	0	0
18	50.814	677.224	15034.5	0	0
19	52.1917	676.301	15731.8	0	0
20	53.5694	675.421	16354.5	0	0
21	54.9471	674.584	16900.8	0	0
22	56.3248	673.786	17370.8	0	0
23	57.7025	673.028	17760.9	0	0
24	59.0802	672.306	18069.1	0	0
25	60.4579	671.622	18295.6	0	0
26	61.8356	670.972	18441	0	0
27	63.2133	670.356	18506	0	0
28	64.591	669.774	18491.9	0	0
29	65.9687	669.224	18399.9	0	0
30	67.3464	668.706	18184.1	0	0
31	68.7241	668.218	17873.2	0	0
32	70.1018	667.761	17502	0	0
33	71.4795	667.334	17072.9	0	0
34	72.8572	666.935	16587.7	0	0
35	74.2349	666.565	16050.2	0	0
36	75.6126	666.224	15464.9	0	0
37	76.9903	665.91	14836.3	0	0
38	78.368	665.624	14167.4	0	0
39	79.7457	665.365	13464.3	0	0
40	81.1234	665.133	12735.5	0	0
41	82.5011	664.927	11988.3	0	0
42	83.8788	664.748	11232.2	0	0
43	85.2565	664.595	10476.7	0	0
44	86.6342	664.468	9732.73	0	0
45	88.0119	664.367	9006.8	0	0
46	89.3896	664.292	8286.93	0	0
47	90.7673	664.242	7575.59	0	0
48	92.145	664.218	6877.71	0	0
49	93.5227	664.22	6198.46	0	0
50	94.9004	664.247	5543.32	0	0
51	96.2781	664.3	0	0	0

Discharge Sections

Entity Information

Distributed Load

X	Y
28.8258	709.3
42.9241	701.474
55.5875	694.3
66.5294	687.602

Tension Crack

X	Y
0	713.517
5	710.763
6.36568	710.011
9.99794	707.641
10.4254	707.309
11.8449	704.531
20.9054	703.83
25.7661	704.208
27.4208	703.391
28.4492	703.79
40.4784	697.113
53.0493	689.991
63.8396	683.386
67.4694	680.974

External Boundary

X	Y
102.18	664.3
96.2781	664.3
87.196	669.3
85.0626	671.935
82.242	675.164
78.2176	679.3
76.4094	680.268
70.9011	684.697
66.5294	687.602
55.5875	694.3
42.9241	701.474
28.8258	709.3
27.6624	708.848
26.7482	709.3
20.9041	708.845
15.0231	709.3
14.3704	710.578
12.9005	711.717
8.94198	714.3
0	719.228
0	640
108.343	640
108.347	664.299

IRVINE GEOTECHNICAL Inc

GEOLOGIC MAP

PROJECT: IC21198 - KAVASCH

CONSULTANT: JAI | SCALE: 1" = 10'

LEGEND

- TP1 --- NUMBER & LOCATION OF TEST PIT
- 30 --- STRIKE & DIP OF BEDDING
- 30 --- STRIKE & DIP OF JOINTING
- --- GEOLOGIC CONTACT

PUENTE FORMATION
SANDSTONE & SILTSTONE
(FILL & SOIL COVER)

TP1
TP2
TP3
TP4
TP5

PROPOSED POOL

38
16
85
75

A
A

W. AVENUE

FRONTENAC AVENUE

700.00
670.00

44

PROPOSED RESIDENCE & DECKS

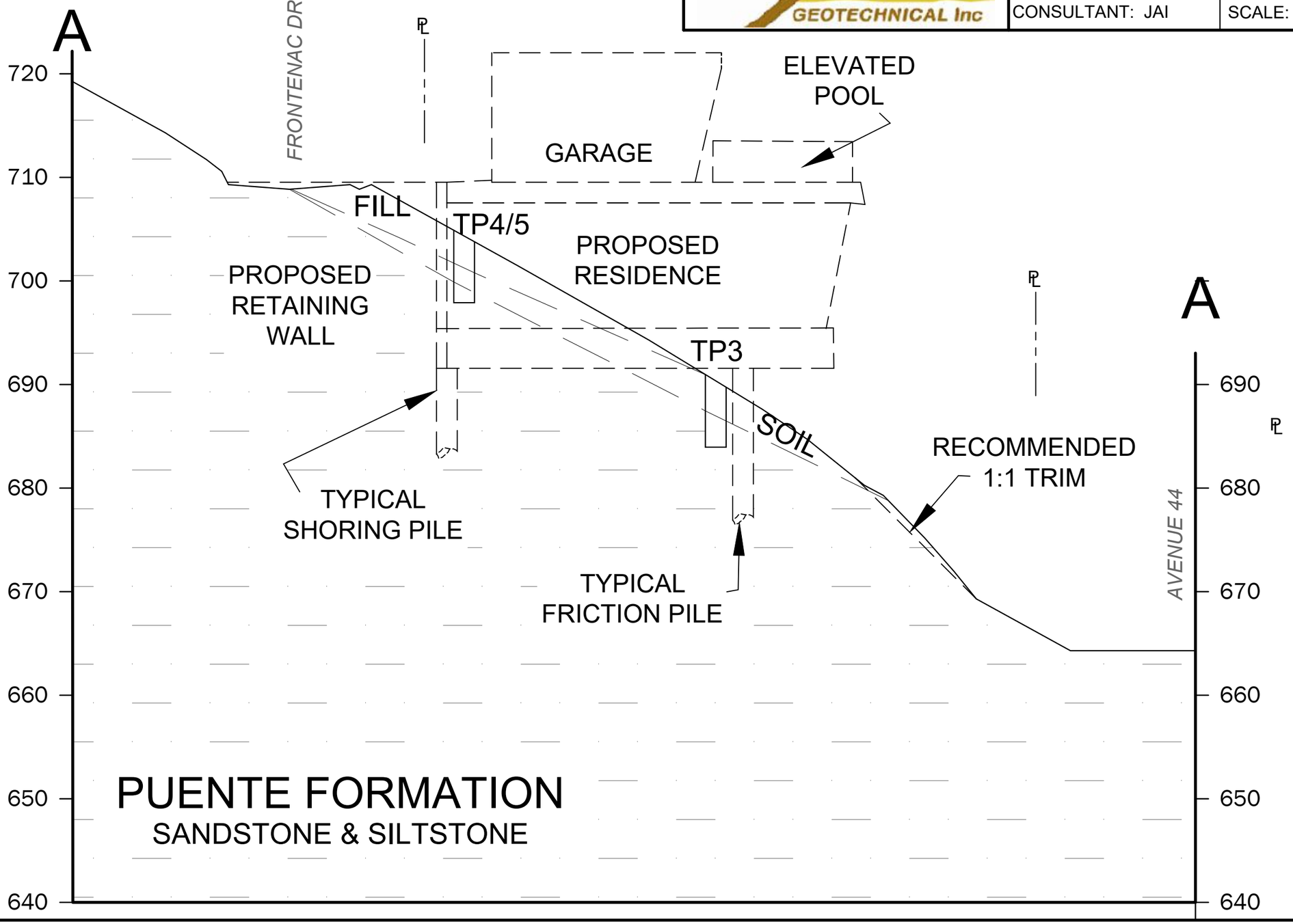
PUENTE FORMATION
SANDSTONE & SILTSTONE
(SOIL COVER)

IRVINE GEOTECHNICAL Inc

SECTION A - A

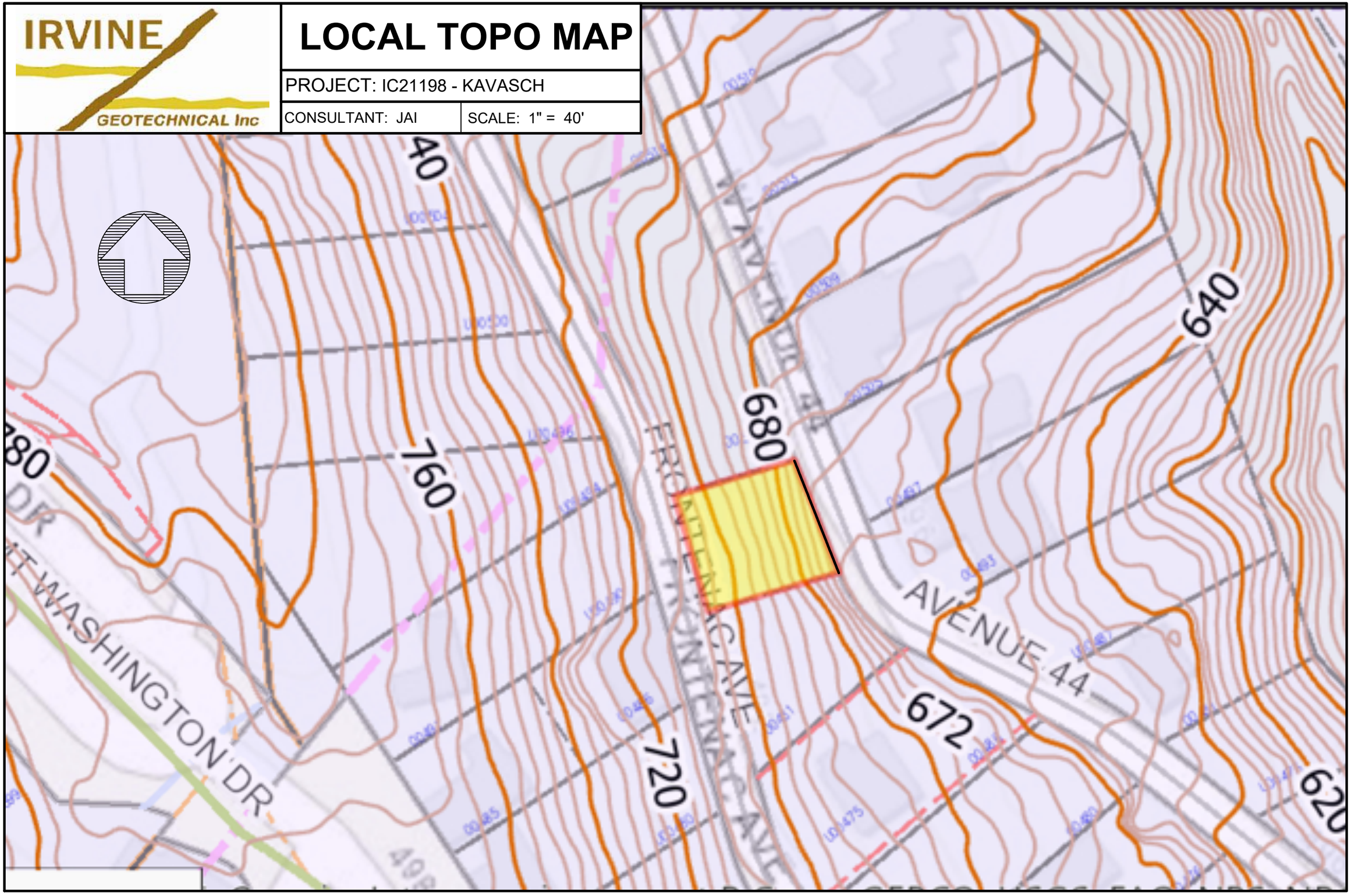
PROJECT: IC21198 - KAVASCH

CONSULTANT: JAI | SCALE: 1" = 10'



IRVINE GEOTECHNICAL Inc

LOCAL TOPO MAP

PROJECT: IC21198 - KAVASCH

CONSULTANT: JAI | SCALE: 1" = 40'