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GEOLOGIC & SOILS ENGINEERING INVESTIGATION

PROPOSED SINGLE FAMILY RESIDENCE

TRACT 7342, LOT 440 - APN 2526-010-009

WEST TRAIL

KAGEL CANYON AREA, COUNTY OF LOS ANGELES, CALIFORNIA

FOR

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PIN# 6927

SEPTEMBER 22, 2017

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INTRODUCTION

This report presents the results of our Geologic & Soils Engineering Investigation performed at APN 2526-010-009, West Trail in the Kagel Canyon area of Los Angeles County. The purpose of our investigation was to evaluate subsurface conditions as they relate to the proposed construction of a single family residence on the subject property. This investigation is limited to the area of the proposed development and does not warrant the remaining portions of the property.

SCOPE

This investigation is based upon:

- A topographic site plan prepared by Clemons Land Surveying - licensed Land Surveyors, was utilized as the base map for the enclosed Geologic Map, Plate A. This map appears to accurately reflect topographic conditions as observed at the property.
- A review of preliminary plans by Zigloo.ca.
- The review of one (1) seismic trench.
- Preparation of the enclosed Geologic Map which locates the proposed improvements and our explorations (see APPENDIX I).
- Preparation of the enclosed Geologic Cross-Section(s) (see APPENDIX I).
- Preparation of site exploration logs (see APPENDIX I).
- Laboratory testing and analysis of samples obtained within the excavations (see APPENDIX II).
- Calculations which may include, but are not limited to, bearing value, lateral pressure, active earth pressure, slope stability (see APPENDIX III).
- The review of available maps and reports prepared by this office and others (see APPENDIX IV).
- Preparation of this report.

The data which supports the following SUMMARY OF FINDINGS, CONCLUSIONS and RECOMMENDATIONS is contained within Appendices I through IV.

The scope of our exploration is limited to the areas explored for the proposed development as delineated on the enclosed Geologic Map. This report should not be considered as a comprehensive evaluation of the entire property. This report has not been prepared for use by other parties or for other purposes (or developments), and may not contain sufficient information for other than the intended use. If construction is delayed more than one year, this office should be contacted to perform an update and to verify the current site conditions.

LOCATION AND TOPOGRAPHY

General

The subject property is located near the base of the southern flank of the San Gabriel Mountains in the Kagel Canyon area of Los Angeles County, California. The subject site is an undeveloped hillside parcel situated along the east side of West Trail, with the northwest corner approximately five hundred feet (500') northwest of its intersection with Kagel Canyon Road, as measured along the roadway centerline. For reference, see the attached Vicinity Map for the location of the subject property (see APPENDIX I).

The property consists of a twenty to fifty-five foot (20' - 55') wide, slightly east-sloping building pad area adjacent to West Trail. Slopes descend from the east margin of the building pad approximately fifty to sixty feet (50'-60') to Kagel Canyon Road below at slope ratios ranging from 1½:1 (33°) to 1:1 (45°). A southeast-trending ravine, which conveys West Trail sheet flow runoff via a corrugated metal pipe culvert, is present at the southwestern corner of the site.

Slope areas are covered with a light to moderate growth of scattered shrubs and trees. For specific topographic conditions, refer to the attached Geologic Map, Plate A and Geologic Cross-Sections, Plate B (see Attachments).

PROPOSED DEVELOPMENT

Final building plans have not been prepared and await the conclusions and recommendations of this investigation. However, it is our understanding that the proposed development will consist of constructing a multi-level, single family residence at the northwest corner of the site. Retaining walls will be constructed along the west and north sides of the residence and will serve as interior walls. An elevated driveway, to be constructed of manufactured steel grids, is proposed and will require foundation support. Temporary cuts up to sixteen feet (16') in height are anticipated during construction of the wall(s). The proposed structure will utilize a slab-on-grade floor system. Grading will be limited to excavations for the lower level retaining walls, excavation of future foundations and placement of retaining wall backfill. For reference, the locations of proposed improvements are shown on the attached Geologic Map, Plate A.

SUMMARY OF FINDINGS

Research

A representative from this office conducted research of available engineering geologic and geotechnical engineering reports prepared for the subject property and adjacent properties at the County of Los Angeles Materials Engineering Division on August 1, 2017.

In addition, research of available geologic maps and publications prepared for the area by the U.S. Geological Survey and California Division of Mines and Geology was conducted. SubSurface Designs, Inc. has reviewed the referenced reports and has incorporated applicable information from these sources into this report.

Soils engineering and/or geologic reports for the subject site were not located during our research. It should be noted that other reports may have been prepared for the subject site in the past but were not found during records research, or were not submitted to the governing reviewing agency, and thus are not part of public record.

Field Investigation

Site exploratory studies were conducted on August 11, 2017. Field investigation consisted of geologic reconnaissance and mapping of the subject site and adjacent areas. Additionally, one (1) continuous seismic trench was excavated with a backhoe within and adjacent to the area of proposed construction. The backhoe test trench was one hundred and sixty-six feet (166') in length and ranged from five to eleven feet (5'-11') deep. For reference, the location and approximate alignment of the seismic trench is shown on the enclosed Geologic Map, Plate A (pocket attachment).

Subsurface conditions encountered in the seismic trench were logged in detail by a representative of this office. Further, representative samples of the earth materials encountered were obtained. The trench was backfilled with the excavated materials. Backfill was not compacted and should be monitored for future settlement.

Undisturbed samples were obtained within the backhoe trench through the use of a thin-walled, steel, hand-held sampler. The soil is retained in 1" high brass rings with a 2.50" outside diameter and a 2.37" inside diameter. Bulk samples were obtained for testing and analysis. All undisturbed and bulk samples were sent to the laboratory for examination, testing, and classification, using the Unified Classification system.

Site Conditions

Currently vacant, it is possible that a structure was present at one time on the subject property. In addition to the wedge of earth fill present on the west side of the building pad, a vacant water meter is present. Concrete rubble and debris are also scattered on the surface of the site.

Drainage within the site comprises of sheet flow runoff of precipitation derived primarily within property boundaries.

GEOLOGIC CONDITIONS

Earth Materials

A relatively limited area of earth fill (ef), up to seven feet (7') thick, is present along a portion of the west side of the subject site and may have been associated with previous site development. Surficial slopewash (Qsw) deposits mantle the terrace deposits and bedrock on the slopes below the existing building pad area. Across the gently sloping building pad area a well-developed natural soil (Qns), including a moderately well developed Bt horizon, ranging from three feet to five feet (3'-5') thick has developed atop the terrace deposits. The terrace deposits (Qt), which appear to thicken to the north, mantle the bedrock over the building pad and surrounding areas, while a younger terrace deposit (Qty) is mapped at the southeast corner of the site. Underlying the surficial soils on the subject site and surrounding areas is sedimentary bedrock assigned to the Modelo Formation (Tm) of late Miocene geologic age.

The earth materials encountered on the subject property are briefly described below. For approximate depths and more detailed descriptions, refer to the enclosed Seismic Trench ST 01, Plate C (see APPENDIX I).

Earth Fill (ef)

Where observed in the seismic trench, the earth fill consists of silty sand matrix with scattered to abundant gravel, cobbles and occasional boulder which is brown to dark brown, slightly moist, slightly to moderately compact with occasional bit of construction debris. An inverted, infilled and roughly seven foot (7') long section of 32" diameter concrete pipe was exposed on the east wall of the trench at approximately Sta 1+33 (Plate C, Appendix I).

Slopewash (Qsw)

The slopewash consists of fine-grained sandy silt with scattered rock fragments and occasional rounded cobbles and boulder which is light brown, dry, loose, porous with abundant roots and rootlets.

Natural Soil (Qns)

Where observed within the seismic trench, the natural soil consists of an A horizon and moderately well developed Bt horizon. The A horizon consists of a silty sand to sandy silt with scattered gravel and cobbles and occasional small boulder which is light brown, dry, loose and porous with roots. The Bt horizon consists of a clayey sand to clayey silty sand (matrix) with scattered gravel and occasional small cobble which is light reddish brown to reddish brown, slightly moist, very stiff to hard with areas of moderately developed, clay- and caliche-lined ped structure.

Terrace Deposits (Qt and Qty)

Terrace deposits (Qt) consist of a gravelly sand to sand/silty sand with gravel (matrix) with scattered sub-rounded to rounded cobbles and occasional boulder that is light yellowish brown to light reddish/orange brown, slightly moist, moderately dense to dense, poorly bedded to massive with occasional sub-horizontal laminations.
As mentioned, what have been interpreted as younger terrace deposits (Qty) have been mapped at the southeast corner of the subject site. Where observed in the Kagel Canyon road cut slope, the younger terrace deposits are topographically at a lower elevation and display greater sorter and stratification than the older terrace deposits (Qt) at higher elevation.

The earth fill, slopewash, natural soil, and terrace deposits were visually classified in accordance with the Unified Soils Classification System.

Bedrock (Tm) - Modelo Formation

Site bedrock consists of interbedded shale, laminated siltstone and fine-grained sandstone which is light tan to light brown to brownish yellow to light gray, slightly moist, moderately hard to hard, well-bedded, moderately grading to slightly weathered and moderately to highly fractured.

Earth material profiles can only be obtained from individual explorations placed on the subject property. Care should be exercised when using these profiles to determine changes in depth or thickness of the earth materials between explorations.

Geologic Structure

Bedding

The terrace (Qt and Qty) deposits underlying the subject site in the near-surface are generally sub-horizontal and thick-bedded, and lack well-developed bedding structure.

Site bedrock exhibits well-defined, continuous and planar bedding (planar or nearly planar surfaces that visibly separate each successive layer of stratified rock). Bedding mapped along outcrops and within the exploratory openings exhibits a dominant east-west to northwest strike accompanied by dips to the north/northeast ranging from 25° to 43°. As depicted on the attached Geologic Cross-Sections (Plate B) the north declination of bedding plane dip is neutrally-oriented with respect to the east-facing descending slope, a condition favorable for continued gross bedrock stability of the site.

Regional bedding depicted on the referenced geologic maps prepared by the CDMG (1974) and Dibblee (1991) exhibits an east-west strike accompanied by dips to the north ranging from 32° to 65°.

Joints

Joint are the surface of a fracture or parting in rock at which no appreciable movement has occurred parallel to the fracture, and only slight movement has occurred normal to the fracture. Joints may be open and void of mineral infilling or joint planes may be occupied with some form of mineral infilling. There were no well-defined, mappable joints within the continuous trench exploration logged by this office.

Folds

A fold is a bend in bedding in rock usually created by deformation in the rock due to faulting and/or regional uplift of the area. Compilation of geologic data obtained on the subject site indicates that there are no significant folds extending through the subject site.

Shears

A shear is defined as deformation resulting from stresses that cause contiguous parts of a body to slide relative to each other in a direction parallel to their plane of contact. A shear plane is a surface or zone along which differential movement has taken place. The presence of a shear plane or shear zone is typically related to local or regional faulting and folding. Shear planes can also develop during mass slope movements such as landslides. The deformation and movement at the shear surface often results in the presence of a zone of gouge or breccia typically consisting of clay and pulverized material derived from the surrounding bedrock. There were no well-defined, significant shears mapped within the seismic trench or road cut exposures logged by this office.

Landslides

A landslide is a general term for a wide variety of processes and landforms involving downslope movement, under gravity, of masses of rock and soil. There is a broad range of landslide morphology, scale, rates and patterns of movement.

The referenced geologic maps prepared by CDMG (1974) and Dibblee (1991) do not depict landslides within or immediately adjacent to the subject property that would adversely affect the stability of the site. Large-scale landslides were not observed by this office during field mapping and reconnaissance of the area.

Groundwater

All water below the surface of the Earth is referred to as groundwater, or subsurface water. The equivalent term for water on the land surface is surface water. Groundwater occurs in two different zones below the subsurface which are referred to as the unsaturated zone and the saturated zone. The unsaturated zone contains both water and air, and is almost invariably underlain by a saturated zone where all interconnected openings within an earth material are full of water.

Water in the saturated zone is the only groundwater available to supply wells and springs, and is the only water to which the term groundwater is correctly applied. The level of water in the saturated zone at which the hydraulic pressure is equal to the atmospheric pressure is referred to as the water table.

Groundwater occurs in aquifers under two different conditions. Where water only partly fills an aquifer, the upper surface of the saturated zone is free to rise and decline. The water in such aquifers is said to be unconfined, and the aquifers are referred to as unconfined aquifers. Where water completely fills an aquifer that is overlain by a confining bed, the water is said to be confined, and the aquifers are referred to as confined aquifers or as artesian aquifers.

The upper surface of a saturated zone in an unconfined condition is referred to as the potentiometric surface. The potentiometric surface is the elevation of water in the subsurface at which the hydraulic pressure of the water is equal to atmospheric pressure. The potentiometric surface or groundwater table is also the level or elevation of the water observed in a well, or exploratory excavation.

The presence, elevation and movement of groundwater are controlled by one or more of the following; climatic conditions, geologic structure, the hydraulic conductivity of the subsurface materials, irrigation and land use. The presence, elevation and movement of groundwater can vary significantly over short distances. Fluctuations in groundwater levels can occur due to tidal action, seasonal variations in the amount of rainfall, runoff, irrigation rates, alterations in the existing groundwater recharge area (i.e. modifications to the surface drainage and surface water infiltration conditions), and other factors not evident at the time site exploration was conducted. In addition, perched water conditions can develop in areas where bedrock is relatively shallow.

Groundwater was not encountered to the maximum depth (11.0') within site explorations. The depth to groundwater, if encountered in site explorations, is only valid for the date of exploration. The Architect, Structural Engineer and Contractor should be aware of the possibility for groundwater fluctuations while designing and constructing the proposed structure(s).

GEOLOGIC PROVINCES

San Fernando Valley

The San Fernando Valley is an elongated synclinal depression that is bounded on the north by north-dipping reverse faults and the San Gabriel and Santa Susana Mountains on the south by the anticlinal Santa Monica Mountains, on the east by the Verdugo Mountains, and on the west by the Simi Hills. The San Gabriel and Verdugo Mountains consists mainly of granitic and gneissic rocks of Mesozoic and older age. Other highlands around the valley are underlain primarily by middle Miocene to Pleistocene sedimentary rock except for the Simi Hills, which are underlain by Upper Cretaceous strata. The northern margin of the San Fernando Valley lies above the two active faults, which produced the 1971 and 1994 earthquakes.

Three sets of low hills within the valley, the Mission Hills, Northridge Hills and Pacoima Hills are related to blind-thrust faults. The northern margin of the valley is in part marked by the active Santa Susana fault system, which is part of a discontinuous north-dipping reverse fault system across the western Transverse Ranges extending from the Red Mountain and San Cayetano faults eastward to the Sierra Madre and Cucamonga faults.

San Gabriel Mountains

According to Weigand et al. (2001) the San Gabriel Mountains make up part of the central Transverse Ranges provinces, a complex, east-trending region of mountain ranges and valleys, about 300 miles long, that averages about 30 miles in width. The province forms a relatively youthful physiographic feature. Uplift began in the late Oligocene (~26 Ma) and accelerated in Pleistocene and recent times (2 Ma to present) in response to concentrated north-south crustal shortening in a region where the plate boundary trends obliquely to the overall motion directions of the North American and Pacific plates.

The San Gabriel Mountains probably did not attain their present height until the late Pleistocene, and have gained their height of over 5,000 feet at their western end by displacements on a set of north-dipping reverse faults, such as the San Fernando and Sierra Madre faults zones, that have caused the mountains to be pushed southward up and over the San Fernando Valley.

STRUCTURAL GEOLOGY

According to Fuchs et al. (2003), a composite image of the fault systems of the M 6.7 San Fernando (1971) and Northridge (1994) earthquakes has been constructed using industry reflection and oil well test data in the upper few kilometers of the crust, relocated aftershocks in the seismogenic crust, and LARSE II (Los Angeles Region Seismic Experiment, Phase II) reflection data in the middle and lower crust. In their image, the San Fernando fault system appears to consist of a decollement that extends 50 km northward at a dip of 25° from near the surface at the Northridge Hills fault, in the northern San Fernando Valley, to the San Andreas fault in the middle to lower crust. It follows a prominent aseismic reflective zone below and northward of the main-shock hypocenter. Interpreted upward splays off this decollement include the Mission Hills and San Gabriel faults and the two main rupture planes of the San Fernando earthquake, which appear to divide the hanging wall into shingle- or wedge-like blocks (see fault section above).

Wright and others (2001) indicate that middle Miocene rifting and rotation left the San Fernando Valley at the northeast corner of an uplifted block that includes the Santa Monica Mountains, Channel Islands, and Simi Hills-Oakridge block. Quaternary north south shortening has inverted the deep East Ventura basin to form the Santa Susana Mountains that form the north edge of the San Fernando Valley and associated folds and thrust faults.

Late Cenozoic faults in the San Fernando Valley are divided into three groups: inactive Miocene normal faults, inactive Pliocene (pre-Saugus) reverse faults, and Quaternary (post-Saugus) active reverse faults that comprise potential seismic sources.

The Mission Hills fault is an east-trending reverse fault that extends along the southern margin of the Santa Susana Mountains and Mission Hills. It dips to the northeast at about 60-70°.

According to Tsutsumi and Yeats (1999), the 1971 San Fernando fault appears to have propagated upward mostly following moderately to steeply north-dipping bedding planes. The fault ruptured along the base of the Saugus Formation in the Mission Wells and Sylmar segments and along the bedding plane within the Modelo Formation in the Tujunga segment.

Measured paleomagnetic declinations for the Saugus Formation taken from Kagel Ridge north of the Merrick syncline show clockwise rotation of about 34° about a vertical axis. The changes in strike for faults and folds at the Pacoima segment (San Fernando fault) boundary are about the same as the amount of clockwise rotation at Kagel Ridge.

According to Tsutsumi and Yeats (1991) the Pacoima segment boundary separates the rotated Kagel Ridge segment from the unrotated Van Norman Lake domain (Mission Hills and Verdugo faults). The Kagel Ridge area represents the area of the San Fernando fault on the south the San Gabriel fault (DiMille and Dillon segments) on the north, the Sylmar basin (segment boundary) on the northwest and Big Tujunga Canyon (segment boundary) on the southeast.

FAULTS

Sierra Madre Fault Zone

The active Sierra Madre fault system is a frontal fault system that bounds the San Gabriel Mountains on their southern edge. The western portion of the Sierra Madre fault system is a frontal fault system that borders the San Gabriel Mountains (forming the 1971 San Fernando Fault) on their southern edge for about 25 miles and is an extension of the Santa Susana fault and Cucamonga fault system (see San Fernando Valley above). This reverse fault dips to the northeast beneath the San Gabriel Mountains. The length of the San Gabriel fault is on the order of about 46 miles long. The most recent surface rupture was during the Holocene. The slip rate on the Sierra Madre fault is on the order of between 0.36 and 4 mm/yr. Recent trenching indicates that the recurrence interval is between 100 to 300 years.

San Fernando Fault

The western portion of the Sierra Madre fault system, a frontal fault system that borders the San Gabriel Mountains on their southern edge, produced surface ruptures during the 1971 San Fernando earthquake forming the San Fernando fault.

The San Fernando fault is generally divided from west to east, into the Mission Wells (western southwest-trending segment), Sylmar (southwest to east-west trending across Sylmar basin) and Tujunga segments (Lakeview Fault Zone of Dibblee, 1991). The slip vectors along the ruptures had a thrust component of up-to-the north as well as left-lateral component of movement. This reverse fault dips to the northeast beneath the San Gabriel Mountains at an angle of about 40°. It may have experienced as much as 25,000 ft of vertical displacement.

The epicenter of the San Fernando (or Sylmar) earthquake occurred on February 9, 1971 with a magnitude of M_w 6.6 and it was located about 6½ miles northeast of San Fernando on the north side of the San Gabriel Mountains. Surface rupture occurred over a distance of 9 miles from Mission Hills eastward to Big Tujunga Canyon. Movement took place on the San Fernando fault. The maximum vertical and horizontal displacements were both about 3 feet. The slip rate is not well known, but trenching studies indicate a recurrence interval as between 100 and 300 years. The slip rate is on the order of about 5 mm/yr.

SEISMIC EVALUATION

General

The Southern California region is located within a tectonically active portion of the earth's crust which has produced both small and sizeable earthquakes throughout time. Faults are generally classified as active, potentially active, or inactive. A fault is considered "active" if it has produced seismic activity within the past 11,000 years. A "potentially active" fault is one where there has been seismic activity along the fault between 11,000 and 1,000,000 years. "Inactive" faults have not produced any seismic activity within the past 1,000,000 years. Active faults are considered to have a high probability of future seismic activity, potentially active faults are considered to have a low probability of future seismic activity, and inactive faults are considered to be no longer capable of producing seismic activity.

The potential exists throughout Southern California for strong ground motion similar to that which occurred during the 1994 Northridge Earthquake. Earthquakes with a magnitude of 5.0 and greater have occurred in Southern California throughout historic time. Strong ground shaking from a moderate to major earthquake can be expected during the lifetime of the structure. This may result in significant damage to structures, hardscape and slopes areas. Since there are so many variables associated with ground movement during an intense earthquake, it is almost impossible to predict the impact of a seismic event to a particular site.

Earthquake Fault Zones

Following the 1971 Sylmar Earthquake, the State of California passed the Alquist-Priolo Special Studies Act in 1972. Active faults within the state were identified and zones were established which prohibited construction of most structures for human occupancy across a known active fault.

The Alquist-Priolo Special Studies Act requires the State Geologist to delineate "special studies zones" along active faults, whereby development therein must include geologic investigation demonstrating the absence of a surface displacement threat prior to construction of habitable structures. "Special Studies Zones" have since been renamed "Earthquake Fault Zones".

The subject property is located within the confines of an "Earthquake Fault Zone". Therefore, the California Geological Survey requires a special earthquake study to be performed prior to construction of habitable structures.

In order to determine the presence/absence of an active fault trace extending through the area of proposed construction, this office excavated and logged a one hundred sixty-six foot (166') long seismic trench across the west side, i.e. the building pad area, of the subject property. The trench was excavated beyond the footprint of the proposed structure. In order to clearly evaluate the potential for active fault splays extending through the proposed area of construction, the seismic trench was excavated in a roughly north-south direction through the proposed area of construction. For reference, a detailed log of this excavation is presented on the attached Seismic Trench ST 01, Plate C.

Careful examination and detailed logging of the trench reveled no evidence (i.e. offsets, fault splays, etc.) of active faulting. Shearing was not observed along the terrace/bedrock contact in the seismic trench, which dipped north roughly parallel to the dip of the bedding within the bedrock beneath the contact. In addition, detailed mapping and examination of the terrace/bedrock contact, where exposed in the road cut along Kagel Canyon Road to the east, did not reveal any evidence suggesting the presence of active faulting extending through the proposed area of residence construction. Therefore, it is the opinion of this office that there are no active fault splays extending through the area of proposed construction.

Seismic Hazard Zones

Following the 1989 Loma Prieta Earthquake, the State of California enacted the Seismic Hazard Mapping Act (SHMA) in 1990. As a result, the California Geological Survey (a.k.a. Department of Conservation) prepared a set of maps designating areas within the state that may be susceptible to seismic slope instability and/or liquefaction during a strong seismic event. The seismic safety zones were published in a series of maps initially released in 1996.

The Seismic Hazards Mapping Act was prompted by damaging earthquakes in northern and southern California, and is 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 the various "seismic hazards zones." The maps depicting the zones are released by the California Geological Survey (CGS). The fact that a site lies outside of a zone does not mean it is free of seismic or geologic hazards such as landslides, rockfall, liquefaction or lateral spreading. Southern California has not been completely mapped, although new maps are issued and existing maps are refined occasionally.

The Seismic Hazards Mapping Act requires a site investigation by a certified Engineering Geologist and/or Civil Engineer prior to development of a project 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 factors, including: surface distribution of soil deposits and bedrock, slope steepness, depth to groundwater, bedding orientation with respect to slopes, bedrock shear strength, and occurrence of past seismic failure. Maps within the series are further designated as Reconnaissance, Preliminary or Official. Official Seismic Hazard Zone Maps are the culmination of mapping, analysis, review and comment of California Geological Survey, other State agencies, and the public following review and revision of the Preliminary Review Map. The Official Maps are the most rigorous and have the highest confidence level.

As defined, a "Liquefaction Hazard" area is an area where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693© would be required. As defined, an "Earthquake-Induced Landslide" area is an area where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693© would be required.

This office has reviewed the Seismic Hazards Maps of the Sunland and San Fernando Quadrangles prepared by the California Geological Survey (a.k.a. State of California Division of Mines and Geology). According to these maps, the site is not located within an area of study for earthquake-induced liquefaction However, the site is located within an area subject to earthquake-induced landsliding.

Recent Seismic Activity

The most recent largest earthquake within the specified search radius and time period is the Northridge Earthquake. The 6.7 magnitude Northridge Earthquake occurred on January 17, 1994 at 4:31 a.m., PST, and created strong ground shaking for approximately 10 seconds in the Los Angeles area resulting in wide spread, random damage. The earthquake occurred along a previously unrecognized south dipping thrust fault. The causative fault, as defined by a pattern of aftershocks, moved under an area roughly 19-miles across its front (approximately east-west in orientation) and 13 miles from front to back (approximately north-south in orientation). Although slip magnitude was approximately 9- to 10-feet, surface rupture along the causative fault did not occur as a result of the earthquake.)

Active Faults with Historic Surface Rupture

The 1857 Fort Tejon Earthquake (8.3±M) occurred along the San Andreas Fault Zone. The San Andreas Fault Zone, located northerly of the property, has a maximum probable event magnitude of 7.8. The 1933 Long Beach Earthquake (6.3M) occurred along the Newport-Inglewood Fault Zone. The Newport-Inglewood Fault, located southerly of the property, has a maximum probable event magnitude of 6.9. The 1971 San Fernando Earthquake (6.4M) occurred along the San Fernando Fault Zone. The San Fernando-Sierra Madre Fault Zone, located just south of the property, has a maximum probable event magnitude of 6.7.

CBC Seismic Design Parameters

The majority of southern California, including all of Los Angeles and Ventura counties, is within a zone requiring structural design to resist earthquake loads. The spectral acceleration parameters and site coefficients can be determined using the program "U.S. Seismic Design Maps" obtained from the U.S. Geological Survey website (http://geohazards.usgs.gov/designmaps/us/application.php).

The following seismic coefficient values were obtained from use of the U.S. Seismic Design Maps utilizing the design code reference document (2010 ASCE 7 w/2013 errata). It is recommended that the project Structural Engineer independently verify the accuracy of all the parameters outlined in the following table, excluding Site Class, prior to use.

California Building Code (Section 1613) - Site Coefficients & Data	
Site Latitude	34.2846°
Site Longitude	-118.3758°
Site Class (average soil properties within the upper 100')	C
0.2 Second Spectral Response - S_s	2.802
1.0 Second Spectral Response - S_1	1.011
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter	
$S_{MS} = F_a S_s$ (0.2 Second Spectral Response)	2.802
$S_{M1} = F_v S_1$ (1.0 Second Spectral Response)	1.315
Five Percent Damped Design Spectral Response Acceleration	
$S_{DS} = \frac{2}{3} S_{MS}$ (0.2 Second Spectral Response)	1.868
$S_{D1} = \frac{2}{3} S_{M1}$ (1.0 Second Spectral Response)	0.876

Conformance with the presented criteria for seismic structural design does not constitute any kind of warranty, guarantee, or assurance that significant damage, or ground failure, will not occur in the event of a maximum level earthquake. The primary goal of the code-required minimum seismic design is to protect life and limb, and catastrophic failure, and not avoid all damage, as such design may be economically prohibitive. The Project Structural Engineer and owner must decide if the level of risk associated with utilizing the minimum required code values is acceptable and, if not, assign appropriate seismic values above the minimum code values for use in the structural design.

Peak Ground Acceleration, Magnitude and Distance

The earthquake magnitude and distance to the fault was determined using the USGS 2008 Interactive Deaggregations program with an exceedance probability of 10% in 50 years. The peak ground acceleration was determined as $S_{DS}/2.5$. The value of S_{DS} was obtained from the "U.S. Seismic Design Maps" from the U.S. Geological Survey website as discussed above.

Fault Values	
Peak Ground Acceleration	0.747 g
PGA_M	1.074 g
Magnitude	6.69
Distance	6.07 km

ON-SITE WASTEWATER TREATMENT SYSTEM

The proposed residence is to be serviced by an on-site wastewater treatment system (OWTS) consisting of a septic tank and seepage pit(s). Upon the drilling and logging of future percolation test borings and completion of percolation testing, this office will address the construction of a site-specific OWTS in a supplemental report to be submitted to the County of Los Angeles under separate cover.

SITE STABILITY

Slope Stability

Slopes were analyzed for stability on the basis of cross-sections prepared by this office. It is our opinion that these cross sections represent the most critical conditions.

The terrace (Qt and Qty) deposits underlying the building site in the near-surface are generally sub-horizontal and thick-bedded to massive, and lack well-developed bedding structure.

Underlying the terrace deposits is well-bedded, sedimentary bedrock assigned to the Modelo Formation (Tm). Site bedrock exhibits east-west to northwest strikes accompanied by dips to the north-northeast ranging from 25° to 43°. In bedded formations, the degree and direction of bedding plane dip are critical to site stability. Sliding can occur along bedding planes that are inclined out of graded cut or natural slopes in an unsupported manner. As depicted on the attached Geologic Cross-Sections (Plate B) the north declination of bedding plane dip is neutrally-oriented with respect to the east-facing descending slope, a condition favorable for continued gross bedrock stability of the site.

The referenced geologic maps prepared by CDMG (1974) and Dibblee (1991) do not depict landslides within or immediately adjacent to the subject property that would adversely affect the stability of the site. Large-scale landslides were not observed by this office during field mapping and reconnaissance of the area.

Saturated shear strength parameters are based on laboratory tests performed on samples selected which, in the opinion of the Soils Engineer, represent the existing conditions at the site. Calculations for slope stability are based upon the ultimate shear strength values of the tested material.

Slope stability analysis on Section B-B' calculates a factor of safety in excess of the code required 1.5. Further, seismic slope stability analysis calculates a factor of safety in excess of the code required 1.1 (refer to APPENDIX III for calculations).

The potential for future erosion and soil slippage exists for all hillside developments. Therefore, it is imperative that pad and slope drainage be controlled in a manner that will prevent water from flowing over the slope areas in a concentrated or uncontrolled manner. Refer to the DRAINAGE AND MAINTENANCE section below for additional information. Additionally, it is recommended that slope area(s) be planted and maintained with erosion retardant ground cover adhering to the following criteria:

- is effective in preventing surface erosion;
- is drought resistant;
- has a relatively low surface mass weight;
- has a fairly deep and extensive root system;
- requires a minimum of maintenance by owner;
- has a low irrigation demand.

It requires approximately 2 to 3 years before an adequate erosion-retardant ground cover can become established on a slope. It is recommended that you consult with a landscape architect to determine specific botanicals that will serve as an effective erosion-retardant ground cover for your area.

Liquefaction Potential

Liquefaction refers to the momentary loss of shear strength. The necessary components for liquefaction include: a shallow groundwater condition; relatively loose soils; fine grained sands and silty sands; and repeated cyclic loading. During an earthquake cyclic loading occurs, allowing pore pressures to increase as a result of individual soil grain particles realign themselves. The realignment of the soil particles allows the water to completely separate and surround the grains. As cyclic loading continues the shear resistance of the soil decreases until the pore pressures equal the confining pressures. The result of the increases in the pore pressure and the decrease in the shear resistance is termed "Liquefaction".

A cursory review of maps contained within our offices indicates that the subject property is not mapped within a special studies zone for seismic induced liquefaction. Therefore, an analysis of the potential for earthquake inducted liquefaction has not been performed as the potential for liquefaction to occur at the subject property is considered to be low. This satisfies the requirement of the State of California Public Resources Code, Section 2690 et seq. (Seismic Hazard Mapping Act). Additionally, the subject property is underlain by bedrock. Therefore, the potential for liquefaction to occur at the subject property is remote.

Expansive Soils

Expansive soils are considered to be one of the most costly natural hazards as related to light structures, slabs, retaining walls, paving etc. Expansive soils are influenced greatly by changes in moisture content and can lead to damage when the moisture content changes significantly over short durations of time (i.e. seasonal).

These changes can result from many factors including the initial moisture content of the soil, climate, groundwater, drainage conditions, irrigation and vegetation to name a few. Therefore, it is imperative that soils underlying the subject property are maintained at a consistent moisture content in order to reduce the potential damage caused by expansive soils. A watering schedule that allows the soil to become saturated, then dry out can result in foundation movement and distress. In addition, the recommendations outlined in the DRAINAGE AND MAINTENANCE section should be followed.

Based upon our investigation and laboratory testing the subject site is underlain by soil in the low expansive range. All foundations and slabs should be designed for expansive conditions. To mitigate the effects of expansive soils, good site drainage should be maintained at all times. Roof gutters and downspouts should be incorporated in the design and construction of the structure. Planters should not be placed near existing or future foundations. Existing planters should be equipped with concrete sides and bottoms. Planters should be connected to a drainage system to convey water away from the foundations. Utility pipes should be checked periodically for leaks.

CONCLUSIONS

General

It is the professional opinion of this office that construction of the proposed single family residence is feasible provided that the recommendations contained herein are followed. In addition, all applicable elements of the governing agency Building Codes shall be followed.

Based upon our field observations, laboratory testing and analysis, the terrace deposits found in the explorations should possess sufficient strength to support the single family residence and associated improvements. The artificial fill, Slopewash and natural soil deposits encountered in site explorations are not considered suitable for foundation support as these materials may possess adverse deformational characteristics.

Excavation Characteristics

Subsurface exploration was performed through the use of a backhoe excavating into the underlying soils. Excavating into the underlying earth materials during construction should be possible with conventional excavating equipment.

RECOMMENDATIONS

1. Grading will consist of excavations into the hillside for the proposed residence. The earth material generated during grading shall be placed as certified compacted fill or wasted off site. If the fill will be placed on site, the location shall be provided to this office and additional recommendations will be provided as necessary.

2. The proposed single family residence shall be supported by foundations extending into the underlying terrace deposits. Portions of the driveway extension will be supported by foundations extending into the underlying bedrock. Based upon the depth to bedrock and the code-required setback for structures planned on or adjacent to descending slopes, deepened foundations should be anticipated. Foundations should be designed as outlined in the FOUNDATIONS section below.

3. Excavations that remove lateral support from offsite properties, existing structures or where trimming the earth material to a 1:1 (h:v) gradient is not feasible, will require temporary shoring. Temporary shoring may consist of slot cutting or shoring piles. Offsite properties are considered unsupported when a 1:1 (h:v) plane projected up from the base of the excavation extends onto the adjacent property. Where the 1:1 (h:v) plane projects onto the ground surface within property lines, offsite properties are not considered unsupported. All excavations shall be made as outlined in the EXCAVATIONS section below.

4. Retaining walls will be required for the proposed improvements shall be designed and backfilled as outlined in the RETAINING WALLS section below.

5. The site shall be maintained as outlined in the DRAINAGE AND MAINTENANCE section below.

It should be noted that, the recommendations contained within this report may be more restrictive than applicable building codes. All recommendations of this report which are in addition to or more restrictive than those outlined in a subsequent review letter, by your governing reviewing agency, shall be incorporated into the plans.

FOUNDATIONS

It is recommended that the proposed single-family residence be supported by foundations extending into the underlying terrace deposits. Portions of the driveway extension will be supported by foundations extending into the underlying bedrock. Based upon the depth to bedrock and the code-required setback for structures planned on or adjacent to descending slopes, deepened foundations should be anticipated.

Foundation excavations shall be cleared of all loose material prior to the placement of steel, then prior to the placement of concrete to reduce the potential for future settlement and differential settlement. Water shall not be allowed to pond or drain into or through the footing trench excavations. All foundations shall maintain the required code setback from descending slopes.

The Code requires foundation setbacks for construction adjacent to slopes. The Code requires deepened foundations for building planned on or adjacent to **descending** slopes steeper than 3:1 (h:v). The setback corresponds to a horizontal distance equal to one-third of the vertical height of the slope, with a minimum distance of five feet (5') and a maximum distance of forty feet (40'). This distance is measured from the slope face to the lowest edge of the footing.

All earth materials derived from the excavations of foundations shall be removed from the site or placed as certified compacted fill. Fill temporarily stockpiled on site should be placed in a stable area, away from slopes, excavations and improvements. Earth materials shall not be cast over any descending slopes in an uncontrolled manner.

Conventional

The minimum continuous footing size is twelve inches (12") wide for one story structures and fifteen inches (15") wide for two story structures. Pad foundations shall be a minimum of twenty four inches (24") square. All depths of embedment for footings are to be measured from the lowest adjacent grade or into the specified bearing material.

Foundation Design Values						
Foundation Type	**Bearing Material**	**Embedment Depth**	**Bearing Value (psf)**	**Maximum Bearing Value (psf)**	**Coefficient of Friction**	**Passive Resistance (pcf)**
Continuous	Bedrock	18"	3000	5000	0.40	400
Pad	Bedrock	18"	3500	6000	0.40	400
Continuous	Terrace Deposits	24"	2000	2700	0.30	300
Pad	Terrace Deposits	24"	2500	3300	0.30	300

Increases in the bearing value are allowable at a rate of twenty percent (20%) for each additional foot of footing width or depth into the recommended bearing material to the maximum bearing value.

The depths specified in the above table are minimum embedment depths required by this office. Deeper foundations may be required during the construction phase of the project due to the presence of unconsolidated soil, uncertified fill or weathered bedrock. Additionally, the project Structural Engineer may need to make the depths deeper to accommodate specific structural loads. The bearing values given above are net bearing values; the weight of concrete below grade may be neglected. These bearing values may be increased by one-third (⅓) for temporary loads, such as wind and seismic forces.

Based upon past experience, all continuous footings shall be reinforced with a minimum of four #4 bars, two placed near the top and two near the bottom. Reinforcing recommendations are minimums and may be revised by the structural engineer.

Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the bedrock or terrace deposits. The coefficient of friction shall be used between the base of the foundation and the recommended bearing material. When combining passive and friction for resistance of lateral loads, the passive component should be reduced by one-third. For isolated poles, the allowable passive earth pressure may be doubled.

All footing excavation depths will be measured from the lowest adjacent grade of recommended bearing material. Footing depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth into the recommended bearing materials will not be acceptable to this office.

Friction Piles

The minimum friction pile diameter is twenty four inches (24"). All friction piles shall extend into the terrace deposits for the single-family residence and into the bedrock for portions of the driveway. Friction piles shall extend a minimum of seven feet (7') into the recommended bearing material, and not less than what will be required to support loading conditions. The friction piles may be proportioned using a skin friction value of 400 and 650 pounds per square foot of shaft exposed to the terrace deposits or bedrock, respectively. Further, friction piles shall be considered fixed at an embedment depth of two feet (2') into the recommended bearing material. The skin friction values may be increased by one-third (⅓) for temporary loads, such as wind and seismic forces. All friction piles should be designed to resist a creep force of 1000 pounds per lineal foot for the upper three feet of shaft exposed to the earth material above the recommended bearing material.

Lateral Load Design (Friction Piles)

Lateral loads may be resisted by passive resistance within the terrace deposits or bedrock. The passive resistance may be assumed to act as a fluid with a density of 300 and 400 pounds per cubic foot for the terrace deposits and bedrock, respectively. A maximum passive earth pressure of 3500 and 5000 pounds per square foot may be assumed for the terrace deposits and bedrock, respectively. For isolated poles, the allowable passive earth pressure may be doubled. Piles are considered isolated where spaced more than 2.5 times the diameter of the piles.

Raised Floor Construction

Construction utilizing raised floors where the grade under the floor is lowered for joist clearance often leads to moisture problems. Surface moisture can seep through or migrate beneath footings and pond in the lowered underfloor area. The problem increases with increasing difference between the interior and exterior grades. Excessive moisture accumulation or ponding water in the underfloor area can lead to warping or cupping of wood floors. Further, consistent moist conditions can lead to the growth of wood destroying fungus, rotting of wood framing elements, and/or mold growth.

Due to the potential problems discussed above, SubSurface Designs, Inc., does not recommend the use of this construction technique. Should you decide to disregard the advice presented herein, positive drainage of the ground surface away from the footings, waterproofing the footings, sealing of utility line penetrations through footings, compaction of trench backfill, placement of foundation drains and the placement of planter drains can help to reduce moisture intrusion. Planters which are not sealed and drained should not be used adjacent to any structures. Subdrains placed directly adjacent to footing stemwalls are beneficial but will generally not completely prevent water from migrating beneath foundations. Lined planters with drains that are located away from the footings and extend deeper than the footings are generally the most effective mitigation technique.

Adequate ventilation of the underfloor area is also critical in preventing high moisture conditions below proposed structures. Creating adequate ventilation is difficult, particularly in larger homes with interior continuous footings. Telescoping vents are generally ineffective, particularly if provided with louvered covers. Consideration should be given to providing more than the minimum Code-required amount of vent space. Mechanical ventilation may be necessary, particularly in larger homes.

SETTLEMENT

Settlement and differential settlement can result in cracks in the exterior and interior finishes, flooring, etc. Controlling drainage around the structure as outlined in the DRAINAGE AND MAINTENANCE section below can help to control settlement/differential settlement. Additionally, foundation excavations cleared of all loose material prior to the placement of steel, then prior to the placement of concrete is required and can significantly minimize future settlement and differential settlement.

Future settlement and/or differential settlement of the structure and secondary features due to long term deformation and natural occurrences are still possible. Any site drainage improvements, such as those outlined in the DRAINAGE AND MAINTENANCE section below, will result in a lower risk of future foundation problems.

Settlement of the proposed single-family residence will occur. Settlement of ½" to ¾" between walls, within 30 feet or less of each other and under similar loading conditions, are considered normal. Total settlement on the order of ¾" should be anticipated. Differential settlement is not expected to exceed ¾".

FLOOR SLABS

Floor slabs should be a minimum of four inches (4") thick, reinforced with minimum #3 reinforcing bars placed at sixteen inches (16") on center each way. Floor slabs underlain by two inches (2") of crusher-run base, compacted into place by mechanical means may be supported directly on terrace deposits. Although precautions can be taken, the recommendations are not intended to stop movement, only to reduce cracking as a result of expansion and contraction of the soil.

Two-car garage slabs should be quartered with weakened plane joints and isolated from the stem walls for crack control. The slab should maintain positive drainage toward the entry of the garage.

Residential floor slab should be protected by a 10 mil vapor retarder/barrier placed beneath the slab. These types of retarders/barriers should be protected to prevent punctures in the vapor retarder/barrier. It should be noted that this type of barrier will not preclude moisture damage to wood floors or vapor sensitive flooring. The commonly used 6-mil and 10-mil polyethylene plastic sheeting can produce less-than-satisfactory results due to its low puncture resistance, inconsistent vapor permeability, and variable product longevity. It is recommend that the retarder/barrier conform with ASTM E1745 Class A and be installed in accordance with ASTM E1643. In particular, care should be utilized to seal sheet boundaries and seal around penetrations.

It should be noted that cracking of concrete floor slabs is very common during curing. The cracking occurs because concrete shrinks as it dries. It is important that additional water not be added to concrete at the site to make pumping easier as this will increase the magnitude of shrinkage.

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 concrete slab 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 stone or tile. A mortarbed or slip sheet is recommended between the slab and brittle floor covering to limit the potential for cracking.

Footing trench spoils should either be removed from the slab areas or compacted into place by mechanical means and tested for compaction.

EXCAVATION EROSION CONTROL

During inclement periods of the year, when rain is threatening (between October 1, and April 15), an erosion control plan shall be implemented and approved by the reviewing agency to reduce the potential of site erosion. The following are several recommendations prepared by this office. The following recommendations are valid for any time of the year that rain threatens an excavation.

Open Excavations

All open excavations shall be protected from inclement weather. This is required to keep the surface of the open excavation from becoming saturated during rainfall. Saturation of the excavation may result in a relaxation of the soils which may result in failures.

Hillside Excavations

All hillside excavations shall be covered during the rainy seasons. Stakes, ropes, and sandbags, along with plastic may be employed to help facilitate the coverage of the excavations. Coverage of the open excavations shall over-extend from the edges of the excavations in all directions.

The project Civil Engineer shall be consulted for the limits of coverage. If possible, slopes around the open excavations shall be trimmed to slope away from the open excavation, so water runoff will not drain into the excavation. Any trees or planters that might cause failure around the open excavations, due to the saturated hillside, shall be anchored safely. After the rain has ceased, the excavations shall be reviewed by the project soil engineer and geologist for safety prior to recommencement of work.

Open Trenches

No water shall be allowed to pond or saturate open trenches. All open trenches shall be covered with plastic and sandbags. Areas around trenches shall be sloped in such a way that water will not runoff into the trenches. After the rain has ceased, trenches shall be reviewed by project soil engineer for safety prior to recommencing work. All footing excavations must be reviewed by the project soil engineer again, prior to pouring concrete.

Open Pile/Caisson Excavations

All open excavations for piles or caissons shall be reviewed and poured prior to rainfall. We do not recommend any pile excavations being left open through any rain storms. However, if it is necessary to leave pile excavations open through any rain storms, all water and runoff must be prevented from entering these pile excavations.

Grading in Progress

During the inclement part of the year, or during periods when rain is threatening, all fill that has been spread and awaits compaction shall be compacted before stopping work for the day or before stopping because of inclement weather. These fills, once compacted, shall have the surfaces sloped to drain to one area where water may be removed.

Work may start again, after the rainy period, once the site has been reviewed by the project soils engineer. Loose materials not compacted prior to the rain shall be removed and aerated so that the moisture content of these fills will be within three percent (3%) above the optimum moisture content.

Surface materials previously compacted before the rain, shall be scarified, brought to the proper moisture content, and re-compacted prior to placing additional fill, if deemed necessary by the Soils Engineer.

Additionally, it is suggested that all stock-piled loose fill materials, not compacted prior to anticipated rainfall, shall be covered with plastic. This action will keep the loose fill from being saturated with water, and will allow the grading to resume when the rain stops. It is always easier and less time consuming to increase moisture content of the fill than to aerate the fill to achieve optimum moisture.

All of the above recommendations shall be considered as part of the erosion control plan for the subject property. However, these recommendations shall and will not supersede, nor limit any erosion control plans produced by the Project Civil Engineer.

EXCAVATIONS

Excavations ranging in vertical height up to sixteen feet (16') will be required for the proposed development. Conventional excavation equipment may be used to make these excavations. Excavations should expose natural soil, earth fill and/or terrace deposits. The natural soil, earth fill or terrace deposits are suitable for unsurcharged vertical excavations up to five feet (5'). Excavations that exceed the stated height limits, and all loose surficial material, shall be trimmed back at a gradient of 1:1 (h:v). This should be verified by a representative of this office during construction so that modifications can be made if variations in the soil occur. The earth material exposed in the proposed cuts should be kept moist, but not saturated, to reduce the potential for raveling and sloughing that may occur during construction.

Vertical excavations along property lines, existing structures or where trimming the earth material to a 1:1 (h:v) gradient as outlined above is not feasible, will require temporary shoring. Temporary shoring may consist of slot cutting or shoring piles.

Vertical excavations that remove lateral support from offsite properties, existing structures or where trimming the earth material to a 1:1 (h:v) gradient as outlined above is not feasible, will require temporary shoring. Temporary shoring may consist of slot cutting or shoring piles.

Offsite properties are considered unsupported when a 1:1 (h:v) plane projected up from the base of the excavation extends onto the adjacent property. Where the 1:1 (h:v) plane projects onto the ground surface within property lines, offsite properties are not considered unsupported.

The slot-cutting method employs the use of the earth as a buttress and allows the excavation to proceed in phases and shall be limited to eight feet (8') in height. The initial excavation is made at a slope of 1:1 (h:v). Slots are cut, using the ABC method, in which all slots are of the same width. The initial slot "A" is cut eight feet (8') in width, leaving the "B" and "C" slots to buttress the excavation. The footing and wall is constructed in the "A" slot, cured, braced, and backfilled; the same procedure is used for the "B" slots; then the "C" slots.

The vertical excavations for the proposed development can be made following the installation of shoring piles. The shoring shall be designed for an equivalent fluid pressure of twenty pounds per cubic foot (20 pcf) for the temporary condition. If the shoring piles will be included in the permanent design, the equivalent fluid pressure values presented in the RETAINING WALL section below shall be utilized. Design values do not include vehicular or structural surcharge. Refer to APPENDIX III for calculations. Shoring piles should be placed six feet (6') edge to edge. Lagging between the shoring piles is not anticipated at this time. However, if it is determined that lagging is required to support the existing soil, uncertified fill, the contractor shall be prepared to install lagging. Lagging should be designed for a maximum pressure of four hundred pounds per square foot (400 psf).

The minimum shoring pile diameter is eighteen inches (18"). All shoring piles should extend into the terrace deposits a minimum of ten feet (10'), and not less than what will be required to support loading conditions. The shoring piles may be proportioned using a skin friction value of 400 pounds per square foot of shaft exposed to the terrace deposits. The skin friction values may be increased by one-third (⅓) for temporary loads, such as wind and seismic forces. Further, shoring piles shall be considered fixed at an embedment depth of two feet (2') into the recommended bearing material.

Lateral loads may be resisted by passive resistance within the terrace deposits. The passive resistance may be assumed to act as a fluid with a density of 300 pounds per cubic foot. A maximum passive earth pressure of 3500 pounds per square foot may be assumed. For isolated poles, the allowable passive earth pressure may be doubled. Piles are considered isolated where spaced more than 3 times the diameter of the piles.

Construction excavations shall be made under the supervision of a qualified "competent person" along with periodic review performed by this office. A "competent person" as defined by California/OSHA, is one who is capable of identifying existing and predictable hazards that are unsanitary or dangerous to employees. The competent person has the authority to impose prompt corrective measures to eliminate these hazards.

All excavations should be stabilized within 15 days of initial excavation. If this time is exceeded, the project soils engineer must be notified, and modifications, such as shoring or slope trimming may be required. Water should not be allowed to pond on top of the excavation, nor to flow toward it. All excavations should be protected from inclement weather. The top of the excavations should be barricaded to ensure that no vehicular surcharge be allowed within five feet (5') of the top of cut.

All other construction methods shall meet the requirements of the Construction and General Industry Safety Orders, the Occupational Safety and Health Act, California OSHA in addition to other public agencies having jurisdiction.

RETAINING WALLS

Cantilever Walls

Retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill. The active earth pressure would normally be taken as equivalent to the pressure exerted by a fluid weighing thirty pounds per cubic foot (30 pcf) for the proposed retaining walls. However, design loads for the additional earth pressure caused by seismic ground shaking are required, refer to the Seismic Design Loading section below. Based on these calculations the seismic design load for cantilevered walls is fifty seven pounds per cubic foot (57 pcf). Stability calculations have a factor of safety in excess of 1.5 for a sixteen foot (16') high excavation. Therefore, it is recommended that the seismic earth pressure be utilized for design over the active pressure.

The active earth pressure should be taken as equivalent to the pressure exerted by a fluid weighing fifty seven pounds per cubic foot (57 pcf) for retaining walls up to sixteen feet (16') in height. The resultant force of the static active load acts at ⅓H above the base of the wall, where H=wall height.

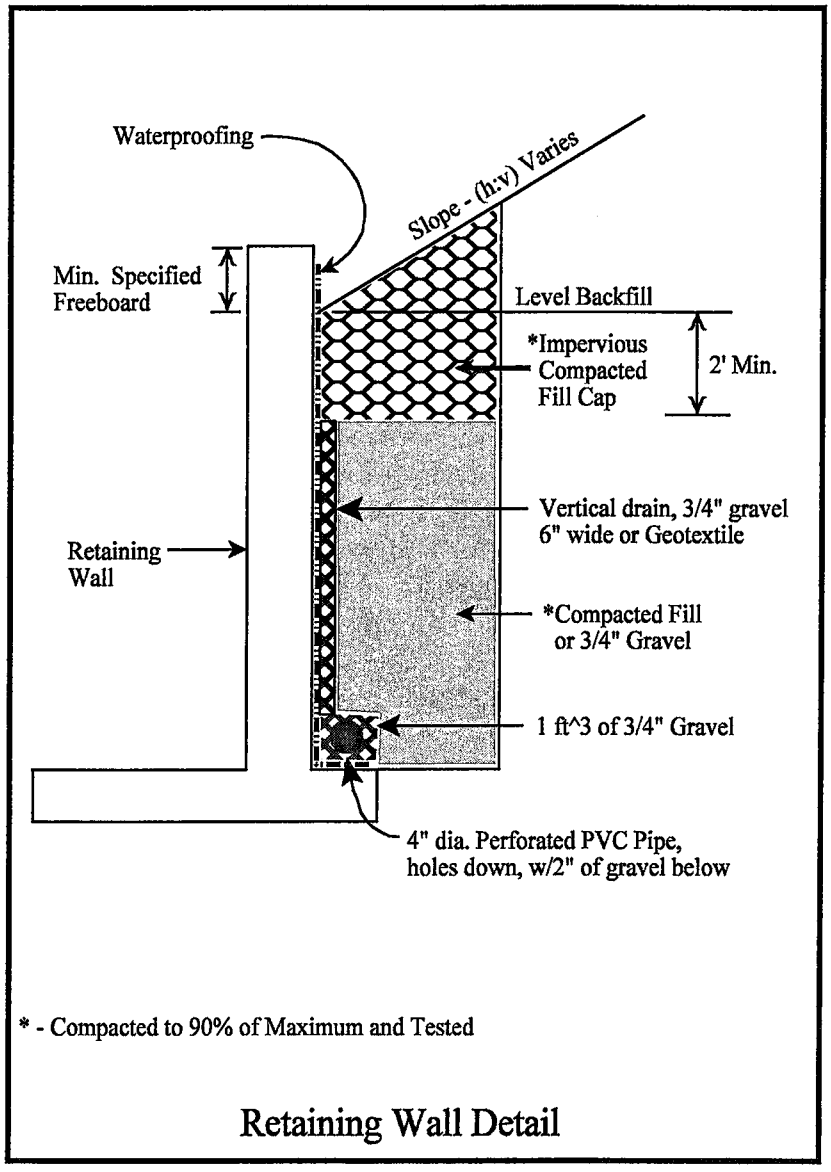
The wall pressure stated above assumes:

1. The wall has been backfilled with non-expansive soils, compacted to 90 percent of the maximum density and tested as outlined in the wall backfill section of this report.

2. The grade behind the wall is level, and there is no structural surcharge.

3. Adequate drainage is provided behind the wall to minimize the buildup of hydrostatic pressures.

   a. A perforated pipe, with perforations placed down, shall be installed at the base of the wall footing. The pipe shall be encased in at least one foot (1') of three-quarter inch (¾") gravel. All drainage from this pipe should be transferred to a approved drainage area via non-erosive devices approved by the governing agency.



Retaining Wall Detail

   b. The back of all retaining walls and/or basement walls shall be waterproofed and a continuous vertical drain shall be placed on the backside of the wall. The presence of water adjacent to retaining walls and/or basement walls may lead to moisture intrusion through the retaining walls and/or into the basement level.

Therefore, it is recommended that the proposed retaining walls and/or basement walls be waterproofed in a manner that will preclude the infiltration of water through retaining walls or into the basement level. This will require the employment of a competent waterproofing specialist/contractor that specializes in this type of work.

A concrete-lined swale should be placed behind the wall that can intercept surface runoff from up slope areas. This surface runoff shall be transferred to the street via non-erosive devices, or an approved drainage area approved by the governing agency. A minimum freeboard of eight inches (8") shall be maintained at all times. Any slough, debris or trash that accumulates behind the wall should be removed immediately.

All excavations shall be reviewed by this office to ascertain if there are any conditions encountered that are different from those observed in the explorations and modeled by the calculations. If changes are observed additional recommendations will be made at that time. All excavations must be stabilized within fifteen days (15) or less.

Foundation design parameters, as given in the preceding section, may be used for retaining walls. All loose material shall be cleaned from the foundation excavations. Water shall not be allowed to pond or drain into or through the footing trench excavations. Proper compaction of the backfill is recommended to provide lateral support to adjacent properties.

Basement/Restrained Walls

Retaining walls where the top is restricted from movement by structural elements such as a concrete slab or stiff framing the walls should be designed as for the "at rest" lateral earth pressure pursuant to section 1610.1 of the building code. Basement walls should be designed to resist a triangular distribution of lateral earth pressure. The "at rest" lateral earth pressure will be ninety two pounds per cubic foot (92 pcf). The retained condition for a restrained retaining wall with level backfill is:

$$\Delta P_{ae} = \tfrac{1}{2}\gamma H^2(0.68PGA_M/g)$$

In addition to lateral earth pressure, this wall should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjacent traffic surcharge. The pressure indicated assumes that the wall will be designed with a proper backdrain system and therefore does not include hydrostatic pressure.

All required backfill adjacent to subterranean walls should be compacted to at least 90 percent of the maximum density or backfilled with gravel. Proper compaction of the backfill is recommended to provide lateral support to adjacent properties. Even with proper compaction of required backfill, settlement of the backfill may occur because of the significant depth of the backfill. Accordingly, utility lines, footings, or false work should be planned and designed to accommodate such potential settlements. All drainage requirements listed in the RETAINING WALL section shall apply.

Seismic Design Loading

The California Building Code requires that retaining walls be designed for the additional earth pressure caused by seismic ground shaking. The PGA_M value was obtained from the U.S. Geological Survey website. In determining the additional earth pressure that could result from a seismic event, a horizontal acceleration equal the PGA_M was utilized. Calculations were determined utilizing the latest County of Los Angeles guidelines. The retained condition for an unrestrained cantilever with level backfill is:

$$\Delta P_{ae} = \frac{1}{2}\gamma H^2(0.42PGA_M/g)$$

Calculations indicate a seismic earth pressure of 7217 lb per foot of wall. This equates to an equivalent fluid pressure of fifty seven pounds per cubic foot (57 pcf).

As discussed above, the active earth pressure should be taken as equivalent to the pressure exerted by a fluid weighing fifty seven pounds per cubic foot (57 pcf) for retaining walls up to sixteen feet (16') in height. The resultant force of the static active load acts at ⅓H above the base of the wall, where H=wall height.

Retaining Wall Deflection

It should be noted that non-restrained retaining walls designed for active earth pressure will deflect ¼ to ½ percent of their height over time in response to this loading. The deflection is normal and reduces the earth pressure on the wall. The deflection will also result in the lateral movement and settlement of the retaining wall backfill. Improvements constructed immediately adjacent to, or incorporated into non-restrained retaining walls, should be designed to accommodate this movement. Curved or angled walls which have a convex, downslope plan pattern should be provided with vertical construction joints at corners and forty feet (40') on center.

Decking which caps a retaining wall should be provided with a flexible joint to allow for the normal ¼ to ½ percent deflection of the retaining wall. Decking that abuts a retaining wall should not be tied to the wall. The space between the wall and the deck should be caulked and will require periodic caulking to prevent water intrusions into the retaining wall backfill or supporting bearing material.

Wall Backfill

1. Walls to be backfilled must be reviewed by the project Soils Engineer prior to commencement of the backfilling operation or placement of the wall backdrain system.

2. After the wall backdrain system has been placed, the back side of the wall has been waterproofed and reviewed by this office, fill may be placed in layers not exceeding four inches (4") in thickness and compacted to 90 or 95 percent of the maximum density, as determined by the latest version of ASTM D 1557. The higher compaction is required for fill material that has less than fifteen percent (15%) of the material finer than 0.005mm.

3. If the wall backfill consists of a granular free-draining material, a vertical gravel blanket at the face of the wall, or similar vertical drainage system, will not be required.

4. If the onsite soils are used for wall backfill, and they have an expansion index of 30 or greater, a vertical gravel drain blanket four to six inches (4"-6") thick, or a manufactured drainage system such as MiraDrain® or similar, along the back side of the wall from top to bottom is required.

5. Where space does not permit compaction of material behind the wall, a granular backfill may be used. The granular backfill shall consist of one-half inch (½") to three-quarter inch (¾") of crushed rock placed in two foot (2') lifts and vibrated. The vibrated rock shall be verified by a representative of this office. Where the backcut excavation is more than eighteen inches (18") from the back of the retaining wall, gravel backfill is not permitted.

6. All granular free-draining wall backfills shall be capped with a clayey compacted soil within the upper two feet (2') of the wall for a depth of two feet (2'). This compacted material should start below the required wall freeboard.

7. Where slopes are steeper than 5:1 (h:v) benching shall be required into competent materials as determined by this office, in the field, at the time of grading. Where benching is incorporated into to construction of a slope, backdrains as outlined in the GRADING AND EARTHWORK section will be required.

CONCRETE DECKING

Concrete decking should be a minimum of four inches (4") thick, reinforced with minimum #3 reinforcing bars placed at eighteen inches (18") on center each way. The decking should be underlain by two inches (2") sand to aid in concrete curing. It should be noted that cracking of concrete slabs and decking is very common during curing. The cracking occurs because concrete shrinks as it dries. It is important that additional water not be added to concrete at the site to make pumping easier as this will increase the magnitude of shrinkage and thus cracking.

For crack control in concrete decking, the maximum control joint spacing should be eight feet (8'). A closer control joint spacing would provide greater crack control. Additional control joints at 90° corners, curves, angle points and where decking transitions to a narrower segment are recommended.

Cracks in slabs or decking, and gaps between the structure and decking, that develop will require periodic caulking to limit infiltration of moisture into the underlying soil.

Decking that adjoins a lawn, planters or the top of a slope should be provided with a twelve inch (12") thick deepened edge. The deck reinforcement should be bent down into the thickened edge. Steel reinforcing bars (#3) should be provided at the top and bottom of the deepened edge. Although precautions can be taken, the recommendations are not intended to stop movement, settlement, differential settlement, or cracking. Recommendations provided are only to reduce movement, settlement and cracking as a result of expansion/contraction or consolidation of the supporting soil.

Footing trench spoils should either be removed from the slab areas or compacted into place by mechanical means and tested for compaction.

UTILITY TRENCHES

It is recommended that utility tenches are not planned or placed parallel to and below a 1½:1 plane projected down from the base of the outer edge of a conventional foundation. Footings should be deepened to satisfy the above recommendations. Clean sand should be placed around utility lines and properly jetted. Flooding and/or jetting of utility trenches does not create compact trench backfill and should be limited to backfilling around, and up to six inches (6") above, utility pipes. Backfill for the remaining portion of the trench above the pipes should be placed by mechanical compaction methods to a minimum of 90 or 95 percent of the maximum density, as determined by the latest version of ASTM D 1557. The higher compaction is required for fill material that has less than fifteen percent (15%) of the material finer than 0.005mm. The upper twelve inches (12") of the certified fill shall be compacted to at least 95 percent of the maximum density in all areas where vehicle loading occurs. All compaction should be tested and certified by this office. Failure to properly backfill and compact utility trenches can result in water migrating through the trench which could lead to foundation settlement or slope instability. Utility penetrations through footings should be tightly sealed when raised-floor construction is utilized.

DRAINAGE AND MAINTENANCE

General

Maintenance of structures and slopes must be performed to minimize the chance of serious damage and/or instability to improvements. Most problems are associated with, or triggered by water. Therefore, a comprehensive drainage system should be designed and incorporated into the final plans. In addition, pad areas should be maintained and planted in a way that will allow this drainage system to function as intended. The following are drainage, maintenance and landscaping recommendations that should be implemented at minimum.

Reductions in these recommendations will reduce their effectiveness and may lead to damage and/or instability to site improvements and adjacent properties. It is the responsibility of the property owner to ensure that the residence and site drainage devices are maintained in accordance with the following recommendations, including the requirements of applicable governing agency.

1. Pad Drainage

   (a) Positive pad drainage shall be incorporated into the final plans. All drainage from the roof and pad shall be directed so that water does not pond adjacent to the foundations or flow toward them. All drainage from the site shall be collected and directed via non-erosive devices to a location approved by the building official. **No alteration of this system shall be allowed.**

   (b) Planters placed adjacent to the structures shall be designed to drain away from the structure. Care should be taken to not saturate the soils (i.e. leaking irrigation lines or excessive landscape watering).

2. Slope Drainage

   No water shall be allowed to flow over any slopes. The drainage structures constructed to enhance slope stability shall be cleaned and/or maintained. This includes, but is not limited to, interceptor ditches, drainage terraces, downdrains, berms, debris fences, grates, and exits for subsurface devices.

3. Landscaping (Planting)

   It is recommended that a landscape architect be consulted regarding planting adjacent to the development and on any slopes. Plants surrounding the development shall be of a variety that requires a minimum of watering. Slope landscaping shall consist of plants with dense and deep root structures that require a minimum of watering. It will be the responsibility of the property owner to maintain the planting. Alterations of planting schemes shall be reviewed by the landscape architect.

4. Irrigation

   An adequate irrigation system will be required to sustain landscaping. Any leaks or defective sprinklers shall be repaired immediately. To mitigate erosion and saturation, automatic sprinkling systems shall be adjusted for rainy seasons. A landscape irrigation specialist should be consulted to determine the best times for landscape watering and the maximum amount of water usage.

5. Rodent Control

The property owner must undertake and maintain a program which eliminates or controls burrowing animals. This must be an ongoing program in order to provide protection to the slope's stability. The uncontrolled burrowing by rodents has proven to be one of the major causes for surficial slope stability problems.

REVIEWS

Plan Review and Plan Notes

The final construction and/or grading plans shall be reviewed and approved by the consultants. This is required to determine if the recommendations of the report have been properly understood and carried forth in the design drawings.

The final plans should reflect the following:

1. The Geologic & Soils Engineering Investigation by SubSurface Designs, Inc., as a part of the plans.
2. Plans must be reviewed and signed by this office.
3. All grading must be reviewed by a representative of this office.
4. All foundations shall be reviewed by a representative of this office.
5. All slot cutting must be performed under the continuous review of a representative of this office.
6. All pile foundation excavations must be performed under the continuous review of a representative of this office.
7. All shoring must be performed under the continuous review of a representative of this office.
8. All retaining wall backdrain systems and backfill shall be reviewed and tested by a representative of this office.
9. All utility trench backfill shall be reviewed and tested by a representative of this office.

Reviews will be required to verify all work. A review will be performed to determine if the intent of the report has been adequately carried forth. This office should be notified at least **two working days** in advance of any reviews of this nature so that staff personnel may be made available.

Construction Review

It is required that all grading, foundation excavations, slot cutting, pile foundations, underpinning, shoring, utility trench backfill, pre-moistened soil, retaining wall backdrain systems and backfill be reviewed by this office. A review will be performed to determine if the intent of the report has been adequately carried forth. This office should be notified at least two working days in advance of any reviews of this nature so that staff personnel may be made available. The reviews will be billed at our current hourly rate.

LIMITATIONS

General

Subsurface conditions were determined on the basis of our field explorations. Soil and bedrock conditions may vary in type, strength, depth and other properties between the exploratory excavations. Recommendations presented herein are for soil/bedrock conditions encountered in the specific locations explored. Non-uniformity of the soil/bedrock conditions or manmade alterations may be revealed during construction. This office will revise design recommendations if conditions are encountered which are different from the presented herein.

Conclusions on building site stability, settlement, slippage, and the developments affects on off-site property are based on visual examination, observations within the explorations, laboratory testing of samples obtained during exploration, analysis of data, and our experience. Our examination does not, however, imply that the subject property is risk free.

Conclusions and recommendations presented herein are based on our experience and background. The conclusions and recommendations presented herein are professional opinions and are not meant to indicate a control of nature. This report makes no other warranty, either expressed or implied, concerning the advice presented herein.

This report may not be copied. If you wish additional copies, you may order them from this office. See your contract for charges.

111 Statement

It is the finding of this firm, based upon the subsurface data, that the proposed single family residence and driveway will not be affected by settlement, landsliding, or slippage. Further, the proposed development and grading will not have an adverse effect on off-site property.

Construction Notice

Construction can be difficult. Recommendations contained herein are based upon several windows (explorations) excavated at locations deemed suitable by your consultants. It is this corporation's aim to advise you through this report of the general site conditions, suitability for construction, and overall stability. It must be understood that the opinions are based upon testing, analysis, and interpretation thereof.

All properties are subject to risk, these risks can be mitigated but not be eliminated. Properties are subject to hazards including but not limited to, floods, mudslides, landslides, seepage, erosion, raveling of slopes, concentrated drainage, limited access, differential settlement, heaving and fire. The damage from these hazards may be reduced by the property owner by maintaining yards, slopes, walls, slough protection devices, drainage facilities, and by correcting any deficiencies found during occupancy of the property. It is not possible to eliminate all hazards.

Quantities for foundation concrete and steel may be estimated, based on the findings given in this report. However, you must be aware that depths and magnitudes will most likely vary between the excavated windows (explorations) given in the report.

If you have any questions concerning this report, please contact this office.

Respectfully submitted:
SUBSURFACE DESIGNS, INC.

Jon Mahn
Project Engineer
RCE 60293

Ed Pongracz-Bartha
Engineering Geologist
CEG 2370

JEM/EPB/ap: 6927.01S

dist: (4) Addressee
(1) file

APPENDIX I

SITE INFORMATION

Vicinity Map

Geologic Map, Thomas W. Dibblee

Geologic Map, CDMG

Earthquake Fault Zones Map

Seismic Hazard Map

Exploration Log, Figure E.1 (SubSurface Designs, Inc., 2008)

Geologic Map, Plate A (pocket attachment)

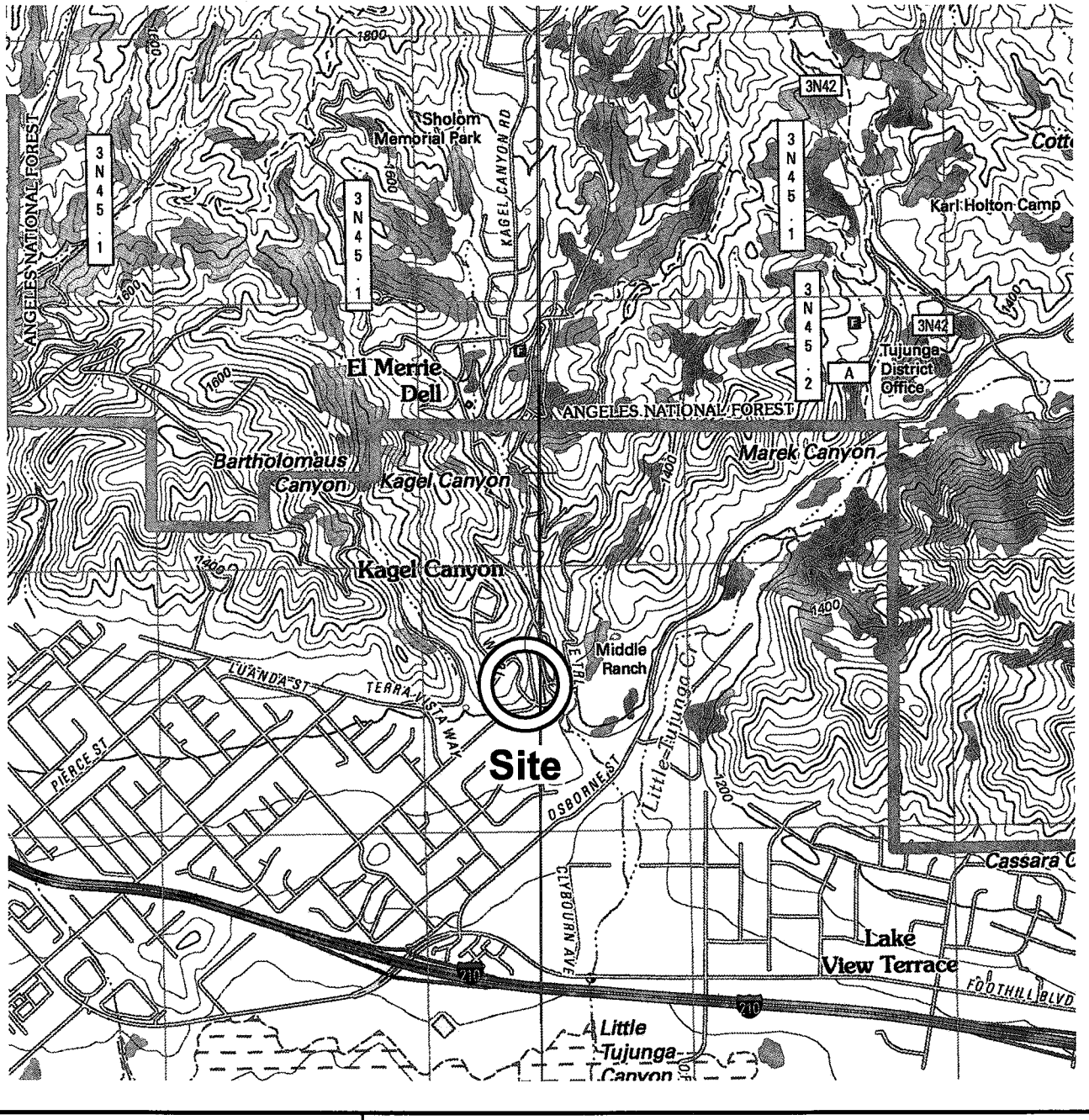
Geologic Cross-Sections, Plate B (pocket attachment)

Seismic Trench ST 01, Plate C (pocket attachment)

Seismic Trench ST 01 (SubSurface Designs, Inc., 2008) (pocket attachment)

VICINITY MAP



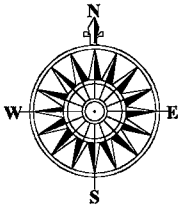


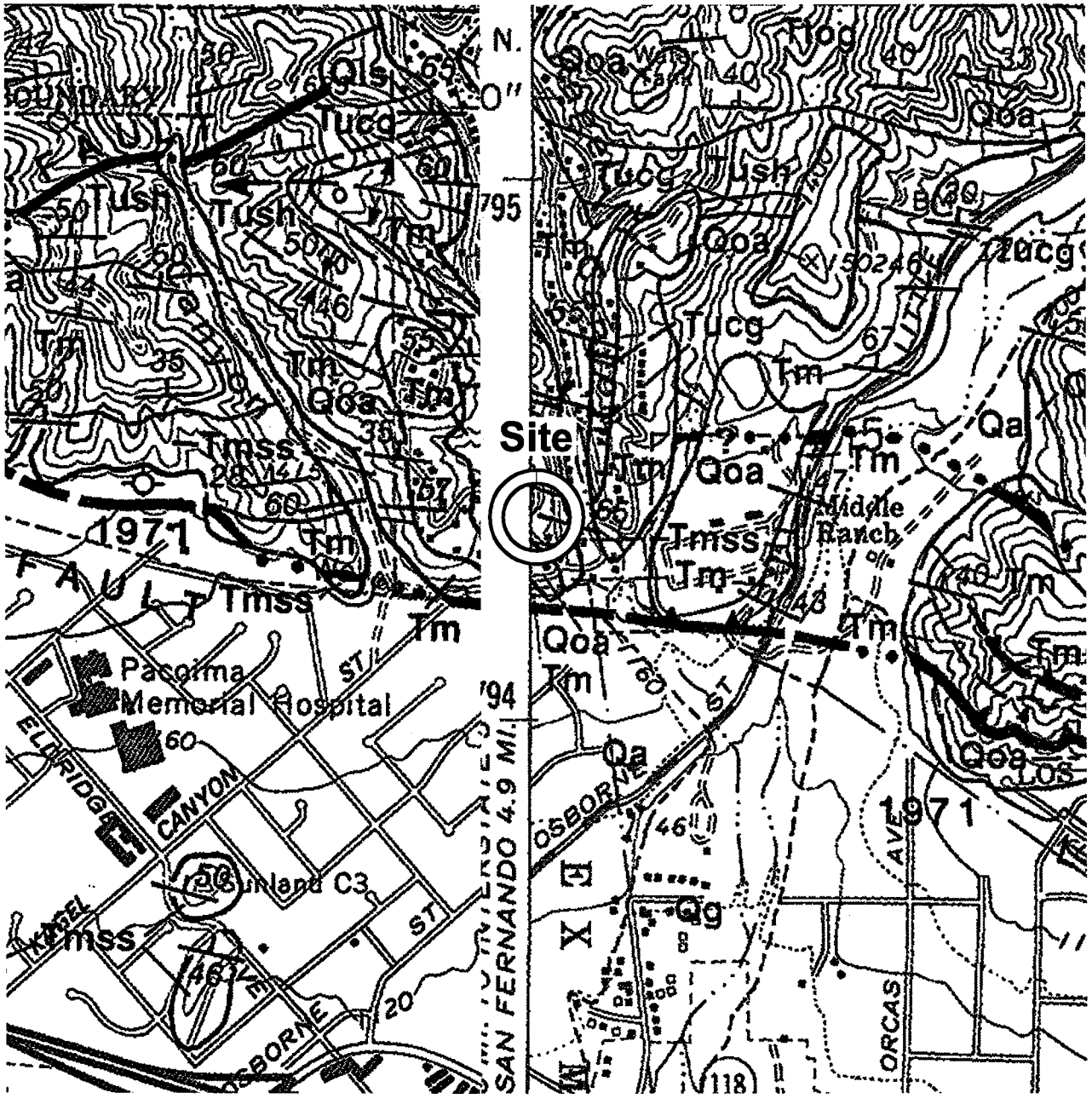
SubSurface Designs, Inc.
Geotechnical Engineers
Engineering Geologists

Ref: Modified from the Topographic Maps of the Sand Fernando & Sunland Quadrangles, prepared by U.S.G.S., 2012.
Scale: 1" = 2000'

DIBBLEE GEOLOGIC MAP



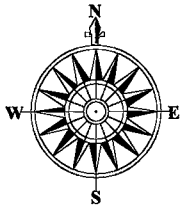


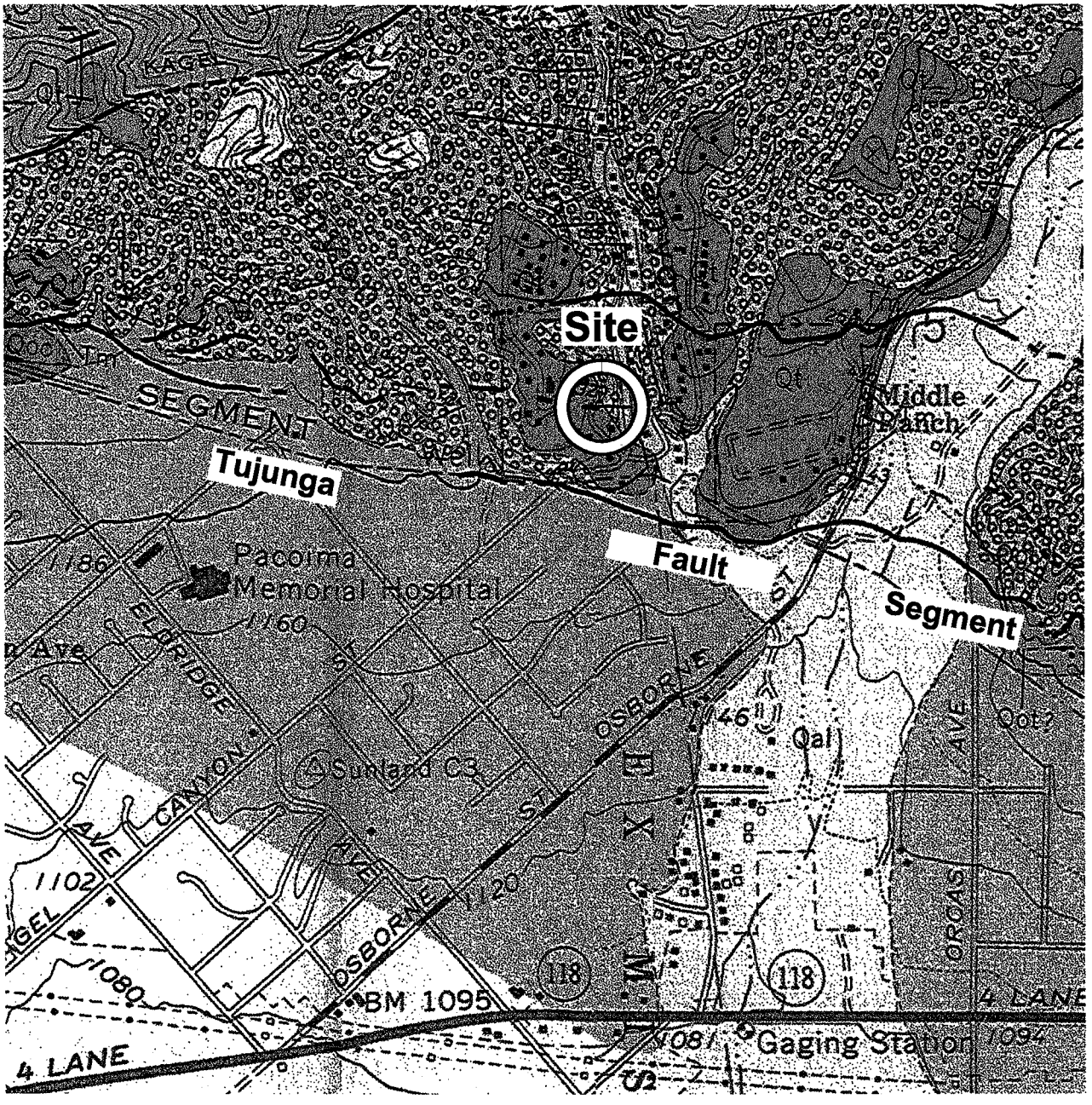


SubSurface Designs, Inc. Geotechnical Engineers Engineering Geologists	**Ref:** Modified from the Geologic Maps of the Sunland / Burbank (North 1/2) & San Fernando/Van Nuys (North 1/2) Quadrangles, prepared by Thomas W. Dibblee, 1991. **Scale:** 1" = 1000'



CDMG GEOLOGIC MAP

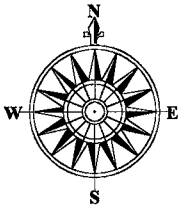


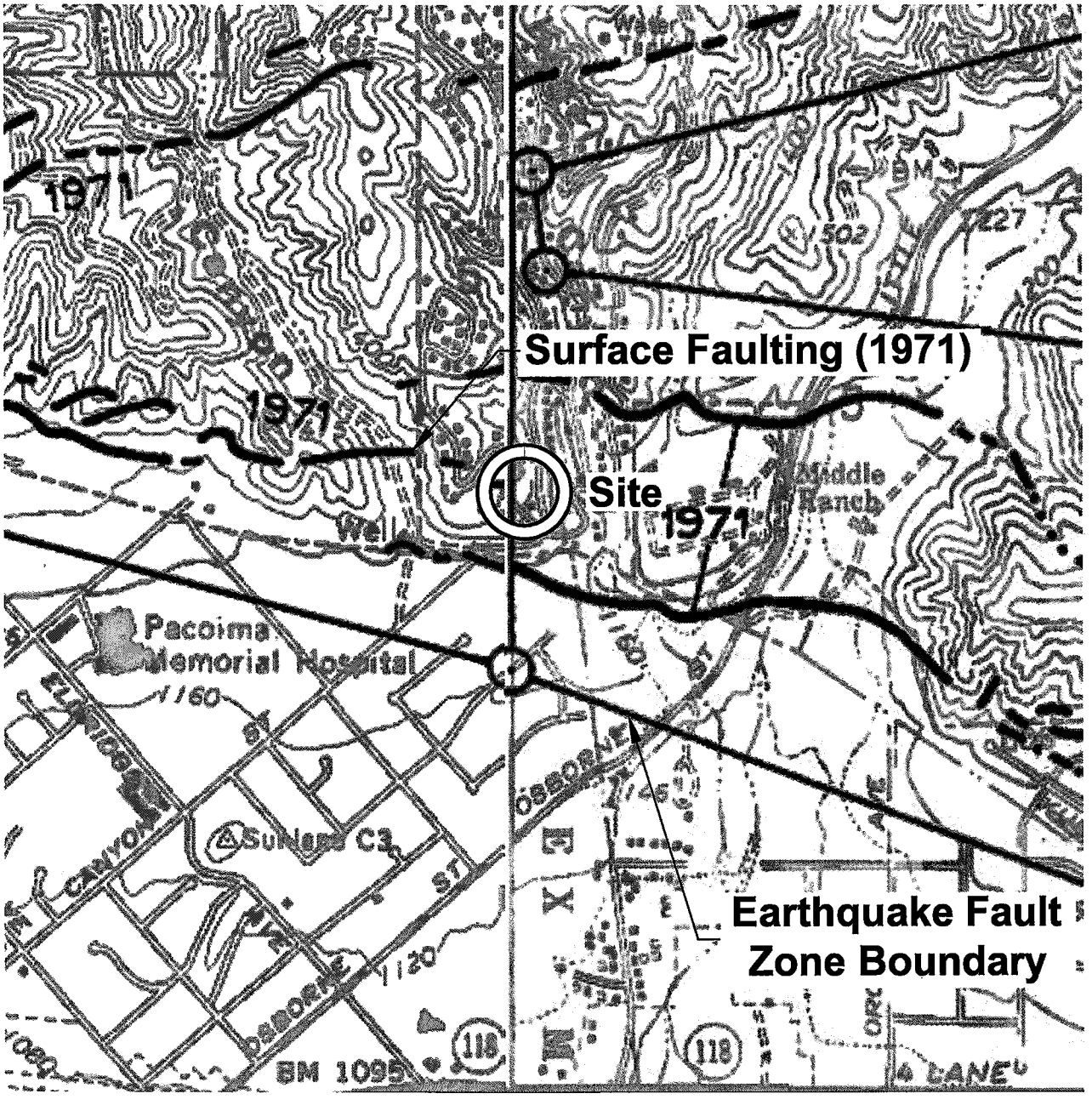


SubSurface Designs, Inc. Geotechnical Engineers Engineering Geologists	**Ref:** Modified from the Geologic Map of the San Fernando Earthquake Area, prepared by the State of California Division of Mines and Geology, 1974. **Scale:** 1" = 1000'



EARTHQUAKE FAULT ZONES



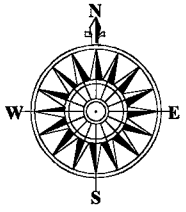


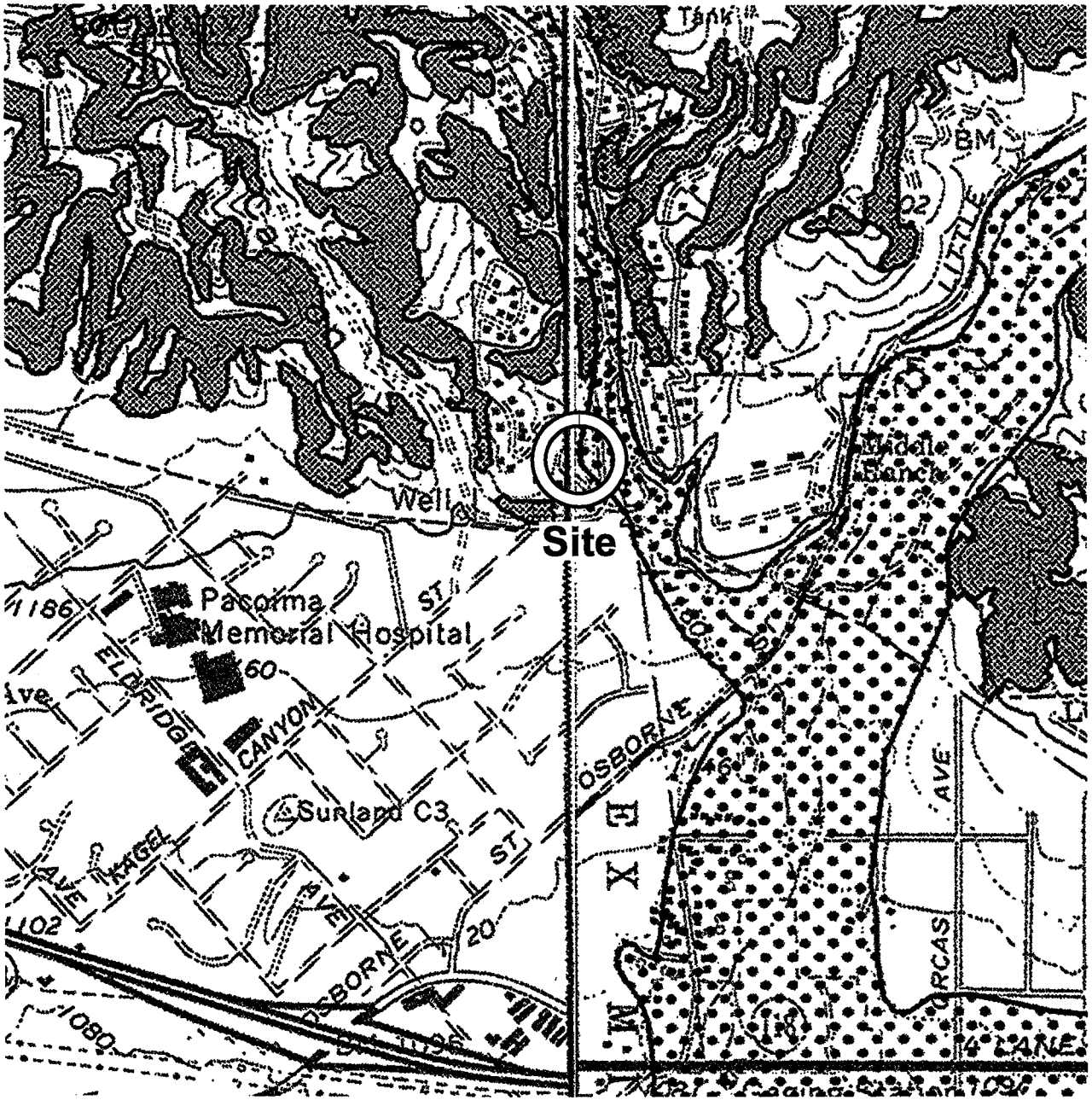
SubSurface Designs, Inc.
Geotechnical Engineers
Engineering Geologists

Ref: Modified from the Earthquake Fault Zones Maps of the San Fernando & Sunland Quadrangles, prepared by the State of California Division of Mines and Geology, 1979.
Scale: 1" = 1000'

SEISMIC HAZARD MAP







SubSurface Designs, Inc.
Geotechnical Engineers
Engineering Geologists

Ref: Modified from the Seismic Hazard Zone Maps of the San Fernando & Sunland Quadrangles, prepared by the State of California Division of Mines and Geology, 1999.
Scale: 1" = 1000'

Exploration Log, Figure E.1 (SubSurface Designs, Inc., 2008)

EXPLORATION LOG

PROJECT NAME: *Julian / Kagel Canyon Road*	EXPLORATION NO: *DH 01*	PAGE *1* OF *1*
PROJECT NUMBER: *PIN 5358*	EXPLORATION EQUIPMENT: *Limited Access Drill-Rig*	
Comments: *See attached Geologic Map for location.*		

Logged By: *CLK*	Total Depth: *34.0'*
Date Started: *March 20, 2008*	Top Elevation (ft.):
Date Completed: *March 21, 2008*	Excavation Width: *24"*

Sample Graphics	Blow Count (Per Foot)	Moisture Content (%)	Unit Dry Wt. (pcf.)	Depth (ft.)	Lithologic Graphics	LITHOLOGIC DESCRIPTION
	20	11	100	2		**Terrace Deposits (Qt)** Sandy Gravel - medium yellowish-brown, moist, moderately dense to dense; many cobbles; some boulders.
	20	6	102	6		
	35	4	112	10		
	--			12		
	--	2	111	16		
				18		18.0' - moderate caving to 28.0'
				28		**Modelo Formation (Tm)** Sandstone and Siltstone - yellowish-brown, moist, hard.
	100	6	97	30		
	--			32		
	100	7	101	34		
				36		
				38		

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Figure E.1

APPENDIX II

LABORATORY TEST RESULTS

Laboratory Testing

Laboratory Recapitulation - Table I-1

Shear Strength Diagram, Figure S.1 through S.2

Consolidation Diagram, Figure C.1 through C.3

LABORATORY TEST RESULTS

Moisture and Density Tests

The moisture content and in-place dry density of all undisturbed samples obtained were determined. The test results are presented in the Laboratory Recapitulation - Table I. Tests are performed in accordance with the latest version of ASTM D 2216.

Shear Tests

Direct single-shear tests were performed on representative undisturbed samples to determine their strength characteristics. The desired normal load was applied to the specimen and allowed to come to equilibrium. The rate of deflection on the sample is approximately 0.01 inches per minute. Depending upon the sample location and future site condition, samples may be tested at field moisture. The results are plotted on the Shear Test Diagrams and values are presented in the Laboratory Recapitulation - Table I. Tests are performed in accordance with the latest version of ASTM D 3080.

Consolidation

Consolidation tests were performed on undisturbed samples to predict the soils behavior under a specific load. Loads are applied in increasing load increments and the results are recorded. The samples are usually inundated at a designated load to determine the effect of water contacting the bearing soil. The results are plotted on the "Consolidation Pressure Curve," figures. The load at which the water is added is noted on the drawing. Tests are performed in accordance with the latest version of ASTM D 2435.

Expansion Tests

Expansion tests are performed on representative samples to determine the expansive potential of compacted soils when inundated with water. The test was performed in accordance with the latest version of ASTM D 4829. The classification of potentially expansive soil is based on the following table.

Expansion Index	Potential Expansion
0-20	Very Low
20-50	Low
50-90	Medium
90-130	High
Above 130	Very High

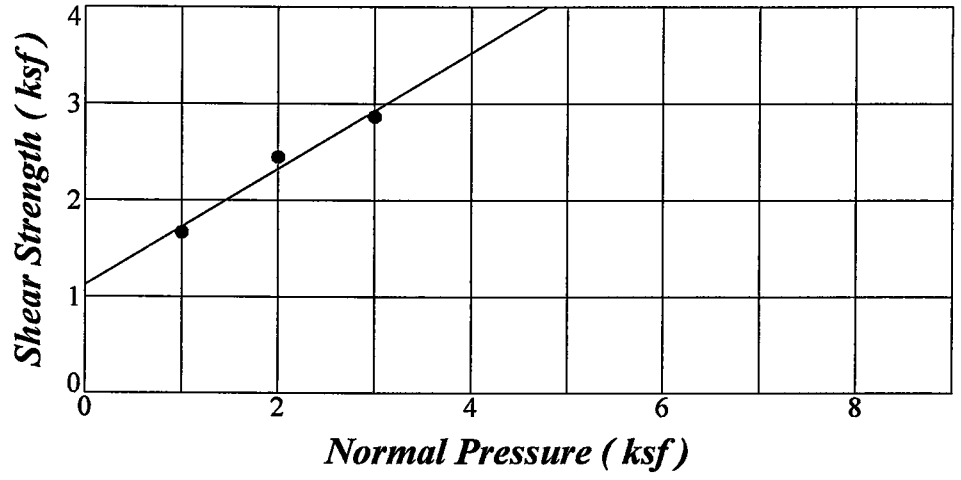
LABORATORY RECAPITULATION

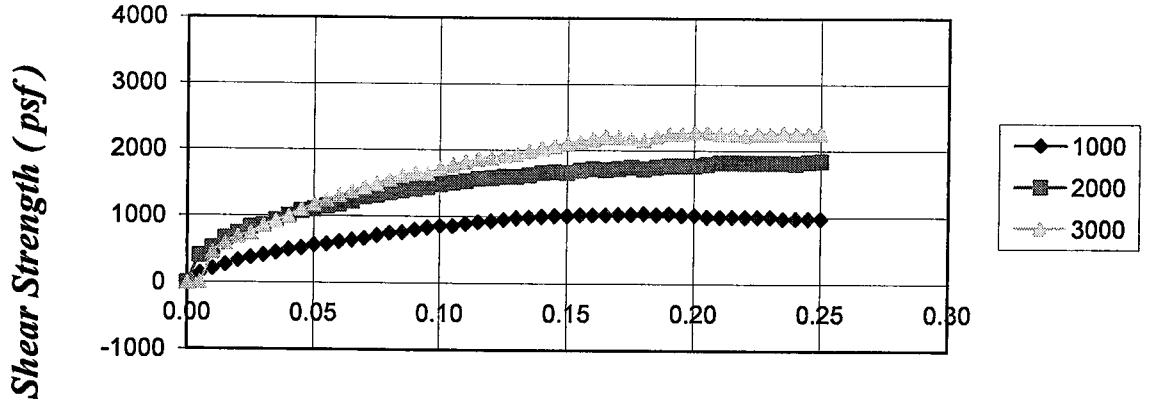
Table I-1

ST 01 Location	Depth (ft)	Material Type	In Situ Dry Density. (P.C.F.)	In Situ Water (%)	Expansion Index	Cohesion (ksf)	Friction Angle (deg)
Sta 0+15	0.5	Qsw	bulk	13.1			
Sta 0+15	1.0	Qsw	96.3	6.8			
Sta 0+15	3.5	Tm	100.1	10.6		1.125	31.0
Sta 0+15	4.0	Tm	bulk	6.0			
Sta 0+30	4.5	Tm	81.4	16.4			
Sta 0+50	3.5	Tm	84.0	18.3			
Sta 0+50	5.0	Tm	bulk	4.7			
Sta 0+85	6.5	Qt	128.0	2.8			
Sta 0+85	7.0	Qt	bulk	7.6			
Sta 0+87	5.5	Qt	125.3	2.9			
Sta 0+87	6.0	Qt	bulk	3.7			
Sta 0+95	3.5	Qt	123.2	4.9		0.410	33.0
Sta 0+95	4.0	Qt	bulk	4.9	10		
Sta 1+00	1.0	Qns	bulk	2.6			
Sta 1+00	1.5	Qns	127.2	3.7			
Sta 1+20	4.0	ef	bulk	5.9			
Sta 1+20	4.5	ef	112.7	5.5			
Sta 1+65	4.0	Qt	bulk	4.7			

SHEAR TEST DIAGRAM

PROJECT NAME: *Simpson / Kagel Canyon Rd.*	SAMPLE ID: *ST 01 Sta 0+15 @ 3.50'*
PROJECT NUMBER: *PIN 6927*	MATERIAL DESCRIPTION: *Bedrock (Tm)*
TEST METHOD: *Ultimate Saturated Shear*	





MOISTURE CONTENT (%)		DENSITY (pcf)		RESULTS	
In Situ:	*10.6*			Phi (deg.):	*31.0*
Saturated:	*29.8*	Dry Density:	*100.1*	Cohesion (ksf):	*1.125*

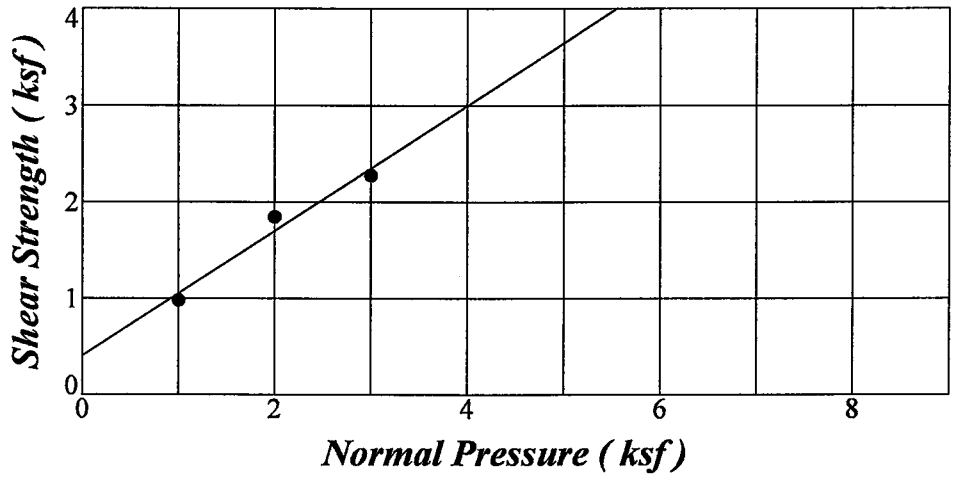
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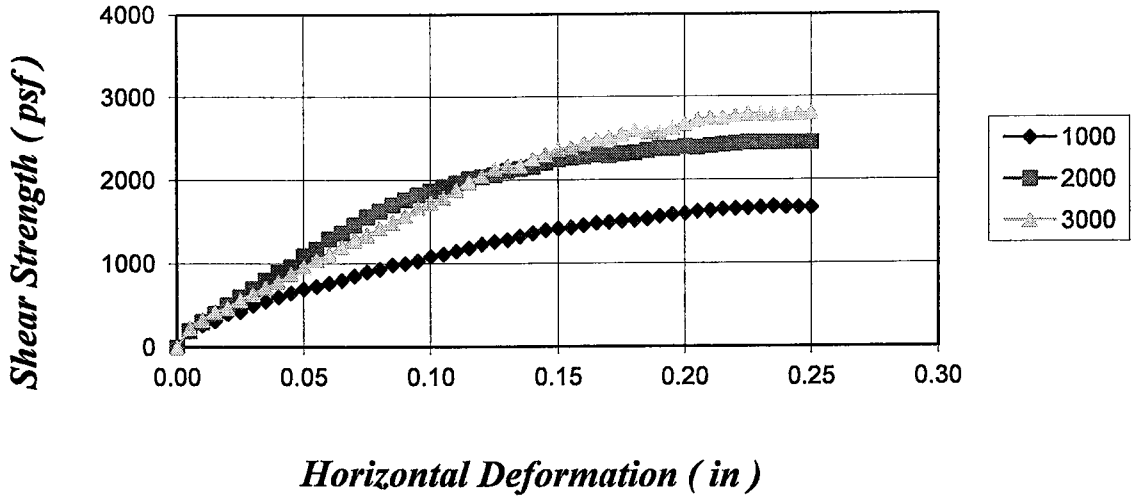
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Figure S.1

SHEAR TEST DIAGRAM

PROJECT NAME: *Simpson / Kagel Canyon Rd.*	SAMPLE ID: *ST 01 Sta 0+95 @ 3.50'*
PROJECT NUMBER: *PIN 6927*	MATERIAL DESCRIPTION: *Terrace Deposits (Qt)*
TEST METHOD: *Ultimate Saturated Shear*	





MOISTURE CONTENT (%)		DENSITY (pcf)		RESULTS	
In Situ:	*4.9*			Phi (deg.):	*33.0*
Saturated:	*12.5*	Dry Density:	*123.2*	Cohesion (ksf):	*0.410*

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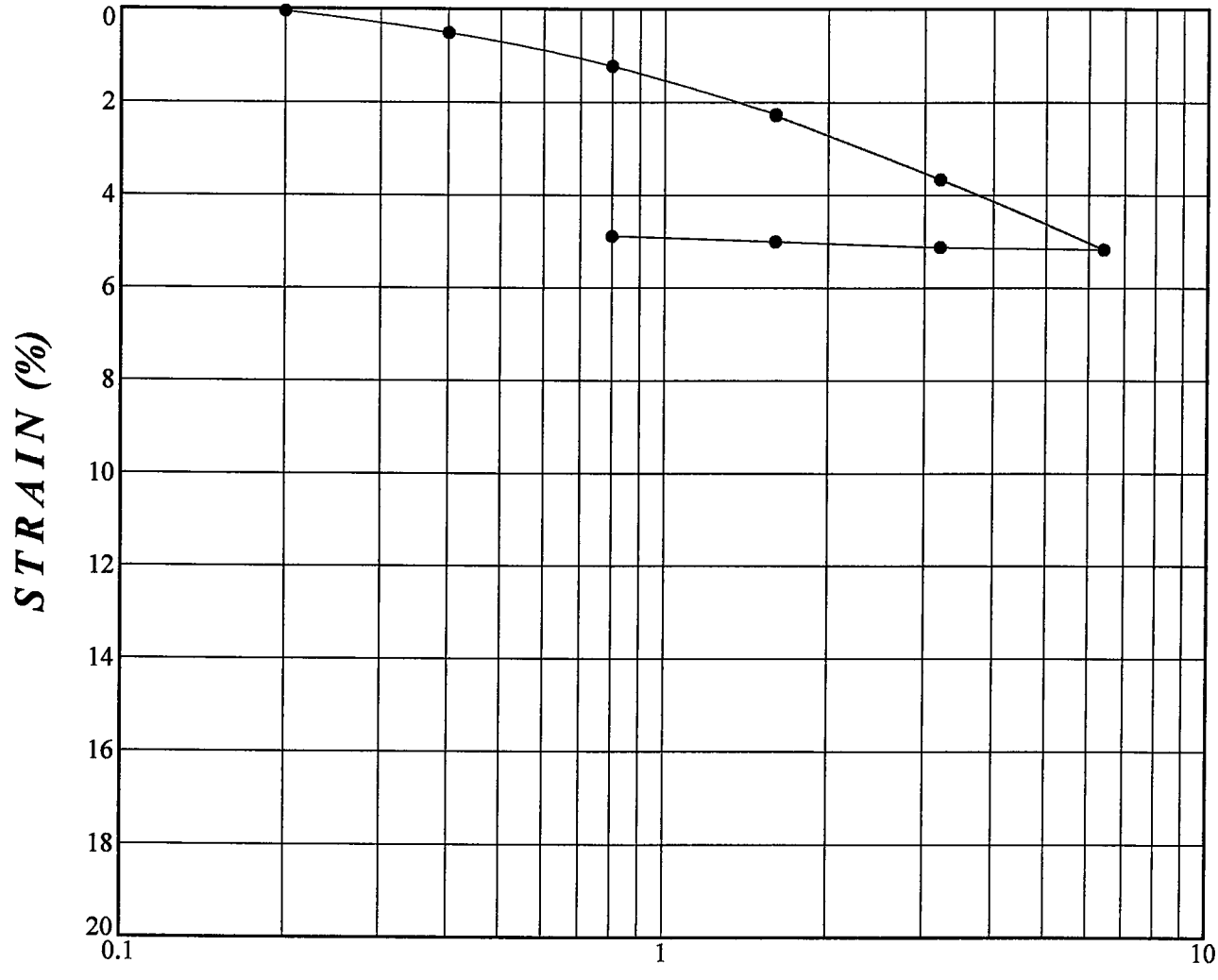
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Figure S.2

CONSOLIDATION TEST

PROJECT NAME: *Simpson / Kagel Canyon Rd.*	SAMPLE ID: *ST 01 Sta 0+85 @ 6.50'*
PROJECT NUMBER: *PIN 6927*	MATERIAL DESCRIPTION: *Terrace Deposits (Qt)*

Load (psf) water added to test at: 1600



MOISTURE CONTENT (%)		DRY DENSITY (pcf)		RESULTS	
In Situ:	*2.8*	Before Test:	*128.0*	Initial Void Ratio:	*0.5724*

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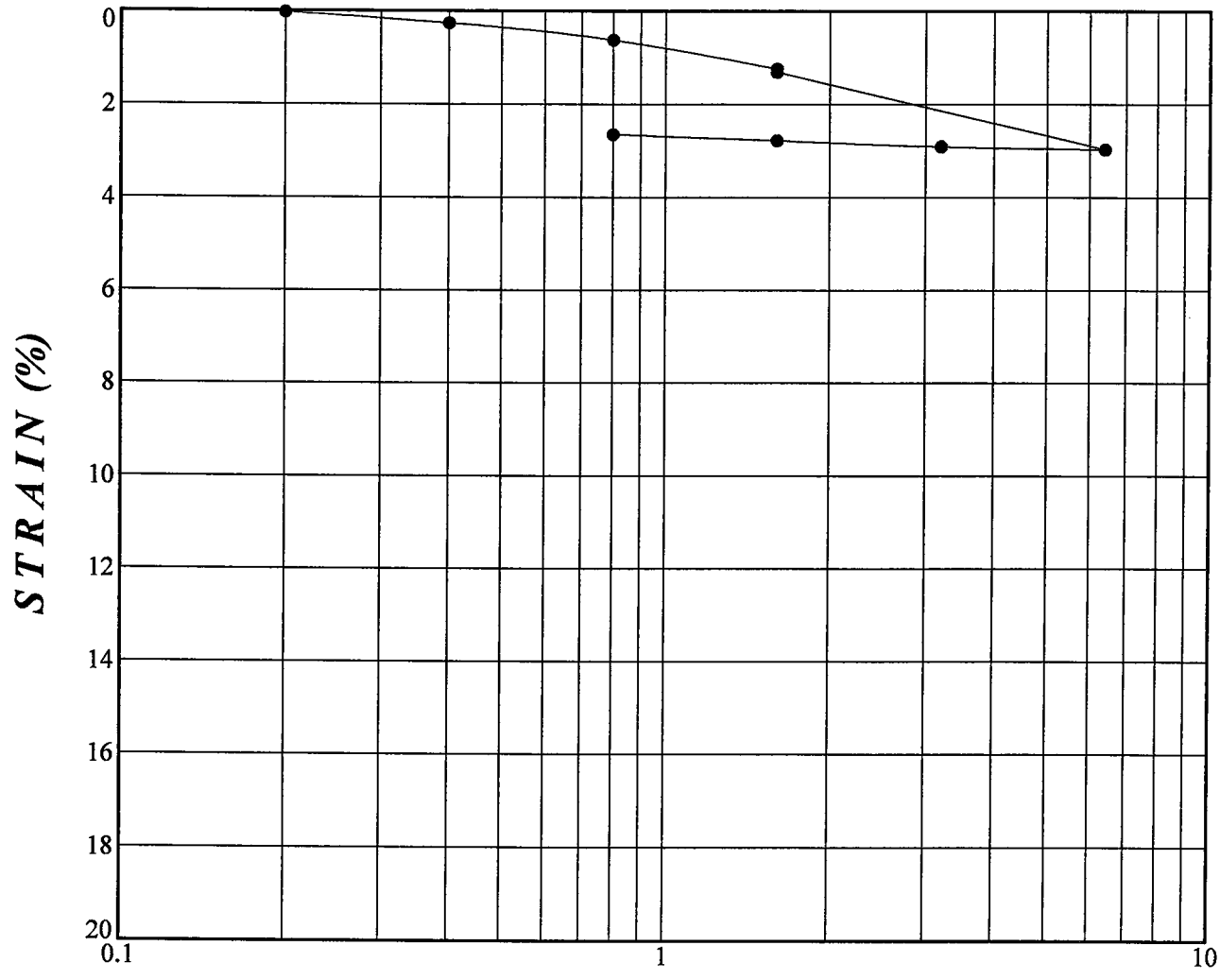
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Figure C.1

CONSOLIDATION TEST

PROJECT NAME: *Simpson / Kagel Canyon Rd.*	SAMPLE ID: *ST 01 Sta 0+87 @ 5.50'*
PROJECT NUMBER: *PIN 6927*	MATERIAL DESCRIPTION: *Terrace Deposits (Qt)*

Load (psf) water added to test at: 1600



MOISTURE CONTENT (%)		DRY DENSITY (pcf)		RESULTS	
In Situ:	*2.9*	Before Test:	*125.3*	Initial Void Ratio:	*0.4266*

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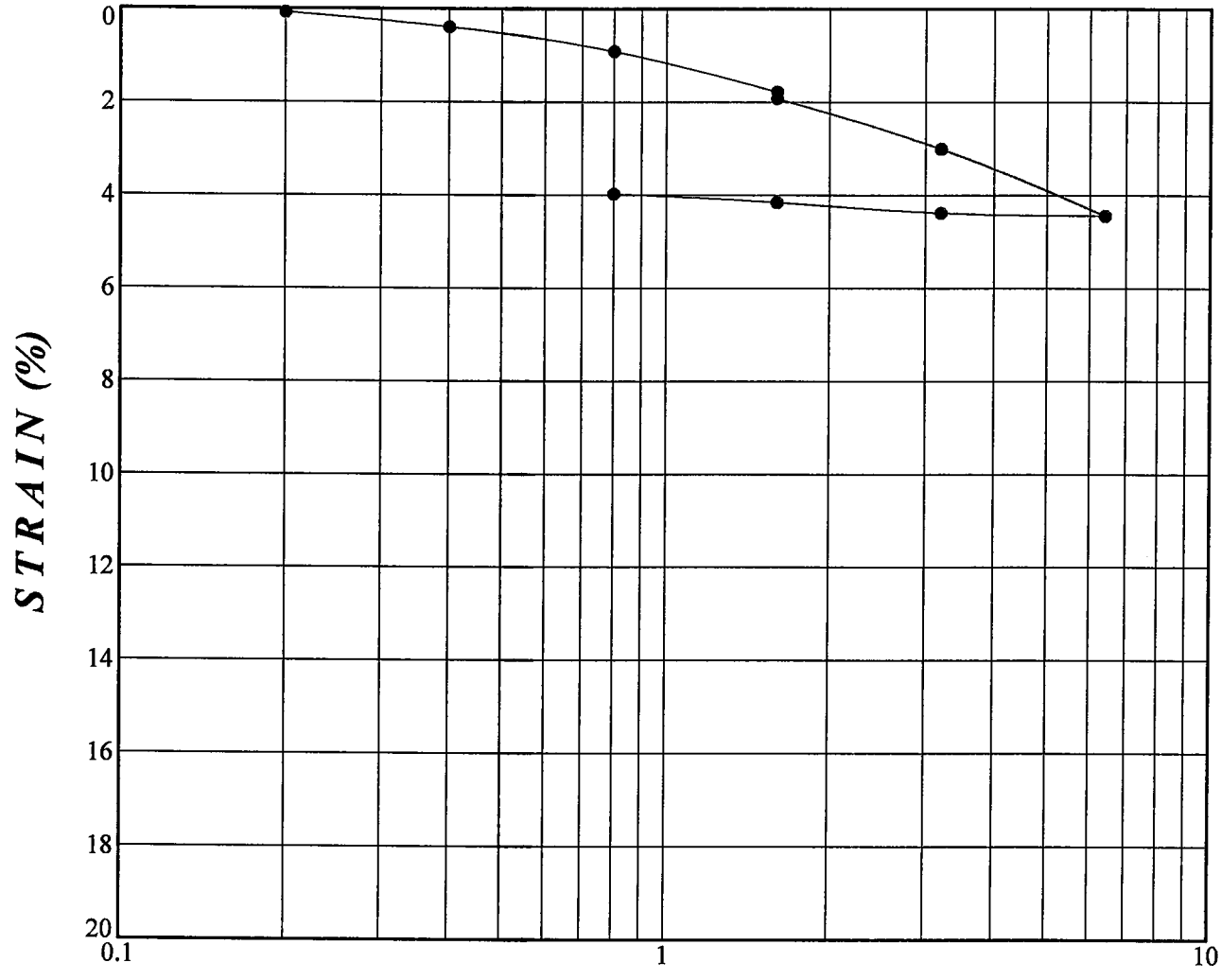
GEOTECHNICAL ENGINEERS & ENGINEERING GEOLOGISTS

Figure C.2

CONSOLIDATION TEST

PROJECT NAME: *Simpson / Kagel Canyon Rd.*	SAMPLE ID: *ST 01 Sta 0+95 @ 3.50'*
PROJECT NUMBER: *PIN 6927*	MATERIAL DESCRIPTION: *Terrace Deposits (Qt)*

Load (psf) water added to test at: 1600



MOISTURE CONTENT (%)		DRY DENSITY (pcf)		RESULTS	
In Situ:	*4.9*	Before Test:	*123.2*	Initial Void Ratio:	*0.4300*

SubSurface Designs, Inc.

GEOTECHNICAL ENGINEERS & ENGINEERING GEOLOGISTS

Figure C.3

APPENDIX III

CALCULATIONS

Bearing Value

Temporary Stability Calculations

Slot Cut Calculations

Stability - Taylor's Method

Shoring Pile Calculations

Retaining Wall Calculations

Slope Stability

SubSurface Designs Inc.

BEARING CAPACITY ANALYSIS

PIN # **6927**
CLIENT: **Simpson**

CALCULATE THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL LISTED BELOW USING HANSEN'S METHOD. (REFERENCE: J. BOWLES, *FOUNDATION ANALYSIS AND DESIGN*, 1988, p. 188-194).

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	EMBEDMENT DEPTH:	1 feet
SHEAR DIAGRAM:	0	FOOTING LENGTH:	100 feet
COHESION:	1125 psf	FOOTING WIDTH:	1 feet
PHI ANGLE:	31 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	135 pcf	FOOTING INCLINATION:	0 degrees
SAFETY FACTOR:	3		
FOOTING TYPE:	S Strip		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	20.63	Dq =	1.28	Sy =	1.00
Nc =	32.67	Gc =	1.00	Dy =	1.00
Ny =	17.69	Bc =	1.00	Iy =	1.00
Sc =	1.01	Iq =	1.00	Gy =	1.00
Sq =	1.01	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

CALCULATED ULTIMATE BEARING CAPACITY (Qult)	56,565.3 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	18,855.1 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	9.5%

CONCLUSIONS:

THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL WERE CALCULATED USING HANSEN'S METHOD. THE RECOMMENDED DESIGN BEARING PRESSURE IS 3000 POUNDS PER SQUARE FOOT.

SubSurface Designs Inc.

BEARING CAPACITY ANALYSIS

PIN # **6927**
CLIENT: **Simpson**

CALCULATE THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL LISTED BELOW USING HANSEN'S METHOD. (REFERENCE: J. BOWLES, *FOUNDATION ANALYSIS AND DESIGN*, 1988, p. 188-194).

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	EMBEDMENT DEPTH:	1 feet
SHEAR DIAGRAM:	0	PAD LENGTH:	1 feet
COHESION:	1125 psf	PAD WIDTH:	1 feet
PHI ANGLE:	31 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	135 pcf	PAD INCLINATION:	0 degrees
SAFETY FACTOR:	3		
FOOTING TYPE:	P Pad		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	20.63	Dq =	1.28	Sy =	0.60
Nc =	32.67	Gc =	1.00	Dy =	1.00
Ny =	17.69	Bc =	1.00	Iy =	1.00
Sc =	1.63	Iq =	1.00	Gy =	1.00
Sq =	1.60	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

CALCULATED ULTIMATE BEARING CAPACITY (Qult)	90,385.9 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	30,128.6 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	9.5%

CONCLUSIONS:

THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL WERE CALCULATED USING HANSEN'S METHOD. THE RECOMMENDED DESIGN BEARING PRESSURE IS 3500 POUNDS PER SQUARE FOOT.

SubSurface Designs Inc.

BEARING CAPACITY ANALYSIS

PIN # **6927**
CLIENT: **Simpson**

CALCULATE THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL LISTED BELOW USING HANSEN'S METHOD. (REFERENCE: J. BOWLES, *FOUNDATION ANALYSIS AND DESIGN*, 1988, p. 188-194).

CALCULATION PARAMETERS

EARTH MATERIAL:	TERRACE DEPOSITS	EMBEDMENT DEPTH:	1 feet
SHEAR DIAGRAM:	0	PAD LENGTH:	1 feet
COHESION:	410 psf	PAD WIDTH:	1 feet
PHI ANGLE:	33 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	125 pcf	PAD INCLINATION:	0 degrees
SAFETY FACTOR:	3		
FOOTING TYPE:	P Pad		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	26.09	Dq =	1.27	Sy =	0.60
Nc =	38.64	Gc =	1.00	Dy =	1.00
Ny =	24.44	Bc =	1.00	Iy =	1.00
Sc =	1.68	Iq =	1.00	Gy =	1.00
Sq =	1.65	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

CALCULATED ULTIMATE BEARING CAPACITY (Qult)	44,900.1 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	14,966.7 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	18.4%

CONCLUSIONS:

THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL WERE CALCULATED USING HANSEN'S METHOD. THE RECOMMENDED DESIGN BEARING PRESSURE IS 2500 POUNDS PER SQUARE FOOT.

SubSurface Designs Inc.

BEARING CAPACITY ANALYSIS

PIN # **6927**
CLIENT: **Simpson**

CALCULATE THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL LISTED BELOW USING HANSEN'S METHOD. (REFERENCE: J. BOWLES, *FOUNDATION ANALYSIS AND DESIGN*, 1988, p. 188-194).

CALCULATION PARAMETERS

EARTH MATERIAL:	TERRACE DEPOSITS	EMBEDMENT DEPTH:	1 feet
SHEAR DIAGRAM:	0	FOOTING LENGTH:	100 feet
COHESION:	410 psf	FOOTING WIDTH:	1 feet
PHI ANGLE:	33 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	125 pcf	FOOTING INCLINATION:	0 degrees
SAFETY FACTOR:	3		
FOOTING TYPE:	S Strip		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	26.09	Dq =	1.27	Sy =	1.00
Nc =	38.64	Gc =	1.00	Dy =	1.00
Ny =	24.44	Bc =	1.00	Iy =	1.00
Sc =	1.01	Iq =	1.00	Gy =	1.00
Sq =	1.01	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

CALCULATED ULTIMATE BEARING CAPACITY (Qult)	28,016.5 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	9,338.8 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	18.0%

CONCLUSIONS:

THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL WERE CALCULATED USING HANSEN'S METHOD. THE RECOMMENDED DESIGN BEARING PRESSURE IS 2000 POUNDS PER SQUARE FOOT.

SubSurface Designs Inc.

TEMPORARY EXCAVATION HEIGHT

PIN # 6927
CLIENT: Simpson

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:	TERRACE DEPOSITS	HEIGHT:	5 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	45 degrees
COHESION:	410 psf	SURCHARGE:	0 pounds
PHI ANGLE:	33 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	125 pcf	INITIAL FAILURE ANGLE:	5 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	85 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	328.0 psf	FINAL TENSION CRACK:	100 feet
PHID = ATAN(TAN(PHI)/FS) =	27.5 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	47 degrees
AREA OF TRIAL FAILURE WEDGE	5.0 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	620.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	8100 trials
LENGTH OF FAILURE PLANE	1.5 feet
DEPTH OF TENSION CRACK	4.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	1.0 feet
CALCULATED HORIZONTAL THRUST	**-232.6 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-18.6 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**5.0 feet**

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE TEMPORARY EXCAVATIONS UP TO 5 FEET HIGH WITH A 1:1 BACKSLOPE HAVE A NEGATIVE THRUST AND ARE TEMPORARILY STABLE.

SubSurface Designs Inc.

SLOT CUT ANALYSIS

PIN # **6927**
CLIENT: **Simpson**

CALCULATE THE FACTOR OF SAFETY OF SLOT CUT EXCAVATIONS. ASSUME COHESIVE AND FRICTIONAL RESISTANCE ALONG THE SIDES OF SLOTS AS WELL AS THE FAILURE SURFACE. THE HORIZONTAL PRESSURE ON THE SIDES OF THE SLOTS IS THE AT-REST PRESSURE (1-SIN(phi)).

CALCULATION PARAMETERS

EARTH MATERIAL:	TERRACE DEPOSITS	EXCAVATION HEIGHT:	8 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	410 psf	SURCHARGE:	1000 pounds
PHI ANGLE:	33 degrees	SURCHARGE TYPE:	P Point
DENSITY:	125 pcf	INITIAL FAILURE ANGLE:	8 degrees
SLOT BOUNDARY CONDITIONS		FINAL FAILURE ANGLE:	85 degrees
SLOT CUT WIDTH:	8 feet	INITIAL TENSION CRACK:	1 feet
COHESION:	410 psf	FINAL TENSION CRACK:	100 feet
PHI ANGLE:	33 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	60 degrees
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	1.0 feet
DEPTH OF TENSION CRACK	6.3 feet
TOTAL EXTERNAL SURCHARGE	1000.0 pounds
VOLUME OF FAILURE WEDGE	57.1 ft^3
WEIGHT OF FAILURE WEDGE	8134.0 pounds
LENGTH OF FAILURE PLANE	2.0 feet
SURFACE AREA OF FAILURE PLANE	16 ft^2
SURFACE AREA OF SIDES OF SLOTS	7.1 ft^2
NUMBER OF TRIAL WEDGES ANALYZED	61410 trials
TOTAL RESISTING FORCE ALONG WEDGE BASE (FrB)	3009.2 pounds
TOTAL RESISTING FORCE ALONG WEDGE SIDES (FrS)	2873.2 pounds
RESULTANT HORIZONTAL COMPONENT OF FORCE	**-65.6 pounds**
CALCULATED FACTOR OF SAFETY	**2.18**

CONCLUSIONS:

THE CALCULATION INDICATES THAT SLOTS CUTS UP TO 8 FEET WIDE AND 8 FEET HIGH HAVE A SAFETY FACTOR GREATER THAN 1.25 AND ARE TEMPORARILY STABLE.

SubSurface Designs Inc.

STABILITY - TAYLOR'S METHOD

PIN # **6927**
CLIENT: **Simpson**

CALCULATE THE MAXIMUM HEIGHT TO WHICH UNIFORM SLOPES ARE GROSSLY STABLE USING TAYLOR'S METHOD FOR THE STABILITY OF EARTHEN EMBANKMENTS (*FUNDAMENTALS OF SOIL MECHANICS*).

CALCULATION PARAMETERS

EARTH MATERIAL:	TERRACE DEPOSITS	SAFETY FACTOR:	1.5
SHEAR DIAGRAM:	0	SLOPE ANGLE:	90 degrees
COHESION:	410 psf	Cd Base (C/fs):	273.3 psf
PHI ANGLE:	33 degrees	PhiD = atan(tan(phi)fs) =	23.4 degrees
DENSITY (w):	125 pcf		

INTERPOLATE STABILITY NUMBER (sn) FROM TAYLOR'S CHARTS:

TAYLOR'S CHART

		SLOPE ANGLES						
	Degrees	20	30	40	50	60	70	80
PhiD	5	0.090	0.110	0.130	0.145	0.160	0.185	0.210
	10	0.045	0.075	0.100	0.120	0.140	0.160	0.188
	15	0.020	0.045	0.070	0.095	0.115	0.140	0.168
	20	0.000	0.025	0.050	0.075	0.098	0.120	0.150
	25	0.000	0.010	0.033	0.055	0.080	0.105	0.130

FROM CHART sn = **0.136**

SAFE SLOPE HEIGHT = $\frac{Cd}{w \times (sn)}$ **16.0 feet**

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE UNIFORM SLOPES IN THE BEDROCK ARE STABLE UP TO 16 FEET.

SubSurface Designs Inc.

SHORING PILE

PIN # 6927
CLIENT: Simpson

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING SYSTEM.. THE HEIGHT, 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:	TERRACE DEPOSITS	RETAINED LENGTH	16 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	410 psf	SURCHARGE:	0 pounds
PHI ANGLE:	33 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	125 pcf	INITIAL FAILURE ANGLE:	5 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	85 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	328.0 psf	FINAL TENSION CRACK:	100 feet

PHID = ATAN(TAN(PHI)/FS) = 27.5 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.000 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0.000 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	60 degrees
AREA OF TRIAL FAILURE WEDGE	50.1 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	6267.9 pounds
NUMBER OF TRIAL WEDGES ANALYZED	8100 trials
LENGTH OF FAILURE PLANE	8.0 feet
DEPTH OF TENSION CRACK	9.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	4.0 feet
CALCULATED THRUST ON PILE	**1238.0 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**9.7 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**20.0 pcf**

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

SubSurface Designs Inc.

RETAINING WALL

PIN # **6927**
CLIENT: **Simpson**

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING SYSTEM.. THE HEIGHT, 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:	TERRACE DEPOSITS	WALL HEIGHT	16 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	410 psf	SURCHARGE:	0 pounds
PHI ANGLE:	33 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	125 pcf	INITIAL FAILURE ANGLE:	5 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	85 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	273.3 psf	FINAL TENSION CRACK:	100 feet

PHID = ATAN(TAN(PHI)/FS) = 23.4 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.000 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0.000 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	57 degrees
AREA OF TRIAL FAILURE WEDGE	68.3 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	8535.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	8100 trials
LENGTH OF FAILURE PLANE	11.0 feet
DEPTH OF TENSION CRACK	6.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	6.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**2351.5 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**18.4 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**57.0 pcf**

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 57 POUNDS PER CUBIT FOOT.

Slope Stability

Saturated shear strength parameters are based on laboratory tests performed on samples selected which, in the opinion of the Soils Engineer, represent the existing conditions at the site. Calculations for slope stability are based upon the ultimate shear strength values of the tested material.

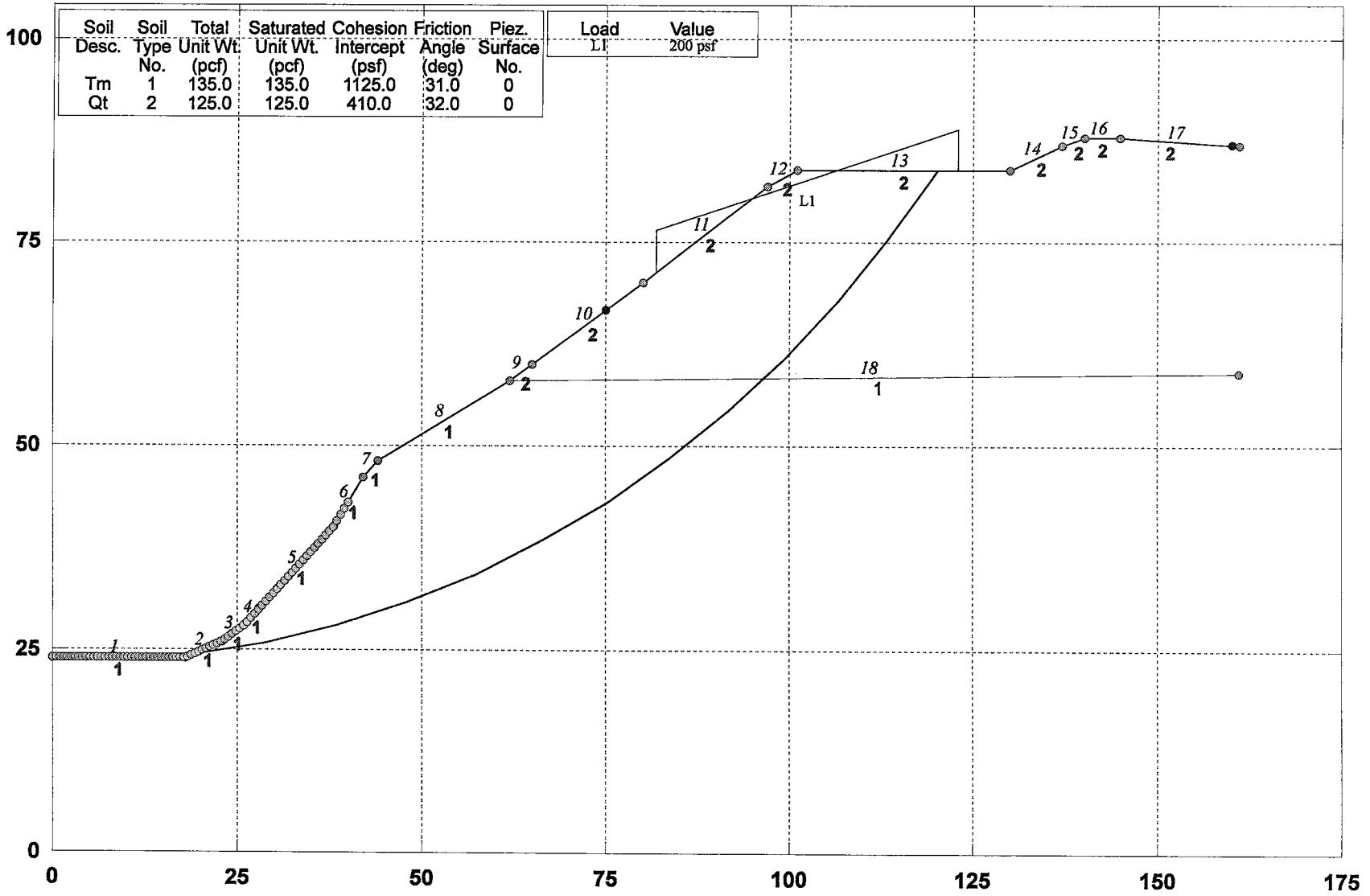
Gross stability analysis was performed utilizing a critical failure surface searching method, using a random technique for generating circular surfaces by Janbu's Method.

Below are results for each section analyzed.

Section	Surface	Factor of Safety	Notes
B-B'	Circular	2.09	Lowest factor of safety for section
B-B'	Circular	1.64	Seismic stability of section

Stability of Section B-B' 6927B

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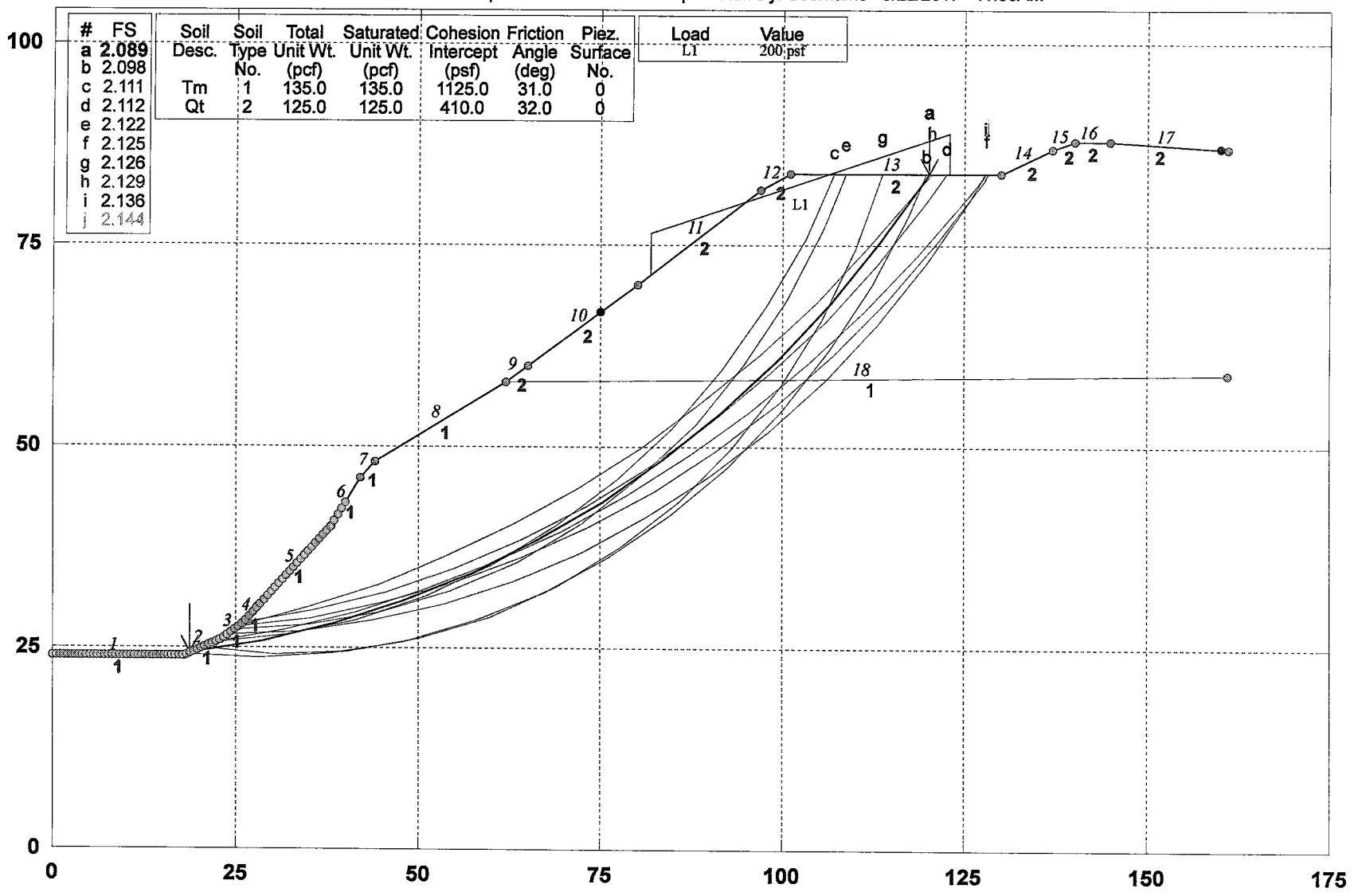
GSTABL7 v.2 FSmin=2.089

Safety Factors Are Calculated By The Modified Bishop Method

SubSurface Designs Inc.

Stability of Section B-B' 6927B

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#	FS
a	2.089
b	2.098
c	2.111
d	2.112
e	2.122
f	2.125
g	2.126
h	2.129
i	2.136
j	2.144

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Tm	1	135.0	135.0	1125.0	31.0	0
Qt	2	125.0	125.0	410.0	32.0	0

Load	Value
L1	200 psf

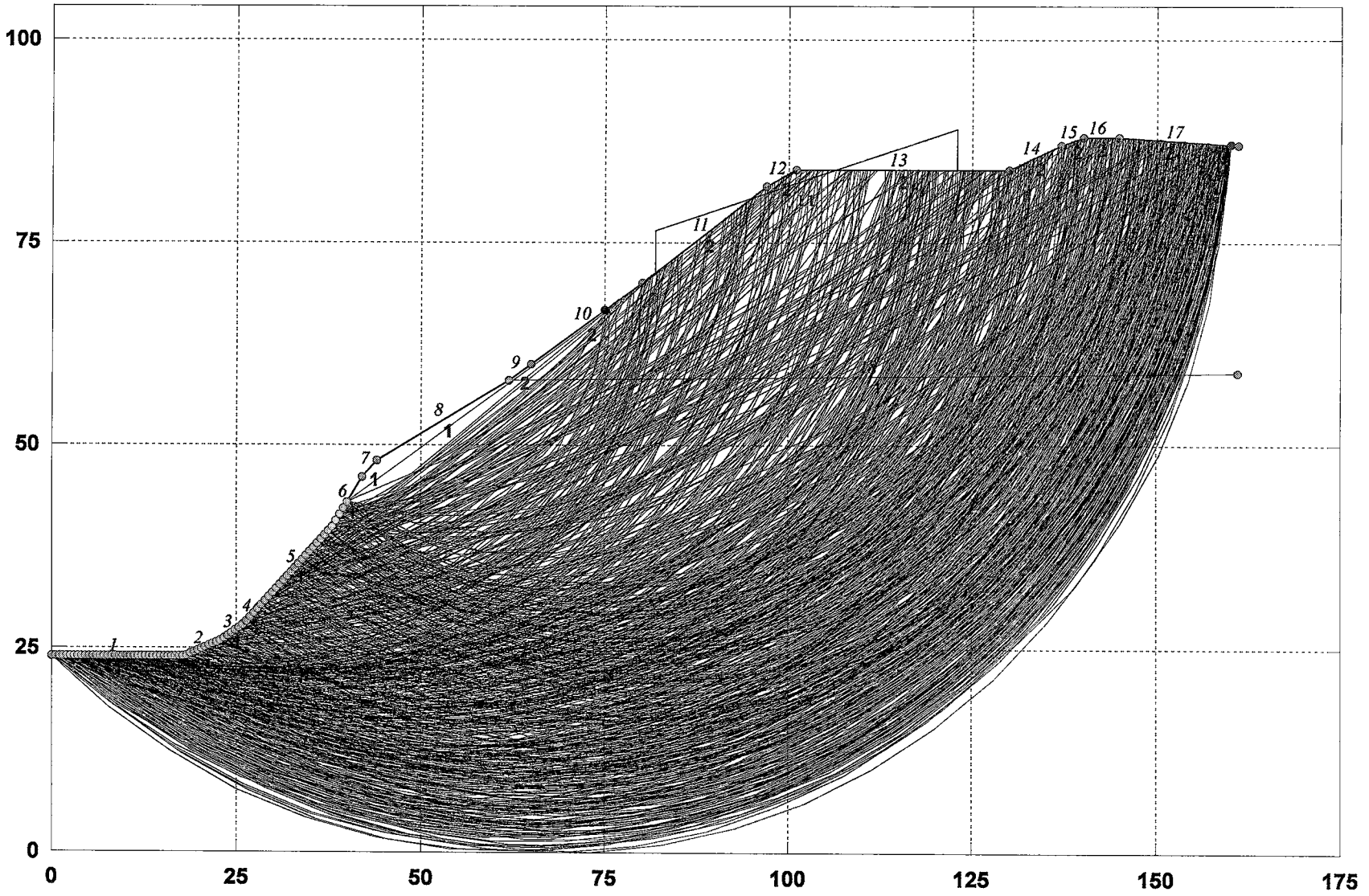
GSTABL7 v.2 FSmin=2.089

Safety Factors Are Calculated By The Modified Bishop Method

SubSurface Designs Inc.

Stability of Section B-B' 6927B

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SubSurface Designs Inc.

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                                    ***  GSTABL7  ***

                              ** GSTABL7 by Garry H. Gregory, P.E. **

            ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                          (All Rights Reserved-Unauthorized Use Prohibited)


*************************************************************************
                          SLOPE STABILITY ANALYSIS SYSTEM
                Modified Bishop, Simplified Janbu, or GLE Method of Slices.
                (Includes Spencer & Morgenstern-Price Type Analysis)
                Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
                Nonlinear Undrained Shear Strength, Curved Phi Envelope,
                Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
                Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*************************************************************************

          Analysis Run Date:        9/22/2017
          Time of Run:              11:38AM
          Run By:                   Username
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          Output Filename:          H:\6900\6927 - Simpson\Calculations\6927B.OUT


          Unit System:              English

          Plotted Output Filename:  H:\6900\6927 - Simpson\Calculati6927B.PLT




          PROBLEM DESCRIPTION:  Stability of Section B-B'
                                6927B



          BOUNDARY COORDINATES

             17 Top   Boundaries
             18 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	24.00	18.00	24.00	1
2	18.00	24.00	23.00	26.00	1
3	23.00	26.00	26.00	28.00	1
4	26.00	28.00	28.00	30.00	1
5	28.00	30.00	38.00	40.00	1
6	38.00	40.00	42.00	46.00	1
7	42.00	46.00	44.00	48.00	1
8	44.00	48.00	62.00	58.00	1
9	62.00	58.00	65.00	60.00	2
10	65.00	60.00	80.00	70.00	2
11	80.00	70.00	97.00	82.00	2
12	97.00	82.00	101.00	84.00	2
13	101.00	84.00	130.00	84.00	2
14	130.00	84.00	137.00	87.00	2
15	137.00	87.00	140.00	88.00	2
16	140.00	88.00	145.00	88.00	2
17	145.00	88.00	161.00	87.00	2
18	62.00	58.00	161.00	59.00	1

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	135.0	135.0	1125.0	31.0	0.00	0.0	0
2	125.0	125.0	410.0	32.0	0.00	0.0	0

BOUNDARY LOAD(S)

1 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	82.00	123.00	200.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

800 Trial Surfaces Have Been Generated.

10 Surface(s) Initiate(s) From Each Of 80 Points Equally Spaced
Along The Ground Surface Between X = 0.00(ft)
and X = 40.00(ft)

Each Surface Terminates Between X = 75.00(ft)
and X = 160.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)

10.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Most Critical Of The Trial
Failure Surfaces Evaluated.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Evaluated = 800

Statistical Data On All Valid FS Values:
FS Max = 9.592 FS Min = 2.089 FS Ave = 2.891
Standard Deviation = 0.614 Coefficient of Variation = 21.25 %

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	18.73	24.29
2	28.62	25.79
3	38.38	27.96
4	47.97	30.80
5	57.34	34.29
6	66.45	38.41
7	75.26	43.16
8	83.71	48.49

9	91.78	54.40
10	99.43	60.85
11	106.61	67.80
12	113.30	75.24
13	119.46	83.12
14	120.05	84.00

Circle Center At X = 2.03 ; Y = 168.45 ; and Radius = 145.12

Factor of Safety
*** 2.089 ***

Individual data on the 26 slices

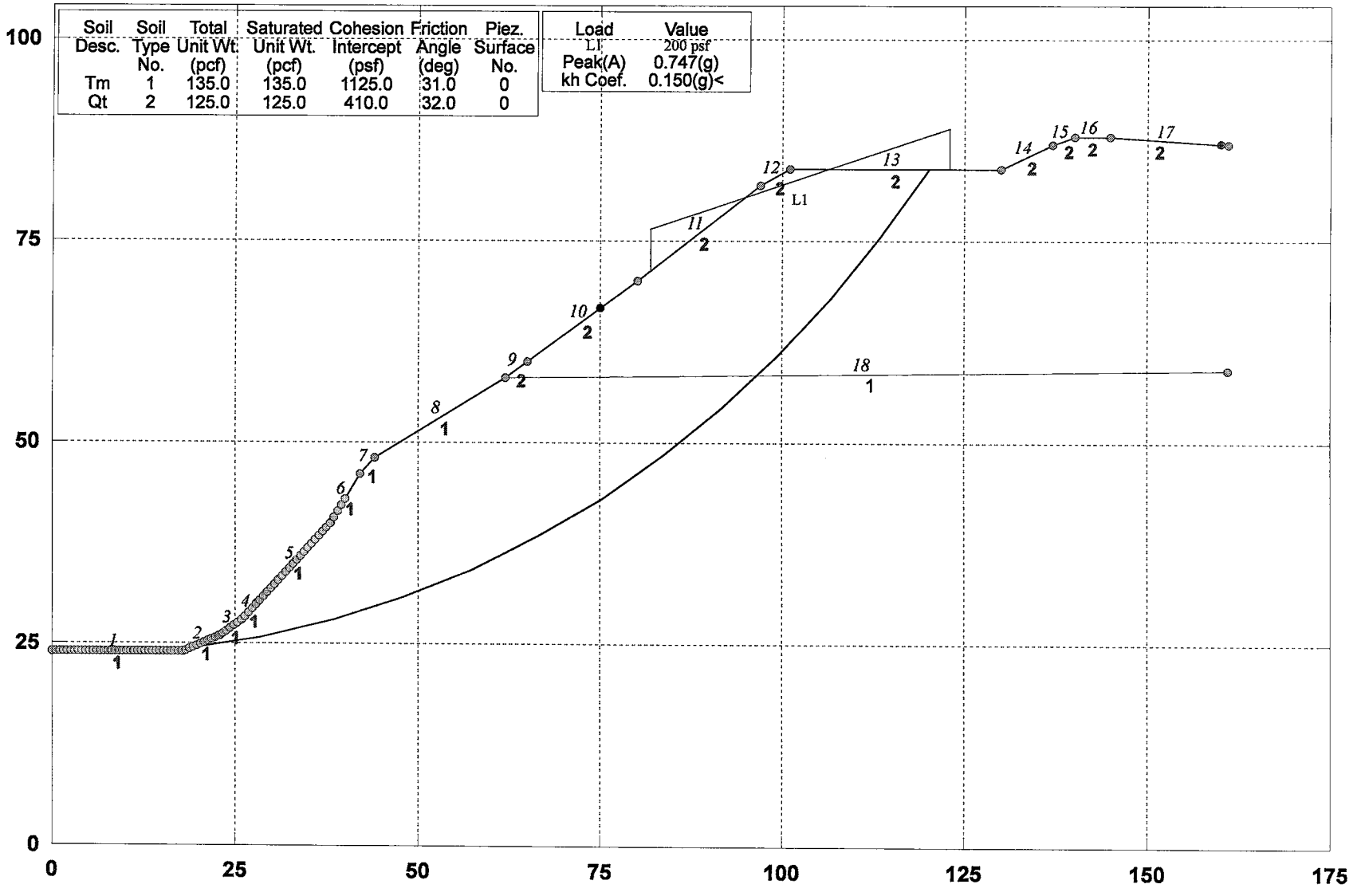
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.3	305.6	0.0	0.0	0.	0.	0.0	0.0	0.0
2	3.0	743.1	0.0	0.0	0.	0.	0.0	0.0	0.0
3	2.0	933.3	0.0	0.0	0.	0.	0.0	0.0	0.0
4	0.6	383.6	0.0	0.0	0.	0.	0.0	0.0	0.0
5	9.4	10735.7	0.0	0.0	0.	0.	0.0	0.0	0.0
6	0.4	640.2	0.0	0.0	0.	0.	0.0	0.0	0.0
7	3.6	7222.7	0.0	0.0	0.	0.	0.0	0.0	0.0
8	2.0	4772.1	0.0	0.0	0.	0.	0.0	0.0	0.0
9	4.0	10132.4	0.0	0.0	0.	0.	0.0	0.0	0.0
10	9.4	25641.4	0.0	0.0	0.	0.	0.0	0.0	0.0
11	4.7	13430.5	0.0	0.0	0.	0.	0.0	0.0	0.0
12	3.0	8849.8	0.0	0.0	0.	0.	0.0	0.0	0.0
13	1.5	4356.8	0.0	0.0	0.	0.	0.0	0.0	0.0
14	8.8	26963.7	0.0	0.0	0.	0.	0.0	0.0	0.0
15	4.7	14733.2	0.0	0.0	0.	0.	0.0	0.0	0.0
16	2.0	6209.3	0.0	0.0	0.	0.	0.0	0.0	0.0
17	1.7	5327.1	0.0	0.0	0.	0.	0.0	0.0	342.5
18	8.1	24780.3	0.0	0.0	0.	0.	0.0	0.0	1613.9
19	4.7	13897.7	0.0	0.0	0.	0.	0.0	0.0	936.2
20	0.5	1559.0	0.0	0.0	0.	0.	0.0	0.0	107.4
21	2.4	6909.4	0.0	0.0	0.	0.	0.0	0.0	485.2
22	1.6	4327.3	0.0	0.0	0.	0.	0.0	0.0	314.8
23	5.6	13259.1	0.0	0.0	0.	0.	0.0	0.0	1121.8
24	6.7	10429.8	0.0	0.0	0.	0.	0.0	0.0	1337.4
25	6.2	3712.2	0.0	0.0	0.	0.	0.0	0.0	1232.0
26	0.6	32.9	0.0	0.0	0.	0.	0.0	0.0	119.4

Seismic Stability of Section B-B' 6927BS

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Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Tm	1	135.0	135.0	1125.0	31.0	0
Qt	2	125.0	125.0	410.0	32.0	0

Load	Value
L1	200 psf
Peak(A)	0.747(g)
kh Coef.	0.150(g)<



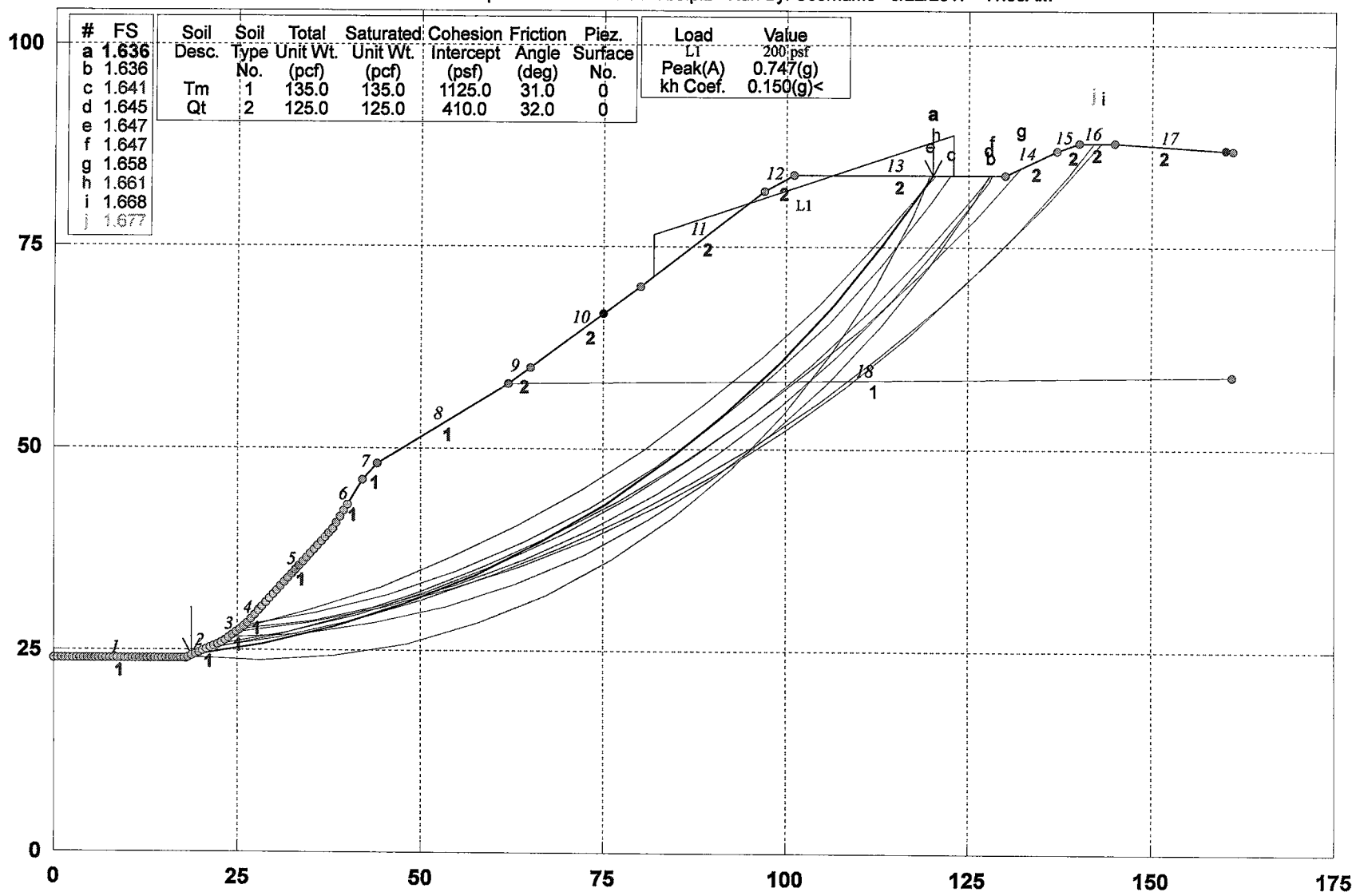
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Safety Factors Are Calculated By The Modified Bishop Method

SubSurface Designs Inc.

Seismic Stability of Section B-B' 6927BS

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#	FS
a	1.636
b	1.636
c	1.641
d	1.645
e	1.647
f	1.647
g	1.658
h	1.661
i	1.668
j	1.677

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Tm	1	135.0	135.0	1125.0	31.0	0
Qt	2	125.0	125.0	410.0	32.0	0

Load	Value
L1	200 psf
Peak(A)	0.747(g)
kh Coef.	0.150(g)<

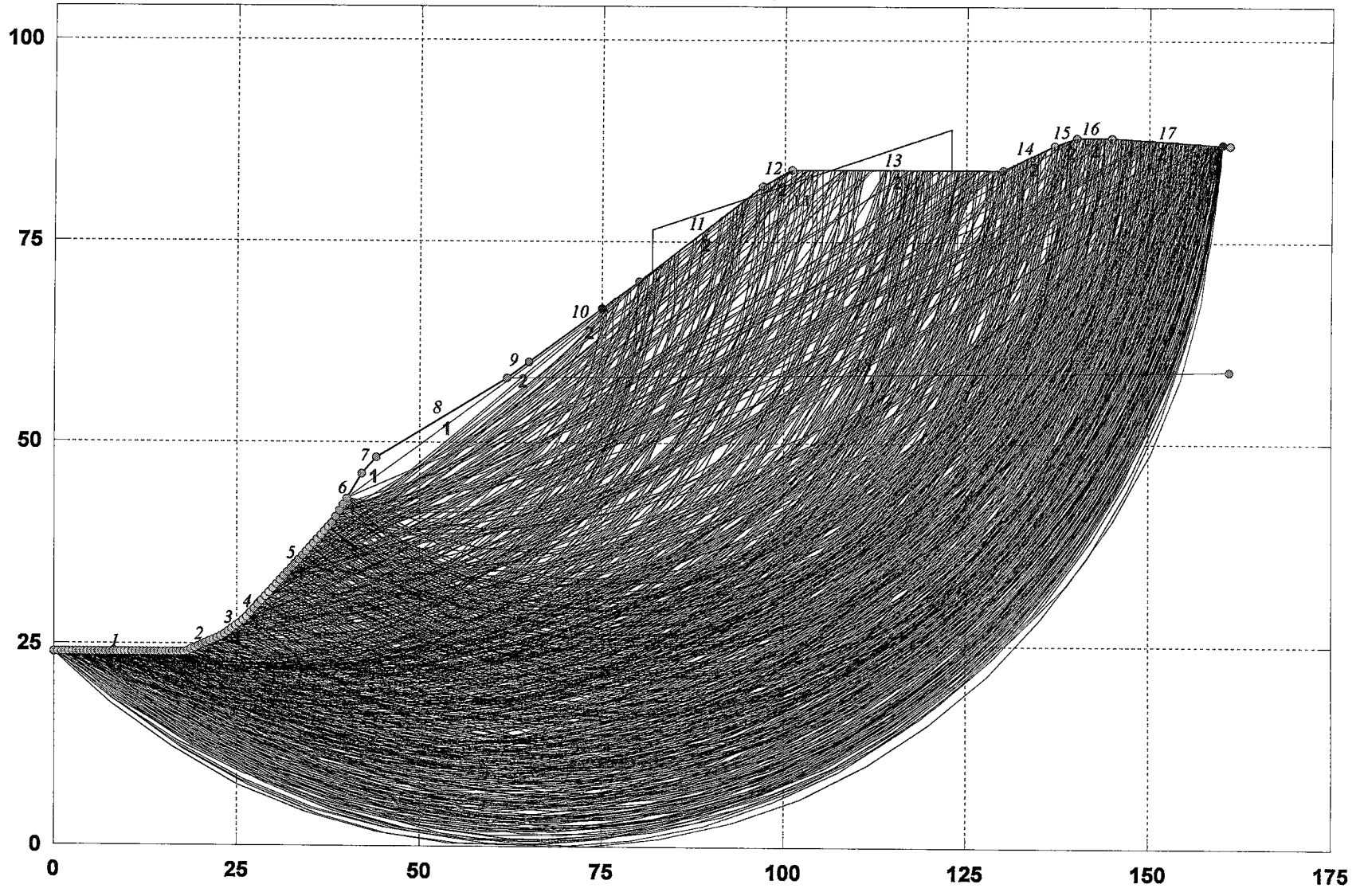
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Safety Factors Are Calculated By The Modified Bishop Method

SubSurface Designs Inc.

Seismic Stability of Section B-B' 6927BS

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SubSurface Designs Inc.

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                                    ***  GSTABL7  ***

                              ** GSTABL7 by Garry H. Gregory, P.E. **

          ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                        (All Rights Reserved-Unauthorized Use Prohibited)


*************************************************************************
                          SLOPE STABILITY ANALYSIS SYSTEM
                Modified Bishop, Simplified Janbu, or GLE Method of Slices.
                (Includes Spencer & Morgenstern-Price Type Analysis)
                Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
                Nonlinear Undrained Shear Strength, Curved Phi Envelope,
                Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
                Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*************************************************************************

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          Time of Run:              11:39AM
          Run By:                   Username
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          Output Filename:          H:\6900\6927 - Simpson\Calculations\6927BS.OUT


          Unit System:              English

          Plotted Output Filename:  H:\6900\6927 - Simpson\Calculati6927BS.PLT



          PROBLEM DESCRIPTION:  Seismic Stability of Section B-B'
                                6927BS



          BOUNDARY COORDINATES

             17 Top   Boundaries
             18 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	24.00	18.00	24.00	1
2	18.00	24.00	23.00	26.00	1
3	23.00	26.00	26.00	28.00	1
4	26.00	28.00	28.00	30.00	1
5	28.00	30.00	38.00	40.00	1
6	38.00	40.00	42.00	46.00	1
7	42.00	46.00	44.00	48.00	1
8	44.00	48.00	62.00	58.00	1
9	62.00	58.00	65.00	60.00	2
10	65.00	60.00	80.00	70.00	2
11	80.00	70.00	97.00	82.00	2
12	97.00	82.00	101.00	84.00	2
13	101.00	84.00	130.00	84.00	2
14	130.00	84.00	137.00	87.00	2
15	137.00	87.00	140.00	88.00	2
16	140.00	88.00	145.00	88.00	2
17	145.00	88.00	161.00	87.00	2
18	62.00	58.00	161.00	59.00	1

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	135.0	135.0	1125.0	31.0	0.00	0.0	0
2	125.0	125.0	410.0	32.0	0.00	0.0	0

BOUNDARY LOAD(S)

1 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	82.00	123.00	200.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.747(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)

Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

800 Trial Surfaces Have Been Generated.

10 Surface(s) Initiate(s) From Each Of 80 Points Equally Spaced
Along The Ground Surface Between X = 0.00(ft)
and X = 40.00(ft)

Each Surface Terminates Between X = 75.00(ft)
and X = 160.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)

10.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Most Critical Of The Trial
Failure Surfaces Evaluated.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Evaluated = 800

Statistical Data On All Valid FS Values:
FS Max = 8.362 FS Min = 1.636 FS Ave = 2.196
Standard Deviation = 0.473 Coefficient of Variation = 21.54 %

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	18.73	24.29
2	28.62	25.79
3	38.38	27.96
4	47.97	30.80
5	57.34	34.29
6	66.45	38.41
7	75.26	43.16
8	83.71	48.49
9	91.78	54.40
10	99.43	60.85
11	106.61	67.80
12	113.30	75.24
13	119.46	83.12
14	120.05	84.00

Circle Center At X = 2.03 ; Y = 168.45 ; and Radius = 145.12

Factor of Safety
*** 1.636 ***

Individual data on the 26 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.3	305.6	0.0	0.0	0.	0.	45.8	0.0	0.0
2	3.0	743.1	0.0	0.0	0.	0.	111.5	0.0	0.0
3	2.0	933.3	0.0	0.0	0.	0.	140.0	0.0	0.0
4	0.6	383.6	0.0	0.0	0.	0.	57.5	0.0	0.0
5	9.4	10735.7	0.0	0.0	0.	0.	1610.4	0.0	0.0
6	0.4	640.2	0.0	0.0	0.	0.	96.0	0.0	0.0
7	3.6	7222.7	0.0	0.0	0.	0.	1083.4	0.0	0.0
8	2.0	4772.1	0.0	0.0	0.	0.	715.8	0.0	0.0
9	4.0	10132.4	0.0	0.0	0.	0.	1519.9	0.0	0.0
10	9.4	25641.4	0.0	0.0	0.	0.	3846.2	0.0	0.0
11	4.7	13430.5	0.0	0.0	0.	0.	2014.6	0.0	0.0
12	3.0	8849.8	0.0	0.0	0.	0.	1327.5	0.0	0.0
13	1.5	4356.8	0.0	0.0	0.	0.	653.5	0.0	0.0
14	8.8	26963.7	0.0	0.0	0.	0.	4044.6	0.0	0.0
15	4.7	14733.2	0.0	0.0	0.	0.	2210.0	0.0	0.0
16	2.0	6209.3	0.0	0.0	0.	0.	931.4	0.0	0.0
17	1.7	5327.1	0.0	0.0	0.	0.	799.1	0.0	342.5
18	8.1	24780.3	0.0	0.0	0.	0.	3717.0	0.0	1613.9
19	4.7	13897.7	0.0	0.0	0.	0.	2084.7	0.0	936.2
20	0.5	1559.0	0.0	0.0	0.	0.	233.9	0.0	107.4
21	2.4	6909.4	0.0	0.0	0.	0.	1036.4	0.0	485.2
22	1.6	4327.3	0.0	0.0	0.	0.	649.1	0.0	314.8
23	5.6	13259.1	0.0	0.0	0.	0.	1988.9	0.0	1121.8
24	6.7	10429.8	0.0	0.0	0.	0.	1564.5	0.0	1337.4
25	6.2	3712.2	0.0	0.0	0.	0.	556.8	0.0	1232.0
26	0.6	32.9	0.0	0.0	0.	0.	4.9	0.0	119.4

APPENDIX IV

REFERENCES

Site References

Area References

Geologic References

Geotechnical References

REFERENCES

Site References

Geologic/soils reports were not on file for the subject site at the time research was conducted.

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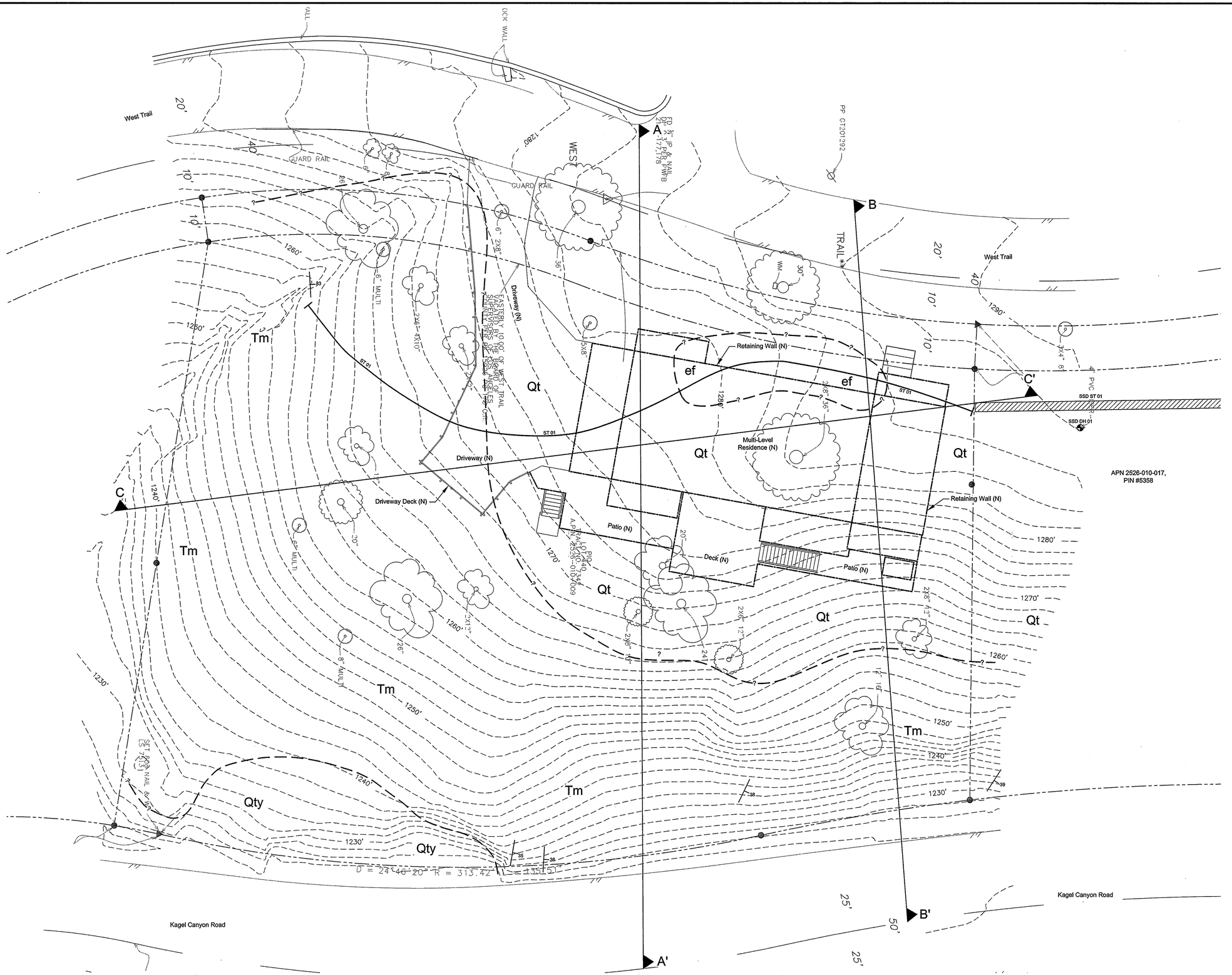
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LEGEND

ef	= Earth fill
Qsw	= Slopewash
Qns	= Natural Soil
Qty	= Younger Terrace Deposits
Qt	= Terrace Deposits
Tm	= Modelo Formation (Miocene geologic age)
ST 01	= Approximate alignment of seismic trench
	= Approximate location of boring (SubSurface Designs, Inc. 2008)
22	= Strike and dip of bedding
	= Approximate geologic contact
	= Geologic cross-section
	= Approximate location of seismic trench (SubSurface Designs, Inc. 2008)

0 5 10
SCALE

NOTE: This document has been created from a digital topographic base map prepared by Clemons Land Surveying, dated July 7, 2015; and preliminary plans prepared by Keith Dewey of Zigloo.ca, dated February 8, 2017. Analyses and/or professional opinions generated from this plan are only as accurate as the plan(s) provided to this office. If discrepancies are found to exist between the plan(s) and the actual site conditions, they should be brought to our immediate attention so that revisions may be made as required.

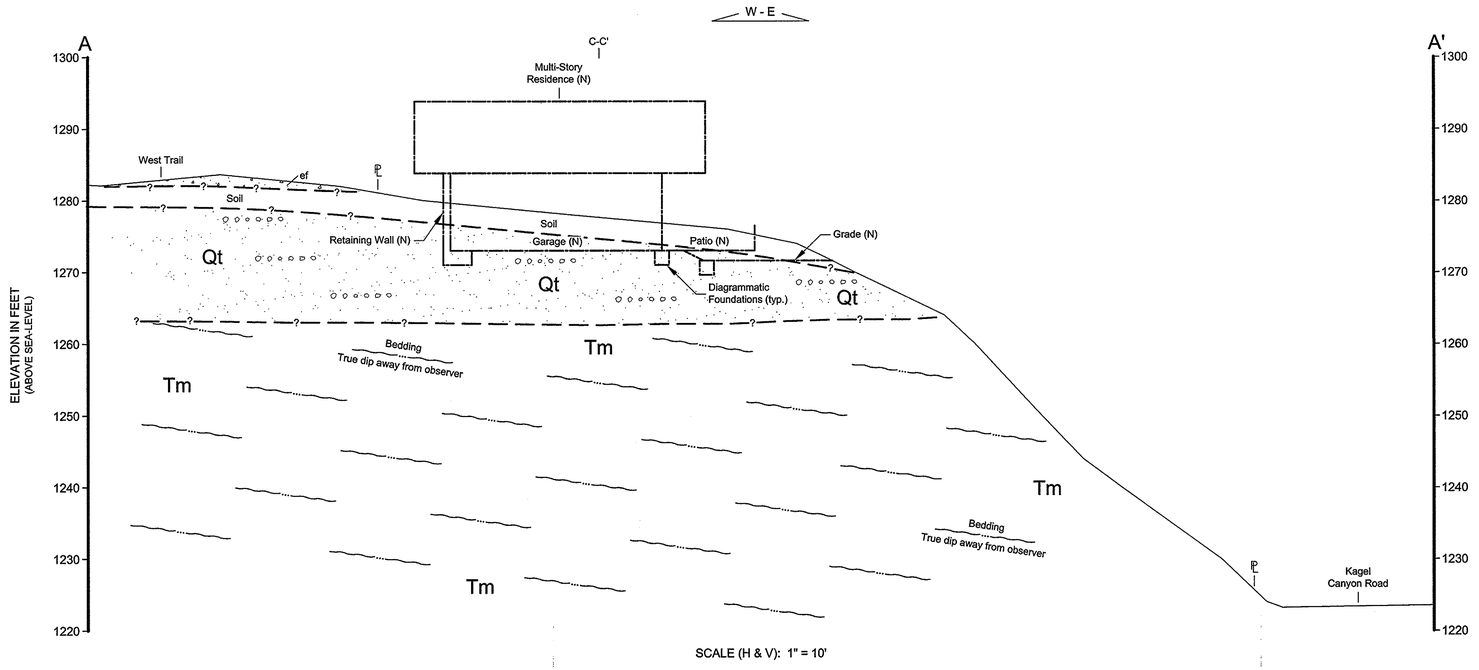
SubSurface Designs Inc.
12848 Foothill Boulevard, Sylmar, CA 91342
(818) 898-1595 or (Fax) 898-4003

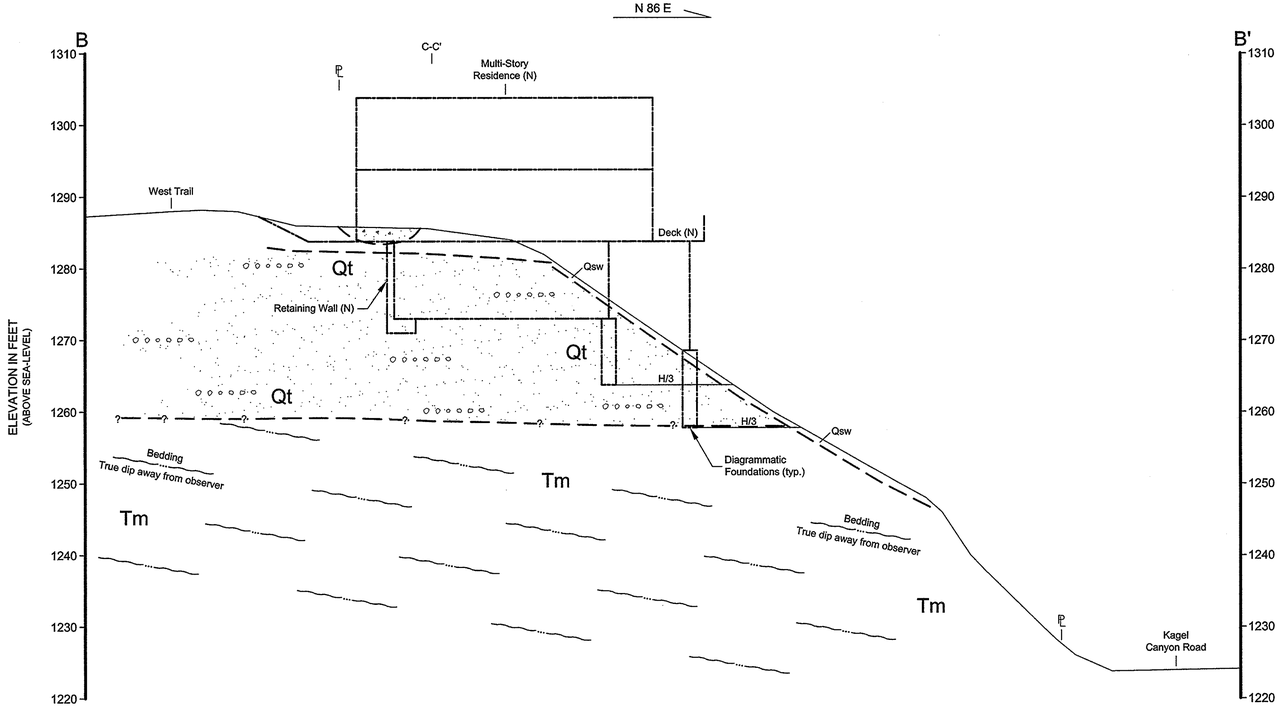
Simpson
APN 2526-010-009 - Lot 44, Tract 7342
West Trail, Kagel Canyon
Pin# 6927.01S

Geologic Map

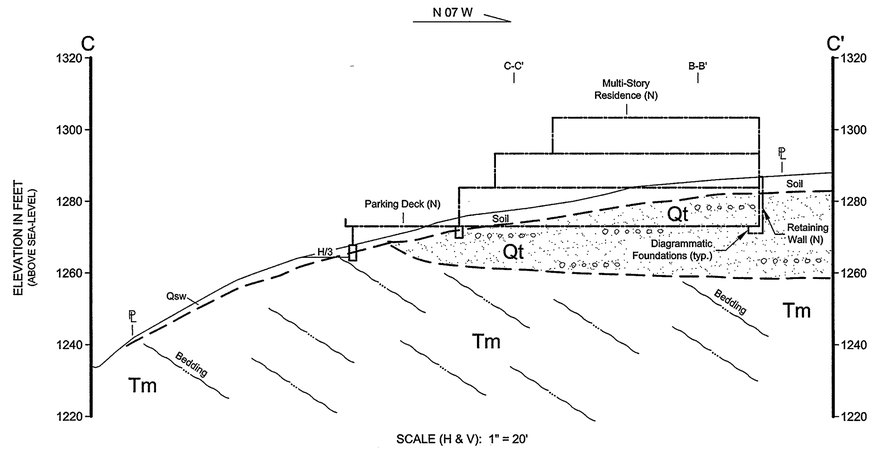
September, 2017

Plate A





The foundation depths shown on Geologic Cross Sections are minimum embedment depths required by this office. However, the project structural engineer may need to make the foundation depths deeper to accommodate specific structural loads.



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12848 Foothill Boulevard, Sylmar, CA 91342
(818) 898-1595 or (Fax) 898-4003

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West Trail, Kagel Canyon
Pin# 6927.01S

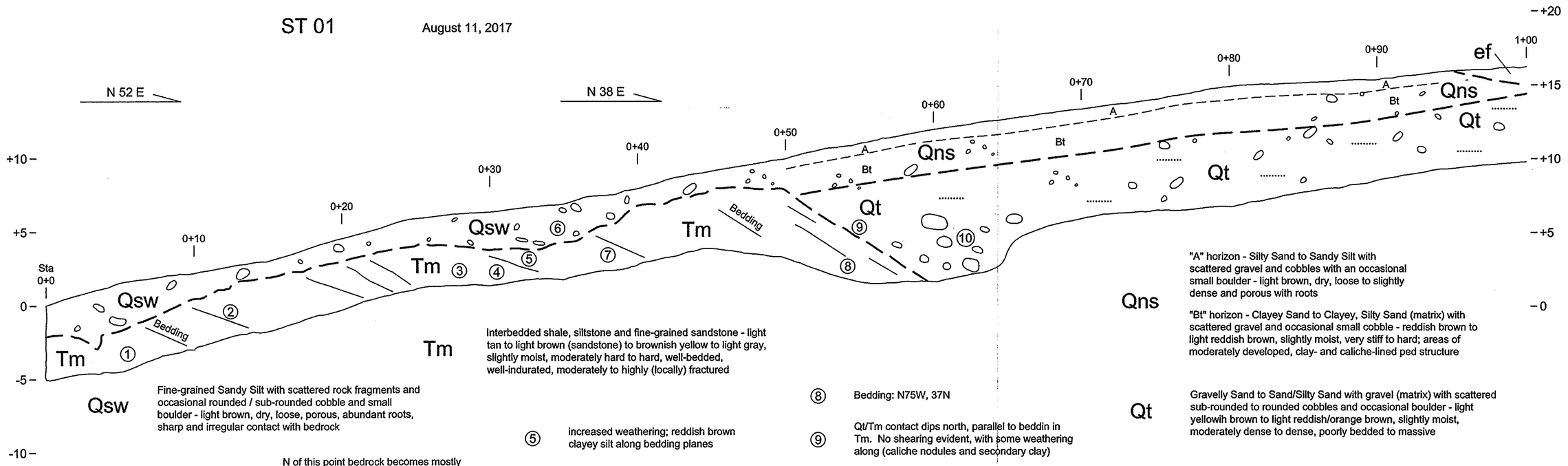
Geologic Cross-Sections

September, 2017

Plate B

NOTE: This document has been created from a digital topographic base map prepared by Clemons Land Surveying, dated July 7, 2015; and preliminary plans prepared by Keith Dewey of Zigloo.ca, dated February 8, 2017. Analyses and/or professional opinions generated from this plan are only as accurate as the plan(s) provided to this office. If discrepancies are found to exist between the plan(s) and the actual site conditions, they should be brought to our immediate attention so that revisions may be made as required.

ST 01 August 11, 2017



Qsw — Fine-grained Sandy Silt with scattered rock fragments and occasional rounded / sub-rounded cobble and small boulder - light brown, dry, loose, porous, abundant roots, sharp and irregular contact with bedrock

Tm — Interbedded shale, siltstone and fine-grained sandstone - light tan to light brown (sandstone) to brownish yellow to light gray, slightly moist, moderately hard to hard, well-bedded, well-indurated, moderately to highly (locally) fractured

Qns — "A" horizon - Silty Sand to Sandy Silt with scattered gravel and cobbles with an occasional small boulder - light brown, dry, loose to slightly dense and porous with roots

"Bt" horizon - Clayey Sand to Clayey, Silty Sand (matrix) with scattered gravel and occasional small cobble - reddish brown to light reddish brown, slightly moist, very stiff to hard; areas of moderately developed, clay- and caliche-lined ped structure

Qt — Gravelly Sand to Sand/Silty Sand with gravel (matrix) with scattered sub-rounded to rounded cobbles and occasional boulder - light yellowih brown to light reddish/orange brown, slightly moist, moderately dense to dense, poorly bedded to massive

① Bedding: N73E, 33NW

② Bedding: N84W, 43N

③ N of this point bedrock becomes mostly siltstone with occasional thin, fine-grained sandstone interbed, closely fractured, hard, slightly to moderately weathered

④ bedrock very closely fractured

⑤ increased weathering; reddish brown clayey silt along bedding planes

⑥ deeper Qsw pocket with secondary clay development

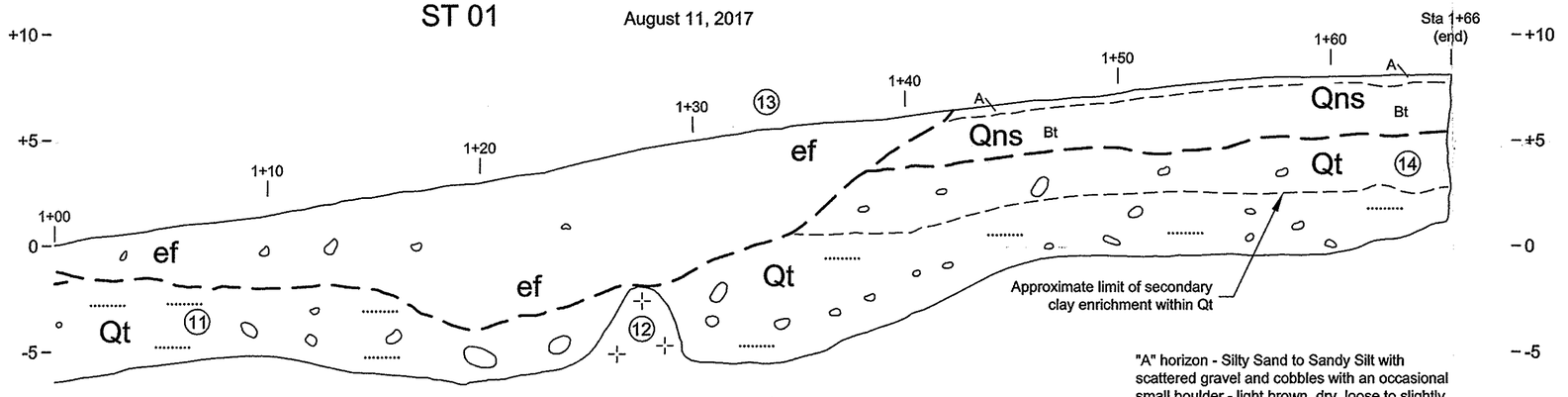
⑦ Bedding: N56W, 26N

⑧ Bedding: N75W, 37N

⑨ Qt/Tm contact dips north, parallel to beddin in Tm. No shearing evident, with some weathering along (caliche nodules and secondary clay)

⑩ Granitic and gneissic boulders to 3' across, increased concentration of large clasts along contact with Tm

ST 01 August 11, 2017



ef — Silty Sand with scattered to abundant gravel, cobbles and occasional boulder - brown to dark brown, slightly moist, slightly to moderately compact with occasional bit of construction debris (asphalt, brick, concrete)

Qns — "A" horizon - Silty Sand to Sandy Silt with scattered gravel and cobbles with an occasional small boulder - light brown, dry, loose to slightly dense and porous with roots

"Bt" horizon - Clayey Sand to Clayey, Silty Sand (matrix) with scattered gravel and occasional small cobble - reddish brown to light reddish brown, slightly moist, very stiff to hard; areas of moderately developed, clay- and caliche-lined ped structure

Qt — Gravelly Sand to Sand/Silty Sand with gravel (matrix) with scattered sub-rounded to rounded cobbles and occasional boulder - light yellowih brown to light reddish/orange brown, slightly moist, moderately dense to dense, poorly bedded to massive

⑪ Coarse-grained Gravelly Sand with occasional cobble - light red/orange brown to light yellowish brown, laminated with crude sub-horizontal bedding

⑫ Large granitic boulder; unable to remove

⑬ On E trench wall at ~ Sta 1+33, inverted and infilled 32" dia. concrete (?) pipe to 2' above trench bottom

⑭ Gravelly Sand with clay binder - reddish brown, hard, massive

SCALE (H & V): 1" = 5'

SubSurface Designs Inc.
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(818) 898-1595 or (Fax) 898-4003

Simpson
APN 2526-010-009 - Lot 44, Tract 7342
West Trail, Kagel Canyon
Pin# 6927.01S

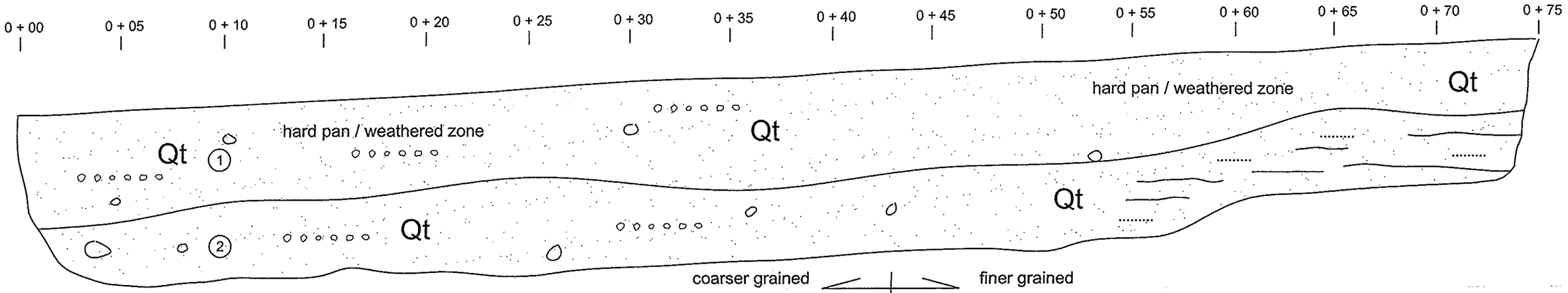
Seismic Trench ST 01

September, 2017

Plate C

SSD ST 01

S - N



coarser grained — finer grained

SCALE: 1" = 5'

(1) Dark to medium brown silty sand and gravel.

(2) Light to medium brown gravelly sand.

Qt - Terrace Deposits - dark-brown to light-brown, silty fine to coarse sand and gravel, abundant cobbles, some boulders, moist, medium dense to dense.

NOTE: This document has been created from a digital topographic base map prepared by Ray Lombera and Associates, Inc. Analyses and/or professional opinions generated from this plan are only as accurate as the plan(s) provided to this office. If discrepancies are found to exist between the plan(s) and the actual site conditions, they should be brought to our immediate attention so that revisions may be made as required.

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Julian

APN 2526-010-017 West Trail, Kagel Canyon

Ref # 5358.01S

Seismic Trench ST 01

April, 2008

Plate C