APPENDIX F – GEOTECHNICAL REPORTS

Preliminary Geotechnical Investigation,
Proposed Everett Street Terraces Apartment Complex,
Northeast Corner of Everett Street and Walnut Canyon Road,
Moorpark, California

W.O. 8953
December 2, 2005

TABLE OF CONTENTS

ENCLOSURE LIST:

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December 2, 2005
W.O. 8953

John W. Newton & Associates, Inc.
165 High Street, Suite 103
Moorpark, California 93021

Attention: John Newton

SUBJECT: Preliminary Geotechnical Investigation of 2-Acre Parcel, Northeast Corner of Everett Street and Walnut Canyon Road, Moorpark, California

Mr. Newton:

In accordance with your request, our firm has undertaken a study of the geotechnical conditions at the subject property (Plate 1.1). Our purpose was to evaluate the distribution and engineering characteristics of the earth materials that occur at the site so that we might assess their impact upon the proposed development of the property.

The scope of work for this project included the following tasks:

- mapping of the site and its immediate vicinity;
- logging of three (3) Cone Penetrometer Tests (CPT) soundings;
- logging and sampling of four (4) exploratory borings excavated with a truck-mounted hollow-stem auger;
- logging and sampling of two (2) exploratory borings excavated with a truck-mounted bucket-auger drill rig and one (1) exploratory boring excavated with a track-mounted limited access bucket-auger drill rig;
- selected laboratory testing of the retrieved samples;
- review of previous work which was judged both pertinent to our purpose and readily available to our office;
- soil engineering analysis of the assembled data;
- preparation of this report.

Field data and the approximate locations of exploratory excavations are shown on the enclosed Geologic Map (Plate 1.2). Descriptions of the materials encountered in the exploratory excavations are provided on the enclosed logs (Plates B1.1 to B7, and CPT1 to CPT3). Pertinent laboratory test results are also provided herein. Our findings are presented in the following sections, followed by a discussion of these findings and geotechnical recommendations.

SITE DESCRIPTION

The project site consists of an approximate 2-acre parcel of land on the northeast corner of Everett Street and Walnut Canyon Road in the City of Moorpark, California. The land is presently partially developed with approximately one dozen older residential structures plus minor remnants of one or two demolished structures that originally faced Everett Street. The southern three-fourths of the site consists of gently southerly sloping land with a gradient of approximately 15:1 (horizontal:vertical) and sparse vegetation. The northern one-fourth of the site consists of moderate to steep southeast to southwest facing vegetated slopes with gradients ranging from 3:1 (H:V) to as steep as 1:1 (H:V) locally.

Regionally the site is located on the northern margin of Little Simi Valley and ranges in elevation from 530 to 585 feet above Mean Sea Level (MSL). The Arroyo Simi drains Little Simi Valley from east to west and is located approximately 1.9 miles south of the site at approximate elevation 490 feet above MSL.

PREVIOUS GEOTECHNICAL STUDIES

No record of previous geotechnical studies on the subject site was found in the City's files and we are not aware of any previous studies performed on the site. Geotechnical investigations performed for nearby projects were reviewed as part of this investigation. These include studies for Tentative Tracts 5505 and 5130, Tracts 5045 and 5187, and studies for smaller projects including 251 Moorpark Road and 180 Wicks Road. Pertinent geotechnical reports from those projects are listed in the attached References.

PROPOSED PROJECT

Based upon project drawings received from the architect and civil engineer, we understand that the proposed project consists of a terraced complex of 44 apartments with two levels of partially subterranean to subterranean parking. Site access will be from Everett Street near the intersection with Walnut Canyon Road. Retaining walls up to 24 feet in height are proposed. Terraced pads will occur at approximate elevations 533', 544', 555' and 564'. Given this configuration the greatest structural loads are anticipated to occur in the northern portion of the project where two stories of apartments will be constructed atop two stories of partially subterranean parking. The highest proposed fill slope is approximately 6 feet, fronting Everett Street. No permanent cut slopes are proposed. The preliminary grading plan by Stantec indicates raw cut and fill volumes of 11,204 cubic yards and 10,243 cubic yards, respectively.

At the time of this writing, specific foundation loads, nor specific foundation locations or types are known. For purposes of this preliminary report, we have assumed that maximum column loads will be on the order of 250 kips, and maximum wall loads will be on the order of 2 to 6 kips per linear foot of wall.

FIELD INVESTIGATION

Our office selected several exploratory locations and several methods of exploration in order to characterize the nature of the earth materials throughout the site.

The subsurface exploration began on June 29, 2004 with performing three cone penetrometer (CPT) soundings. The soundings were performed using a 23-ton truck-mounted CPT rig provided by Holguin, Fahan & Associates, Inc. The cone tip has a cross-sectional area of 10 square centimeters. The CPT is capable of obtaining tip pressure and side friction data at 2 inch (0.05 meter) intervals. The cone tip was pushed to a depth of approximately fifty feet or refusal, whichever was shallower.

Exploratory borings were performed by three types of drill rigs. Borings B1 through B4 extended into the alluvium underlying the site. These borings were performed with a hollow-stem auger drill rig. Samples were driven with a 140 lb. hydraulic winch safety hammer lifted 30 inches. The estimated efficiency of the hydraulic winch hammer is approximately 68 percent (Kovacs et.al. 1978]). Drilling rod was not used, the sampler and hammer was suspended by cable. The boring diameter was approximately eight inches (outer diameter). The samplers consisted of an SPT Split Spoon Sampler and a lined California split spoon sampler (2.375 inch id.).

Borings B5 and B6 were excavated with a truck-mounted bucket-auger drill rig and Boring B7 was excavated with a track-mounted limited access bucket-auger drill rig. Samples were driven with the drill rig Kelly bar lifted with a hydraulic winch dropped 12 inches, except for Boring B7 which was dropped 15 inches. The estimated efficiency of the hammer is approximately 68 percent (Kovacs et.al. 1978). Drilling rod was not used, the sampler and hammer were suspended by cable. The boring diameter for borings B5-B7 was approximately 24 inches. The samplers consisted of a lined California Split Spoon sampler (2.375 inch id.).

Both disturbed (bulk) and relatively undisturbed samples were obtained from each boring. These samples were secured and transported to our laboratory for testing.

GEOLOGIC SETTING

The site is located in the Transverse Ranges geomorphic province of Southern California. The Transverse Ranges are essentially east-west trending elongate mountain ranges and valleys that are geologically complex. Structurally, the province reflects the north-south compressional forces that are the result of a bend in the San Andreas fault. As the Pacific Plate (westerly side of the fault) and the North American Plate (easterly side) move past one another along the fault the bend creates a deflection which allows for large accumulations of compressional energy. Some of these forces are spent in deforming the crust into roughly east-west trending folds and secondary faults. The most significant of these faults are typically reverse or thrust faults, which allow for the crustal shortening taking place regionally.

The site lies in the central portion of the Transverse Ranges province, in the City of Moorpark. The site is situated at the base of a range of low foothills that define the northern margin of Little Simi Valley. The foothills are underlain by Quaternary-age sediments (Saugus Formation) that were deposited in a fluvial/floodplain environment that has subsequently been uplifted and eroded. The valley margin is underlain by alluvial deposits which thicken considerably toward the south into the broad floodplain of Arroyo Simi.

EARTH MATERIALS

The subject property is underlain by alluvium and Saugus Formation bedrock (see Plate 1.2). Due to past development of the site, minor thin artificial fill may be present; however, artificial fill was not encountered in the subsurface exploration and is not present in significant quantity to be a mappable earth unit. A brief description of each material is provided in the following sections.

Artificial Fill (Af)

Fill was encountered to a depth of approximately 5 feet in offsite boring B7 which was drilled on the Bowen property between Wicks Road and the subject property. This material consisted of dark brown fine to coarse grained sand with minor amounts of clay and silt, in a moist, medium dense to dense condition. This fill is presumably associated with the construction of Wicks Road.

Alluvium (Qal)

Alluvium was encountered in all three CPT soundings as well as in borings B1 through B6. The alluvium consists predominantly of fine to coarse grained silty sand with infrequent lenses and strata of gravelly sand, clayey sand, silt and clay. An exception is a stiff clayey silt unit approximately 5 feet thick between 11-16 feet in Boring B1 which correlates well with CPT1, CPT2, CPT3 and B3. The alluvial soils tested in our laboratory have dry densities ranging from 104 to 123 pcf and moisture contents ranging from 1.8 to 18.2 percent.

Saugus Formation Bedrock (TQs)

Bedrock of the Plio-Pleistocene age Saugus Formation underlies the hillside terrain and alluvial deposits at the site. Saugus Formation was encountered in borings B3, B5, B6 and B7. The lithologies encountered include fine to coarse grained sandstone, gravelly sandstone and minor conglomeratic sandstone, with less prevalent interbeds of siltstone and clayey siltstone. Claystone was absent with the exception of a 4-inch thick bed in Boring B5. The coarser grained units were typically dense, weakly cemented to uncemented, and friable. These units frequently exhibited scoured irregular contacts, internal scours and channel fills, and crude bedding to well-defined cross bedding. Finer grained units were typically stiff and massive. Overall we would characterize the Saugus Formation in this location to be very thinly to thickly bedded.

GEOLOGIC STRUCTURE

The site is located on the far southern flank of a regional structure originally described as the Moorpark Anticline (Weber, 1973; Dibblee, 1992) which is a broad, possibly asymmetrical structure occupying the foothills north of Little Simi Valley and downtown Moorpark. Numerous studies conducted on sites to the west, northwest, north and northeast suggest that this broad anticlinal structure is folded and faulted, and that gentle lower-order folds and warps are superimposed upon the larger anticlinal structure. Pertinent geologic structural data surrounding the site include Tentative Tract 5505 approximately 850 feet northwest (GWV, unpublished), and Tentative Tract 5130 approximately 1250 feet to the northeast (Gorian, 1998). These data suggest an overall gentle west-southwesterly dip, ranging from 2 to 8 degrees, for the Saugus Formation in the area of the subject site. Although the data are being projected across significant distances, the data are projected from different directions (northeast and northwest) and when viewed in conjunction with the site specific data, suggest that the overall structure across the area is somewhat consistent with gentle southwest dips.

Site-specific geologic data were obtained from the downhole logging of Borings B5 through B7 and these data are illustrated on Plate 1.2. Attitudes were measured on bedding planes, and in some cases on cross-bedding and scoured contacts. It should be noted that the cross-bedding and scoured contact attitudes are not representative of the overall geologic structure of the site vicinity. In general, bedding attitudes measured in the borings ranged in strike from N55E to N45W, with gentle dips to the northwest, west and southwest. The boring data for B5 revealed some northwest to northeast dipping Saugus Formation structure. A sharp moderately southwest-dipping contact identified as a possible fault was logged at 43.5 feet in B5; however, displacement could not be measured and no clayey shear surface was identified. Below this feature bedding structure changed to gentle southwesterly dips.

Local Faulting

Fault investigations performed on Tract 5045 (PML, 1996, 1997) northeast of the subject site identified two significant fault features (termed the *Northern Area Thrust* and *Southern Area Thrust*) and concluded that these features were active faults but not seismogenic structures (i.e., not deeply rooted into the regional tectonic framework and not capable of individually producing earthquakes), rather, they were secondary fault features that originated along bedding planes at relatively shallow depths during folding (aka "bending moment structures"). The faults were modeled as features associated with active deformation (i.e., folding and warping in response to north-south regional compression and uplift) that presumably occurs co-seismically with events on either the Oak Ridge Fault or Simi-Santa Rosa Fault which bracket the area to the north and south, respectively. The *Northern Area Thrust* was concluded to be a blind thrust fault that warped older alluvial sediments that were younger than 50,000 years (ECI, 1997). The *Southern Area Thrust*, a north-dipping feature, was interpreted by PML to be an active fault based upon geomorphic expression (lineament) and displacement of older alluvial sediments; however the *Southern Area Thrust* was never studied further on that site--apparently due to designation of that area of the site as Open Space. We are aware that representatives of the State Geologist (Mr. Jerry Treiman) reviewed the fault trenches on Tract 5045; however, the State chose to not zone the features under the Alquist-Priolo Earthquake Fault Zoning Act program.

A fault investigation on Tract 5187 (GWV, 1999), which is approximately 2500 feet north of the subject property, encountered a north-dipping thrust fault which may well be a southwesterly extension of the *Northern Area Thrust* from Tract 5045. This investigation concluded that the observed fault did not displace sediments on the order of 15 ka to 20 ka, and therefore the fault was not considered active under the State's criteria (GWV, 1999; Shlemon, 1999).

A fault investigation on Tract 5130, north of the subject site, was performed by Gorian & Associates, Inc. (GAI, 1998). The investigation was based upon a geomorphic lineament traversing the property approximately 500 to 600 feet north of Wicks Road. This lineament appeared to be a westerly projection of the *Southern Area Thrust* lineament from Tract 5045. A prominent south-dipping reverse fault was encountered in two trenches on Tract 5130. The projected surface trace of this feature is approximately 750 feet north of the subject site. GAI concluded that the fault observed was a bending moment or back-thrust feature, and that due to its apparent association with the *Southern Area Thrust* to the east (although oriented differently), it should also be considered active. The fault was found to displace Saugus Formation bedrock and warp and displace an older alluvial unit described as Qoal2. Studies by others (ECI,1997) estimated the Qoal2 unit on adjacent Tract 5045 to range in age from 80 ka to 130 ka (80,000 to 130,000 years before present). A younger alluvial unit described as Qoal3 (and estimated by ECI to be younger than 50 ka on adjacent tract 5045) was not displaced or warped in the GAI trenches. Nevertheless, GAI assumed the fault to be active and recommended building setbacks from this feature.

GROUNDWATER

Groundwater was not encountered to the depths explored (51.5 feet bgs in the alluvium, 70 feet bgs in the Saugus Formation).

We have reproduced Plate 1.2 from CDMG Open-File Report 2000-007 (Seismic Hazard Zone report for the Moorpark Quadrangle) to illustrate the site's location at the edge of the alluviated valley. This figure also illustrates historical high groundwater that has been encountered in the alluviated valley, generally south of the subject site. At the southern end of the subject site, groundwater was not encountered in the CPT soundings or borings which were extended to 51.5 feet below ground surface, with the southern-most boring (B1) extending to an elevation of approximately 477 feet above mean sea level (from a surface elevation of 529 feet).

Approximately 2500 feet south of the site, at a similar point in time, GWV encountered groundwater at elevation 465 feet from a surface elevation of approximately 503 feet in September 2004 (GWV, 2004).

Research was performed at the County of Ventura to obtain information regarding the history of groundwater in the area. Our research indicates there are two wells in Moorpark with a significant history of water depth readings. They are well 02N19W05K001S and 02N19W04K001S. Several others have information dating back a decade or so. The following table summarizes the groundwater information:

Well Designation	Date Drilled	GS Elevation	GW Elevation	Comments
02N19W05K001S	06/1975*	497'	361' to 469'	GW highest after 1985 (27')
4K001S	10/1950	530'	290' to 500'	GW highest after 1985 (29')
4H	07/1995	543'	530.5'	Near Arroyo Simi (12.5')
4K	09/1991	550'	None	(>50')
4M	12/1989	520'	485' to 488.5'	West of Moorpark Rd. (31.5')
4M	11/2000	520'	469.5' to 471'	(49')
4M	06/2002	520'	470'	412 High St. (50')
9B	07/1988	500'	467' to 470'	Spring & New L.A. Ave. (30')

* Denotes date of 1st reading

Based on well data, the groundwater has been rising in the last half of the 1900's and has leveled off since significant development occurred in Moorpark in the mid 1980's. In addition, we feel it can be reasonably concluded that groundwater has not historically risen above approximate elevation 490' at the site, nor immediately south of the site.

Historic high groundwater is indicated on Plate 1.2 in the SHZ report to be about 20 feet below the ground surface in the alluviated valley south of the site. However, it should be noted that as the topography rises at the valley margin, a 40-foot below ground surface groundwater contour is illustrated in some locations (CDMG, 2000). Based upon the historic groundwater information as well as the site elevation range of 530 to 580 feet above sea level (which at the south end is roughly 25 feet above the valley floor), it can be reasonably concluded that the subject site falls into this category. As such we have assumed historic high groundwater to be 40 feet below the ground surface for the purposes of liquefaction analysis.

FAULTING AND SEISMICITY

The subject site contains no known active or potentially active faults, nor is it within an Earthquake Fault Zone designated along faults judged sufficiently active and well defined by the State Geologist. Local faults which appear to be secondary, bending-moment structures encountered north and northeast of the site, are described above in the Geologic Structure section. The closest of these known faults is some 750 feet north of the subject property and poses no ground rupture hazard to the subject property. Therefore, the potential for ground rupture is considered to be very low. However, the property is situated within the seismically active Southern California region and ground shaking is likely to occur due to earthquakes caused by movement along nearby faults.

One method of seismic design is to utilize the Static Force procedure (structures less than five stories) presented in the Uniform Building Code (UBC), which can be used to estimate base shear/on-site acceleration based upon site location, occupancy classifications, and the planned structural system. For the 1997 UBC this site has a Seismic Zone Factor, Z of 0.4 (Tbl 16-I). The Soil Profile Type is considered S_D (Tbl 16-J). The Seismic Source Type is considered B (Tbl 16-U) for the Simi-Santa Rosa fault, and the Near Source Factors are estimated as N_a =1.3 and N_v =1.6 (Tbl 16-S & 16-T). These values were derived from the computer program UBCSEIS. The UBCSEIS output is included in Appendix A.

Another method of seismic design is to assess the potential on-site ground acceleration based upon a site's proximity to specific, known faults. This relies upon prediction of a maximum earthquake for each fault considered, relationships that characterize the diminution of ground response with distance from the causative event, and relationships that assess impact of site-characteristics upon ground response. Two commonly used methods of estimating possible on-site accelerations are the deterministic seismic hazard analysis method (DSHA) and the probabilistic seismic hazard analysis method (PSHA). The deterministic method is of interest in evaluating how individual faults affect the site, and when a design is based on a deterministic analysis (such as a Caltrans structure). The probabilistic methods are of interest in evaluating the design-basis earthquake as prescribed by the UBC. The probabilistic type analyses can be further separated into a site specific PSHA and a probabilistic analysis using the "Simple Prescribed Parameter Value" Method (SPPV). The results of probabilistic analyses using these methods are discussed below. Analysis summaries are attached in Appendix A, Seismic Analyses.

Probabilistic "Simple Prescribed Parameter Value" (SPPV)

We have employed the "Simple Prescribed Parameter Value" Method (SPPV) for estimating the peak ground acceleration (PGA) for a 10 percent exceedance probability for an exposure period of 50 years (UBC Design-Basis Earthquake, 475 year return period). As discussed in CGS Seismic Hazard Evaluation Reports, the attenuation relationships of Boore et.al. (1997), Campbell (1997), Sadigh et.al. (1997), and Youngs et.al. (1997) were utilized to generate PGA maps. We have reproduced Figure 3.3 of CDMG Open File Report 2000-007 (for the Moorpark 7.5 Minute Quadrangle) in Appendix A to illustrate the project location with respect to SPPV PGA values for alluvial conditions. A peak ground acceleration of 0.69g is estimated for a UBC design-level event. A design earthquake magnitude of M_w=6.9 is the predominant earthquake, per Figure 3.4 of Open File Report 2000-007.

Seismic Discussion

The methodology in the Uniform Building Code has been to protect and preserve life and limb. Building designs using previous UBC codes (pre-1997) has apparently been successful in that regard. With the acceptance of the 1997 UBC, the seismic design of structures has generally become more conservative. On that basis, we recommend minimum structural design be in compliance with the seismic design provisions of the UBC. Though still not performance based, this most recent Building Code will enhance performance over designs based on previous codes.

Design per the UBC (and hence adoption of the philosophy that life and limb need be protected) is commensurate with the local building ordinance. Being that higher standards of design (i.e. that intend to minimize property damage in the case of a much less likely event) have not been adopted by the governing agency (which is responsible for setting such standards), use of a higher acceleration (than provided by the UBC) is discretionary.

LABORATORY TESTING

Undisturbed and bulk samples of soil and rock materials encountered at the site were collected during the course of our fieldwork. Selected laboratory tests completed on the retrieved samples are described below. A comprehensive summary of laboratory test results is provided in Plate LS in Appendix B.

Moisture-Density

The field moisture content and dry unit weight were determined for each undisturbed sample. Dry unit weight is expressed in pounds per cubic foot and the moisture content represents a percentage of the dry unit weight. This test data is presented in the attached boring logs.

Compaction and Expansion Tests

To determine the compaction characteristics of the onsite materials, compaction tests are performed in accordance with ASTM D 1557-00. The maximum dry density is reported in pounds per cubic foot and the optimum moisture content as a percentage of the maximum dry density. Expansion index tests were performed in accordance with the criteria in U.B.C. 18-2. The results of these tests are included below in Table I.

Laboratory Test Data – Table I

Sample	Description	Maximum Dry Density (PCF)	Optimum Moisture Content (%)	Expansion Index
B1@0-3'	Silty Sand	129.0	8.0	0

Shear Test

Shear tests were performed in a Direct Shear Machine of the strain control type. The rate of deformation is approximately 0.01 inches per minute. Shearing occurred under a variety of confining loads in order to determine the Coulomb shear strength parameters. The test was performed on undisturbed and remolded (@ 90% relative compaction) samples in an artificially saturated condition. The test results are presented graphically on Plates S-B1.0 to SB6.50).

Consolidation Test

Settlement predictions of the soil's behavior under load are made on the basis of consolidation tests. A one-inch high sample is loaded in a geometric progression and the resulting deformation is recorded at selected time intervals. Porous stones are placed in contact with the sample (top and bottom) to permit addition and release of pore fluid. The sample is inundated at a selected load during the progression. Selected samples had data recorded at timed intervals for specific loads to obtain data for time-rate evaluations. Results are plotted on the enclosed Consolidation-Pressure Curves (Plates C-B1.10-C-B6.30).

Particle Size Analysis

The distribution of various particle sizes in selected representative samples was determined using both mechanical sieves and hydrometer tests. The percentage and distribution of particles larger than a #200 sieve (0.075 mm) are determined using mechanical processes. Particle distributions for fine-grained soils are determined using hydrometer methods. The particle distribution is presented as the relative percentages of sand, silt and clay particles in each sample tested. The results are presented on the attached boring logs and on Plate PS.1.

Liquid Limit, Plastic Limit, and Plasticity Index

The Liquid Limit, Plasticity Limit, and Plasticity Index for selected cohesive soil samples were determined in the laboratory. The Standard Test Method (ASTM D4318-84) was utilized. These parameters are used in the classification of cohesive soils.

A cohesive plastic soil may go through four consistency states as the moisture content of the soil is increased. These states are the solid state, the semisolid state, the plastic state, and the liquid state. The limits between these consistency states are the Shrinkage limit, Plastic limit, and the Liquid Limit (respectively). These limits are often referred to as the Atterberg limits. The Plasticity Index is defined as the numeric value of the Liquid limit minus the numeric value of the Plastic limit (see Plate AL).

Resistivity

The laboratory test for resistivity is performed in order to determine the relative quantity of soluble salts present in a specific soil. It is most often used as a method to determine the likelihood of corrosion potential for steel pipe, pile, or reinforced concrete structures. The resistivity test is also a means for determining the necessity of further chemical analysis of the soil or water for pH, sulfate and chloride-ion content.

A representative sample of the earth materials encountered at the site was delivered to M.J.Schiff & Associates, Inc. where it was tested for resistivity. The test method utilized is in conformity with the procedures outlined in California Test 532/643. Resistivity of soils is inversely proportional to corrosiveness. Thus, the analysis helps in determining whether the soils may have a deleterious affect on underground metallic structures. Test results are presented below in Table II. A generally accepted correlation between resistivity and soil corrosiveness toward metals is provided below:

Resistivity (Ohm-Centimeter)	Corrosiveness
< 1,000	Severely Corrosive
1,000 - 2,000	Corrosive
2,000 - 10,000	Increasingly Moderate
> 10,000	Increasingly Mild

Laboratory Test Results-Table II

Sample	Description	Status	Resistivity (ohm-centimeters)
B1@0-3'	Silty Sand	as-received	52,000
		saturated	5,000

Soluble Sulfates

A sample was taken from each lot and submitted to our laboratory for a soluble sulfate analysis. Please refer to Table III for a list of the results. When test results exceed 150 ppm, special considerations for concrete design are appropriate per UBC Table 19-A-3. This table contains specific requirements for concrete that is exposed to sulfate.

Laboratory Test Results-Table III

Sample	Description	Soluble Sulfates (ppm)
B1@0-3'	Silty Sand	ND (Not Detected)

pH

The pH of selected samples was tested. The results indicate the sample was slightly basic, with a pH of 7.3.

HYDROCONSOLIDATION POTENTIAL

Hydroconsolidation is a condition where dry or moist soils undergo settlement upon being wetted. In many cases no additional surcharge load is necessary to trigger the hydroconsolidation.

The potential for hydroconsolidation has been evaluated based upon the results of consolidation tests performed on samples taken from the excavated borings. The results from our testing suggest that the soils within the upper 5 to 7 feet of the site have a potential for hydroconsolidation, considering the results from our consolidation test on the sample from boring B3 at a depth of 5 feet. Other consolidation test results indicate potential hydroconsolidation on the order of 0 to 3 percent. The 3 percent consolidation was noted in a sample from the Saugus formation bedrock in B6 at a depth of 30 feet. This material was noted in the boring log as being friable. The alluvial samples obtained from depths below five feet and exhibiting hydroconsolidation during testing are also coarse-grained. Each of these alluvial samples required a high number of blows to drive the sampler.

Based on our data, we believe that much of the hydroconsolidation noted in our laboratory samples is related to disturbance of sandy samples. It is our opinion that the potential for hydroconsolidation is significant only in the upper 5 to 7 feet of the soil profile. If the recommended removals are accomplished (see the grading recommendation portion of this report), the materials to support the planned construction will have an insignificant potential for hydroconsolidation.

LIQUEFACTION POTENTIAL

Liquefaction is a condition where the soil undergoes continued deformation at a constant low residual stress due to the build-up of high porewater pressures. The possibility of liquefaction occurring at a given site is dependent upon the occurrence of a significant earthquake in the vicinity; sufficient groundwater to cause high pore pressures; and on the grain size, relative density, and confining pressures of the soil at the site.

As part of our analyses of the liquefaction potential on the site, we have performed three CPT soundings and seven borings to obtain subsurface data. Based upon our subsurface information and review of published data, the site is situated at the northern edge of Little Simi Valley where alluvial deposits range from 0 to >50 feet depth beneath the site. We have reproduced Plate 1.2 from CDMG Open-File Report 2000-007 (Moorpark Quadrangle) to illustrate the site's location at the edge of the alluviated valley. As discussed above in the Groundwater section, despite the fact that groundwater was not encountered in the subsurface exploration of the alluvial soil to a depth of 51.5 feet, we feel that available information supports an assumption of historic high groundwater to be 40 feet below the ground surface. This, coupled with the likelihood of significant ground shaking, was cause to perform further evaluation of the liquefaction potential at the site.

General Discussion

Liquefaction is a condition where the sedimentary soils, primarily recently deposited sands and silts, below the water table lose strength and behave as a viscous liquid rather than a solid. This is related to ground shaking when these soils undergo continued deformation at a constant low residual stress due to the build-up of high porewater pressures. The possibility of liquefaction occurring at a given site is dependent upon the occurrence of a significant earthquake in the vicinity; sufficient groundwater to cause high pore pressures; and on the grain size, relative density, and confining pressures of the soil at the site.

We have performed both borings and CPT soundings for use in evaluating the liquefaction potential at the site. Boring B1 has been excavated immediately adjacent to CPT1 to allow for confirmation of the CPT correlations used in our analysis. Based on the exploration information, the CPT data appears to correlate well with the boring information (comparative information is available in the Liquefaction Analysis Appendix with the CPT 1 data). Considering the positive correlation between the boring information and the CPT information, we have chosen to use primarily the CPT data in analysis of liquefaction potential because of its inherent repeatability and its superior ability to define the underlying earth material stratigraphy.

Based on our data, there are some coarse-grained materials below the assumed design groundwater elevation that have a potential to liquefy during a design-level earthquake.

In the liquefied condition, soil may deform with little shear resistance. The amount of soil deformation following liquefaction depends on the looseness of the material, the depth, thickness, and areal extent of the liquefied layers, the ground slope, and the distribution of loads applied by structures. When liquefaction is accompanied by ground displacement or ground failure, it can be destructive. Adverse effects of liquefaction can include ground oscillation, lateral spreads, flow failures, loss of bearing strength, settlement, and increased pressures on retaining walls.

Discussion of Liquefaction Hazard Assessment

As part of our analyses of the liquefaction potential on the site, we have performed several CPT soundings and borings to obtain subsurface data for use in analyses. Based upon our data, coarse-grained sedimentary soils are present on the site within the upper fifty feet of the soil profile; however it should be noted that these alluvial soils pinch out to zero thickness as bedrock crops out in the northern portion of the site. Groundwater was not encountered within the upper fifty feet during our exploration but is assumed to occur at forty feet below ground surface to reflect probable historic highs.

To address the possible impacts of liquefaction, the practice of geotechnical engineering currently has methods of approximating the potential liquefaction-induced settlement, lateral spreading, and the possibility of surface manifestations.

Liquefaction-Induced Settlement Potential

The potential for liquefaction-induced settlement has been evaluated using the procedures proposed by Tokimatsu and Seed (1987). Our analysis indicates the potential seismic settlement due to a design-level earthquake could be on the order of 3 ½ inches in the southern portion of the site, where the alluvial deposits are the thickest. The potential seismic settlement is actually due mostly to anticipated compression of the unsaturated alluvial soils, with only a minor contribution from the liquefiable soils. In northern portions of the site, where alluvial deposits are thinner, the potential seismic settlement is reduced. This methodology does not apply to fine-grained materials. Currently, the practice of geotechnical engineering does not have effective means to estimate seismic settlement of fine-grained materials. Recommended design settlement values are discussed subsequent to the Foundation Systems section of this report.

Lateral Spreading and Surface Manifestations

Do to the depth of the groundwater, lateral spreading and surface manifestations are not anticipated using the evaluation methods noted in the attached liquefaction appendix.

SLOPE STABILITY

Stability analyses of the planned and existing slopes were performed using a computerized limit-equilibrium method, the Spencer's Method. The computer program SLIDE v5.0 (Rocscience, 2004) was used. Spencer's Method of stability analysis was chosen because with its use of inter-slice forces, it solves for both force and moment equilibrium. A search of postulated failure surfaces was performed along a fine-grained layer in the bedrock along what we have considered the most critical geologic section. The results of these analyses are provided as a factor of safety. The factor of safety is considered the ratio of available shear strength to the shear strength required for just-stable equilibrium. The minimum computed factor of safety for the static permanent case is in excess of 1.5; however, for the pseudostatic case it is below 1.10.

Cross-Section B-B'

Cross-Section B-B' was drawn through the subject site and projected approximately 1,000 feet north of the northerly property line. Available geologic data projected into the cross-section from offsite areas results in an overall apparent dip of approximately 3 degrees to the southwest, based upon southwest true dips of 4 to 6 degrees. For purposes of analyses, we projected the weakest material encountered in the subsurface exploration (clay at 46 feet in B5) upslope along a 3 degree dip. Based upon attitudes in the lower section of B5, we also projected the bed upslope along a 5 degree apparent dip. Both projections were assumed to be truncated by the south-dipping fault encountered by GAI on Tract 5130. It should be noted that these upslope clay bed projections assume: 1) that the clay bed is laterally continuous across a distance of >500 feet; and, 2) that it is perfectly planar (i.e., not warped by local folding). Neither of these assumptions may be true but we have modeled the slope stability in this manner based upon the limited available geologic data that can be projected to the site.

For material strengths we have utilized our shear test results from our work on this project, along with our knowledge of strength results for these geologic units in the general area of the project. The use of residual strength (based on Stark & McCone correlation) for the aforementioned clay bed, as opposed to the fully-softened state, further assumes that the clay bed has been previously sheared (i.e., via flexural slip) when in fact downhole observation of the material yielded no such evidence. The following strengths were used in our analyses.

Material	Wet Unit Weight (pcf)	Cohesion (psf)	Internal Friction Angle (deg)
Eng. Fill	130	130	32
Alluvium	130	200	38
Saugus Form. Across-bedding	130	500	27
Saugus Form. Along-bedding	130	300	20
Saugus Form. Fine-Grained Bed	130	Non-linear*	Non-linear*

* Based on results of Stark & McCone correlation

Slope Stability – Static Analyses

Based on our static slope stability analysis of cross-section B-B', the slopes superjacent to the subject site appear to have a factor of safety in excess of 1.5. This stability analysis considers the planned cuts for structural improvements.

Slope Stability – Pseudostatic Analyses

We have performed pseudostatic slope stability analysis along cross-section B-B'. These analyses utilized a pseudostatic coefficient of friction of 0.2, in keeping with our understanding of the standards for the City of Moorpark. The results of our analysis using the Spencer's Method indicate a factor of safety of approximately 1.03. This does not meet the customary factor of safety of 1.10 for pseudostatic analysis.

When conditions are such that the customary pseudostatic factor of safety is not met, a secondary analysis is performed to estimate the potential deformation that could occur for the pseudostatic condition. For our deformation analysis, we have used the methods proposed in the "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California" ("Guidelines"). We have performed seismic slope stability analyses using the method proposed by Bray, et.al., (1998). This stability methodology was developed for analyses of geosynthetic-lined solid waste landfills. It has been adopted by the implementation committee as being applicable to conventional fill and natural slopes.

Bray Slope Stability Method

The Bray slope stability method utilizes the results of the PSHA, a shear wave velocity for the material, a maximum depth to failure plane, and a failure yield acceleration. The yield acceleration is that pseudo-static coefficient that produces a factor-of-safety of 1.0.

The shear wave velocity acceleration for the bedrock was obtained from reference materials. The analyses, for a CBC level earthquake (475 year return period), predicts displacements of 1 inch and 2 inches for the 3° and 5° apparent dip cases, respectively. Typically, estimated displacements of less than 5 cm are considered acceptable for residential structures. We do not consider this estimated displacement to have negative consequences for the project.

DISCUSSION AND RECOMMENDATIONS

Data from our field exploration, laboratory testing, reference reports, and engineering analyses, coupled with inferred conditions about our exploratory excavations, is the basis for the following discussion. Recommendations, based upon the presently available data, are presented for your consideration.

Removals

Based upon our findings, it is recommended that the upper 5 to 7 of alluvial soil be removed down to firm native materials in areas proposed to support fills or structural loads. In addition, design cuts into bedrock which may support foundations should be observed to confirm that the uppermost weathered zone, which typically affects the upper 3 feet of bedrock, has been removed.

Temporary Excavations

Temporary excavations (such as backcuts for retaining wall excavations) may be considered stable if cut vertical, providing they are restricted to a maximum of 4 feet in height, are provided with permanent support as soon as possible, and they are protected from erosion and saturation. Portions of temporary excavations in excess of 4 feet high should be laid down to 1 1/2:1 for excavations exposing alluvial soils, or 1:1 for excavations exposing bedrock, unless specific alternative treatments are evaluated and found acceptable.

It should be noted several offsite structures occur near the property line of the subject site. Alternative construction methods, such as slot cutting, or shoring, may be required to ensure temporary stability of offsite properties during construction of retaining walls proposed along property lines. This should be evaluated further during the plan review stage of the project, unless the Client desires an earlier assessment for purposes of evaluating economic feasibility.

Engineered Fill—Compaction Standard

The on-site materials are suitable for use as engineered fill. All roots, organic matter, and other deleterious material should be hand-picked from the soils prior to their use as engineered fill. The majority of soils at the site are coarse-grained, having more than 15% fines passing a 0.005 mm particle size sieve. These materials should be moistened and/or air-dried to near optimum moisture content and compacted to at least 90% of their maximum density as determined using the Modified Proctor Test (ASTM D 1557-00).

Grading—Engineered Fills

The following recommendations pertain to the placement of, and preparation for, engineered fills;

1. The on-site soils are suitable for use as structural fill. Any import materials that are to be used as structural fill should be approved by this office prior to placement.

2. Shrinkage refers to the lesser volume of fill that results from a given volume of excavation. The shrinkage of the alluvial materials is anticipated to be between 12% and 17%. The Saugus Formation bedrock is anticipated to shrink on the order of 7% to 12% considering the planned cuts.

3. All vegetation, trash debris or other deleterious material should be stripped from the area to be graded. Soils bearing sparse grasses may be thoroughly mixed with at least ten parts clean soil and incorporated into the engineered fill. Other materials should be wasted from the site.

4. Compressible soils that lie within the areas to receive engineered fill should be removed to relatively incompressible material, moisture conditioned, and replaced as properly compacted fill. Portions of the compressible materials that are sufficiently thin may be scarified, watered or air dried to approximately the material's optimum moisture content, and compacted in-place. A combination of removal and recompaction in-place may be used, providing the recommended compaction is obtained throughout the recommended depth interval. Based upon the materials exposed in our exploratory excavations, we anticipate the removals to extend to depths of 5 to 7 feet. Removal bottoms must be field verified by a representative of the geotechnical consultant.

5. Exposed surfaces should be scarified, moistened or air dried as appropriate, and compacted to the appropriate percentage of the material's maximum dry density prior to placement of fill (see COMPACTION STANDARD section).

6. We recommend a uniform blanket of compacted fill be created for support of structural footings. The fill cap should extend to at least three feet below the base of proposed footings and five feet beyond their perimeter. Special consideration should be paid to locations where property lines or existing improvements (buildings, retaining walls, fences, power poles, etc.) interfere with the creation of the desired fill cap. Such conditions should be brought to the attention of this office so that the specific site conditions may be evaluated and recommendations provided. Depending upon the circumstances, special excavating techniques may be employed (i.e. slot cutting), alternative foundation designs may be used (i.e. grade beams supported by pad footings or piles), or the compaction standard may be increased.

7. Transition lots (building pads partially cut and partially fill), building pads underlain by non-uniform earth materials (e.g. differing expansive properties) and shallow cut lots (where the depth of cut is less than the thickness of compressible soils) should be provided with a uniform blanket of compacted fill for support of structural footings. The fill cap should extend to at least three feet below the base of proposed footings and five feet beyond their perimeter.

8. Where the ground slopes steeper than 5:1 (H:V), the engineered fill should be properly benched into competent material. Typical benching is illustrated in Appendix E.

9. Fill slopes that toe onto sloping ground should be founded below the compressible surface soils in [MATERIAL]. The key should be at least 20 feet wide and 3 feet deep (measured on the downslope side). The bottom of the key should be graded so that there is at least one foot of fall across its width (toward the upslope side). The key should be located in front of the toe of slope (as shown on the plan) so that the outside limit of the key lies at or beyond a 1:1 projection from the planned toe of the slope. Typical fill key construction is illustrated in Appendix E.

10. Areas that are to be paved should be scarified to at least 12 inches below the existing or rough grade (whichever is deeper), brought to near the material's optimum moisture content, and compacted to the appropriate relative compaction (see COMPACTION STANDARD section).

11. Fill materials should be placed in thin lifts, watered to near the material's optimum moisture content, and compacted to the appropriate relative compaction prior to placing the next lift.

12. Fill slopes constructed of clean sand are commonly subject to excessive erosion or shallow slope failures. Similarly, fill slopes constructed with clayey soils may be subject to desiccation, cracking, creep or other surficial deterioration. Utilizing mixed soils (sand with

some proportion of fines, i.e. clayey sand) in the outer 20 feet of the fill slope may serve to minimize the potential for surficial slope deterioration.

13. The compaction standard applies to the face of fill slopes. This may be achieved by overfilling the constructed slope and trimming to a compacted finished surface, rolling the slope face with a sheepsfoot, or any method that achieves the desired product.

14. All grading should comply with the grading specifications and requirements of the local governing agency.

Grading—Temporary Excavations

Temporary excavations (such as backcuts for stability fills, removals, and retaining wall excavations) may be considered stable if cut vertical, providing they are restricted to a maximum of 4 feet in height, are provided with permanent support as soon as possible, and they are protected from erosion and saturation. Portions of temporary excavations in excess of 4 feet high should be laid back to 1 1/2:1 unless specific alternative treatments are evaluated and found acceptable.

Utility Trench Backfill

Backfill for utility trench excavations should be compacted the appropriate relative compaction (see COMPACTION STANDARD section). Where installed in sloping areas, the backfill should be properly keyed and benched.

Foundation Systems

For planning purposes, this section provides preliminary foundation recommendations for conventional foundations. Once specific building types and foundation loads and locations are known, project specific foundation recommendations can be prepared.

Considering the planned excavations near property lines, there is a probability that portions of the buildings will have walls designed to use soldier piles. Recommendations for pile foundations are provided in the retaining wall section of this report.

Conventional Foundations

Continuous or pad footings may be used to support the proposed structures. In order to achieve the capacities specified below, they should be founded a minimum of 12 inches into engineered fill, with the concrete placed against in-place, undisturbed material. Foundation design criteria are based, in part, upon the expansive properties of the materials anticipated to be present near the finished pad grade. Laboratory testing to verify the expansive properties of the near-pad-grade materials should be performed at the completion of rough grading.

Pre-saturation guidelines are presented in the following table. Pre-saturation of the foundation soils should be initiated well before concrete is scheduled to be placed. Care should be taken to see that the water has properly penetrated the soil. Last minute flooding is not a good practice. Excess water remaining in the target pre-saturation zone at the time of concrete placement will penetrate further into the soil, possibly causing additional expansion and uplift of the curing concrete.

Anticipated Expansion Index Range	**0 - 20**
Pre-moisten	12"
Footings[1]	
Allowable Bearing Capacity	1800 PSF[2]
Lateral Resistance	400 PSF/Ft[2] [3]
Maximum Lateral Resistance	2500 PSF[2] [3]
Coefficient of Friction	0.4
Minimum Embedment Into Foundation Material	12 inches
Minimum Embedment Below Adjacent Grade[4]	24 inches
Minimum Reinforcement	2 #4 bars, 1 near top, 1 near bottom
Slabs-On-Grade	
Bedding	2" of clean sand[5]
Thickness	Full 4"
Minimum Reinforcement	#4 bars @ 16" o.c., e.w.

Anticipated Expansion Index Range	**21 - 90**
Pre-saturation	18" (EI 21-50) 21" (EI 51-90)
Footings(1)	
Allowable Bearing Capacity	1800 PSF(2)
Lateral Resistance	250 PSF/Ft(2) (3)
Maximum Lateral Resistance	1800 PSF(2) (3)
Coefficient of Friction	0.3
Minimum Embedment Into Foundation Material	12 inches
Minimum Embedment Below Adjacent Grade(4)	24 inches
Minimum Reinforcement	2 #4 bars, 1 near top, 1 near bottom
Slabs-On-Grade	
Bedding	4" of clean sand(5)
Thickness	Full 4"
Minimum Reinforcement(6)	#4 bars @ 16" o.c., e.w.

(1) Bearing portions of all footings should be at least five feet (measured horizontally) from the face of adjacent, descending slopes. All footings should bear at least three feet below an imaginary plane projected upward at 1.5:1 from the toe of locally over-steepened slopes. Pad footings should be at least 24 inches square.

(2) May be increased by 1/3 for short duration loading such as by wind or seismic forces.

(3) Decrease by 1/3 when combined with friction.

(4) Applies to exterior footings. Depth must meet the CBC requirements for the specific level of stories supported.

(5) Place vapor barrier (10 mil. visqueen) one inch below top of sand layer beneath all areas where moisture penetration of the slab is undesirable.

(6) Dowel slab to exterior footing using #3 bars @ 32" on center, bent 3' into slab for EI=51-90.

For design of mat foundations or slabs-on-grade, a modulus-of-subgrade reaction of 125 PSI/IN may be used. This value is a unit value for use with a 1-foot-square plate. The modulus should be reduced in accordance with the following equation when used with a larger area:

$$K_S = K_1\left[\frac{B+1}{2B}\right]^2$$

Where: K_S =Reduced subgrade modulus
K_1 =Unit subgrade modulus
B =Foundation width in feet

Settlement

For planning purposes, structural foundations designs should consider total static settlement from foundation loads to be on the order of 1 inch with differential settlement on the order of ½ inch over a distance of 30 feet.

The site is defined as having a potential for seismically induced settlement. Our analysis indicates a potential for seismic settlement as great as approximately 3 ½ inches during a design-level earthquake. The differential seismic settlement can be assumed to be equal half of the total seismic settlement. Considering the estimated potential total seismic settlement of 3 ½ inches, the differential seismic settlement can be considered 1 ¾ inches over an assigned horizontal distance of 30 feet.

Retaining Wall Recommendations

Retaining walls are planned throughout the property, with some to be constructed near the perimeter property lines. It is anticipated that the walls away from the property lines will use conventional foundations, while those walls along the perimeter of the property will be designed as soldier pile walls. Foundation design criteria for conventional foundations are provided in the preceding Foundation section. Pile design criteria is provided in subsequent portions of this report. Lateral loading criteria for cantilevered wall designs are presented in the table below.

Slope of Backfill	Equivalent Fluid Density Active Condition (pcf)	Equivalent Fluid Density At-Rest Condition (pcf)
Level	43	64
3:1	56	95
2:1	70	---

All retaining walls should be provided with adequate backdrainage systems. Either weep holes or pipe outlets should be installed. Free draining material should be used behind weep holes or about pipe drains. Care should be exercised to see that weep holes are installed and maintained above the finish grade adjacent to the face of the wall.

Backfill for retaining walls should be properly compacted. An impervious cap should be provided at the top of the backfill to retard infiltration of water.

Additional surcharge, such as that due to proposed structures, traffic, or other loading, should be included in the wall design. Use of expansive soil as backfill for retaining walls will result in a surcharge to the wall, the magnitude of which is dependent upon the expansion index of the backfill. This may be avoided by using sand or gravel as backfill adjacent to the wall. Details regarding this type of construction may be provided upon request.

In areas where sloping of the sidewalls of temporary excavations is not possible, such as where retaining walls are planned along property lines, soldier pile walls may be used as an alternative to cantilever retaining walls with conventional shallow spread footings. The following geotechnical recommendations are provided for of cantilever soldier piles with lagging. It is anticipated that for the property line walls, the temporary lagging will be covered over with a reinforced concrete wall between soldier piles. These recommendations are general in nature, additional recommendations may be warranted once construction methods and specific data regarding the shoring design are available.

For soldier pile retaining walls the aforementioned active-earth lateral pressure may be used for the retained soil. Additional loading from any adjacent foundations should be incorporated into the design of the retaining wall. The lateral surcharge load from foundations should be continued to a depth where the pressure exerted by the surcharge is 100 psf or less. At this point the foundation surcharge may be discontinued provided it is below the bottom of the excavation. Nearby traffic loads within a 1:1 projection from the base of the excavation should also be incorporated into the design loading. The lateral load from traffic loads should be continued to a depth of 10 feet or to the bottom depth of the excavation, whichever is less.

The cantilever soldier piles are anticipated to resist lateral movement or overturning through transmission of these lateral forces to the soils below the excavation elevation. The passive resistance provided by the soils below the base of the excavation can be assumed to be an allowable pressure of 600 psf/ft to a maximum of 6000 psf (considering a factor of safety of 1.5) for piles spaced at least three pile diameters apart. This passive resistance is applicable for undisturbed soil in direct contact with the soldier pile. The depth of the pile penetration below the base of the excavation must be sufficient to resist the lateral movement and over-turning of the soldier pile system. We recommend that passive resistance be ignored for a depth equal to 1.5 times the effective pile diameter below the base of the excavation. The effective pile diameter is considered the dimension of the soldier pile taken parallel to the line of the wall for driven piles, or the diameter of the drilled hole, whichever is greater.

Drilled holes may be backfilled with structural concrete below the excavation line. The remainder of the hole may be backfilled to the ground surface with sand-cement slurry or lean concrete that is strong enough to prevent collapse of the hole, but weak enough to be excavated for installation of lagging.

Wood or steel lagging should be used to support the excavation wall between the soldier piles. If the lagging is to remain in place permanently, then treated lumber should be used for the wood lagging. Much of the lateral force is anticipated to be distributed to the cantilever soldier piles through soil arching. Therefore, the lagging may be designed to resist 60% of the theoretical lateral load on a simple span, but need not exceed a value of 400 psf (without surcharges). For the arching effect to occur, the backside of the soldier pile must bear against the soil. Placement of lagging behind the back flange of the soldier pile is not recommended.

Cast-in-Drilled-Hole Pile Foundations

Based on the site conditions and our understanding of the project, the proposed structures may be supported on cast-in-drilled-hole (CIDH) friction piles founded in competent native materials. The alluvium and bedrock on the site can be excavated by a drill rig. This material is sandy and caving of the excavations may occur.

CIDH piles may be designed with a minimum diameter of 24 inches. The recommended allowable vertical capacity is presented on Plate C1. Concrete must be placed in direct contact with the undisturbed in-place materials in order to achieve the specified allowable capacities. Piles may be assumed to derive vertical support via skin friction in the native materials or fill beginning at a depth of approximately 1.5 pile diameters below grade.

Pullout resistance may be taken as one-half the allowable capacity. Capacities may be increased by one-third for short duration loading (i.e., by wind and seismic loading). Settlement of piles is anticipated to be less than one half inch. Lateral deflection of 24 inch diameter CIDH piles is anticipated to be less than one quarter of an inch.

Factors of Safety

The factor of safety for the allowable bearing pressure provided is greater than three. The allowable passive pressure provided is based upon a factor of safety of 1.5. The factor of safety for the sliding friction is one. The factor of safety for the active pressure is one.

With regard to retaining walls, the Uniform Building Code calls for a 1.5 factor of safety for both sliding and overturning. We defer to the Uniform Building Code and the project structural engineer on this matter.

Corrosion Potential

Preliminary testing of a sample obtained from our borings indicates the on-site soils have a negligible level of sulfates-indicates a low corrosion potential for concrete. Resistivity tests indicate the soils are mildly corrosive to ferrous metals. Near the completion of grading additional testing should be performed to verify the corrosion potential of the soils.

TABLE 19-A-4 REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

SULFATE EXPOSURE	WATER-SOLUBLE SULFATE (SO_4) IN SOIL, percentage by weight	SULFATE (SO_4) IN WATER, ppm	CEMENT TYPE	Maximum Water-Cementitious Materials Ratio, by Weight, Normal-Weight Aggregate Concrete[1]	Minimum f'c Normal Weight and Lightweight Aggregate Concrete, psi[1] x 0.00689 for MPa
Negligible	0.00 - 0.10	0 – 150	--	--	--
Moderate[2]	0.10 - 0.20	150 - 1,500	II, IP(MS), IS(MS)	0.50	4,000
Severe	0.20 - 2.00	1,500 - 10,000	V	0.45	4,500
Very severe	Over 2.00	Over 10,000	V plus pozzolan[3]	0.45	4,500

1 A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 19-A-2).

2 Seawater

3 Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Preliminary Pavement Structural Sections

Preliminary plans indicate improvements will include constructing parking lots, access drives, and perhaps improvements to existing exterior streets. The parking stalls should be designed using 3 inches of asphaltic concrete on 7.5 inches of base. The driveways should be designed using 3 inches of asphaltic concrete on 10.5 inches of base. At this time, the location and planned traffic index of exterior streets is not known. Street recommendations can be provided once supplemental street improvement design information is known.

The upper 12 inches of the subgrade soil should be compacted to at least 95% relative compaction. Base materials should be compacted to at least 95% relative compaction.

R-value tests should be performed at the completion of grading and final pavement section designs developed at that time.

Drainage

Positive drainage should be established to carry pad waters away from structures and foundations, and to prevent uncontrolled or sheet flow over manufactured slopes. We recommend as steep a gradient as practical be established around the structures, to the street or other non-erosive drainage devices. Fine-grade fills placed to create pad drainage should be compacted in order to retard infiltration of surface water.

Preserving proper surface drainage is also important. Planters, decorative walls, plants, trees or accumulations of organic matter should not be allowed to retard surface drainage. Area drains and roof gutters (if present) should be kept free of obstruction. Roof gutters (if present) and/or condensation lines from air conditioners should outlet to a non-erodible device, i.e., walkways, patios, driveways, drain lines or splash blocks that direct the water away from the structure. Swales and/or area drains should outlet to the street or acceptable non-erodible device. Positive drainage along the backs of retaining walls should be maintained. Any other measures that will facilitate positive surface drainage should be employed.

Construction Monitoring

Finalized grading plans and foundation plans should be submitted to this office. The project Civil Engineer should incorporate the removal recommendations into the grading plans. Additional recommendations may be provided at that time of our review, if such are considered warranted.

Placement of all fill and backfill should be monitored by representatives of this office. This includes our observation of prepared bottoms prior to filling. All excavated slopes, both temporary and permanent, should be observed by a representative of this office. Supplemental recommendations may prove warranted based upon the materials exposed in the actual excavations.

Foundation excavations should be observed by representatives of this office to see if the recommended penetration of proper supporting strata has been achieved. Such observations should be made prior to placing concrete, steel or forms. This office should be notified at least 24 hours prior to placing concrete.

CLOSURE

This geotechnical report has been prepared in accordance with generally accepted engineering practices at this time and location. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

Thank you for this opportunity to be of service. Please do not hesitate to call if you have any questions regarding this report.

Respectfully submitted,
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REFERENCE LIST:

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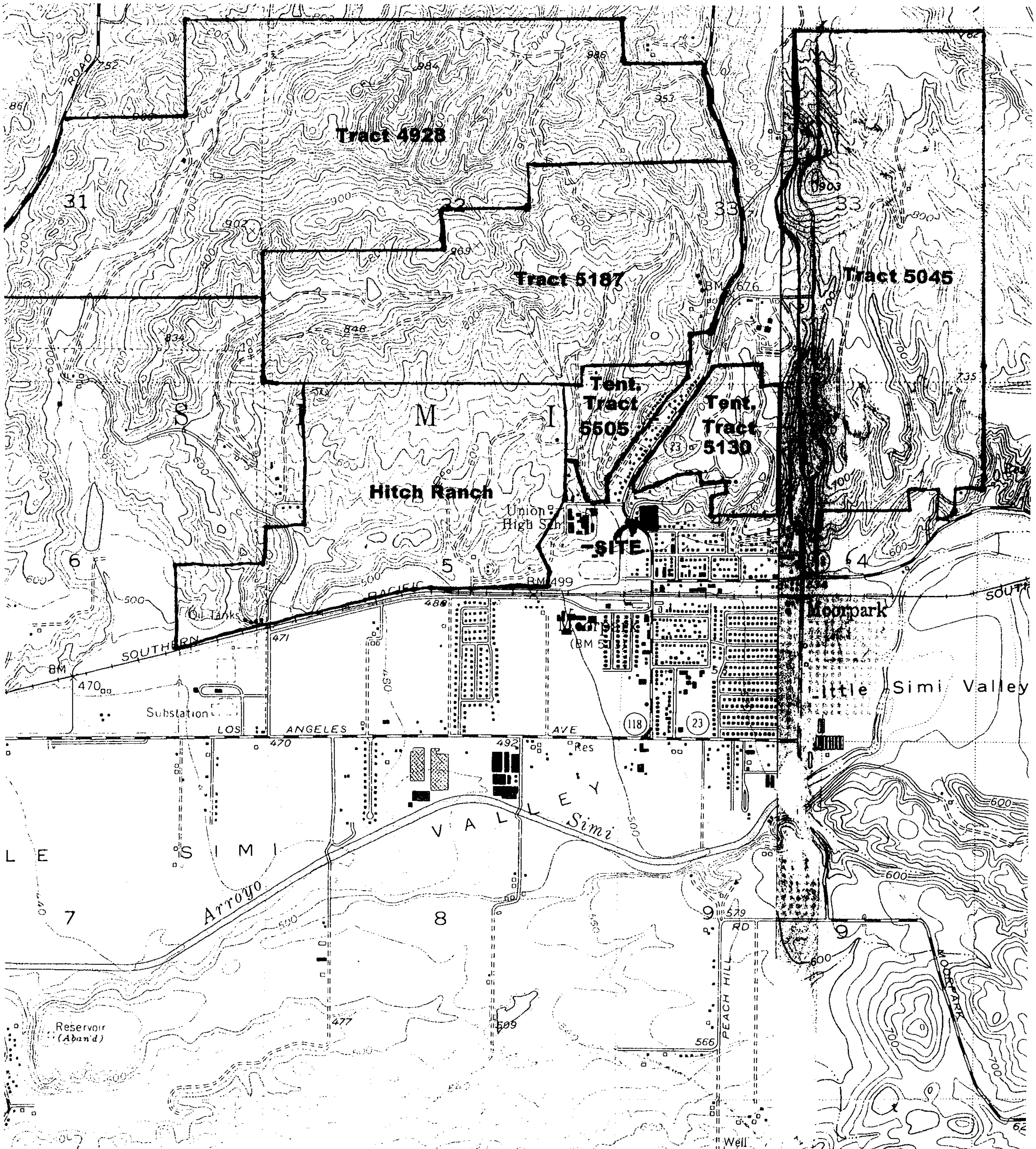
LOCATION MAP - EVERETT TERRACE

City of Moorpark, California



Geolabs – Westlake Village

GEOLOGY AND SOIL ENGINEERING

DATE ______ BY SD

SCALE 1"=2000' W.O. 8953

Base map: Moorpark 7.5-minute quadrangle, 1969.

PLATE 1.1

SUBSURFACE DATA

LOG OF BORING B1

CLIENT: John Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 529'	DATE: 7/2/04
RIG TYPE: 8" HSA	HAMMER WEIGHTS: 140 lbs.	DROP: 30"

	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0			X				
			X				
	8/12/14	C	X	4.4	103.7	@2.5' - Alluvium: Dark brown silty SAND with infrequent coarse grains, dry, loose, porous, roots and rootlets.	
5	5/10/12	C		6.6	106.5	@5' - Dark brown silty SAND with occasional coarse grains, damp, loose, porous (frequent 1/16" diameter pores occasionally larger).	
	11/16/21	C		6.2	119.2	@6.5' - Yellowish brown slightly silty SAND, medium grained, with occasional subrounded coarse grains to fine grained sequences, damp, medium dense, friable.	
			X				
			X				
10	13/25/25	C		8.2	113.1	@10' - Yellowish brown slightly silty fine grained SAND, damp, dense.	
	11/30/50	C	X	18.2	112.4	@11' - Dark yellowish brown silty CLAY with sand, moist, very stiff, frequent white calcium carbonate filaments (veinlets).	
			X				
15	12/25/32	C		17.8	108.5	@15' - Dark yellowish brown CLAY, moist, very stiff, frequent soft calcium carbonate rich pockets; LL=56.3, PL=17.8, PI=39; 12% sand, 58% silt, 30% clay (0.002).	
	7/8/10		S			@16' - Yellow brown fine to coarse grained SAND with occasional gravels, damp to dry, medium dense, friable.	
20	10/11/10		S			@20' - Yellowish brown silty SAND with occasional subrounded to subangular gravels, damp, medium dense.	
	9/10/12		S			@22.5' - Yellowish brown silty clayey SAND to sandy CLAY with occasional coarse grains, moist, medium dense to very stiff; 1% gravel, 52% sand, 33% silt, 14% clay (0.002).	
25	11/12/17		S			@25' - Pale yellowish brown SILT with clay stringers, grading to medium to coarse grained SAND with gravels, finer sequences moist, coarser sequences dry, very stiff to dense, non-plastic; 2% gravel, 60% sand, 28% silt, 10% clay (0.002).	
	10/14/15		S			@27.5' - Yellow brown SILT with small white calcium carbonate veinlets, damp, very stiff, grading to fine grained SAND, friable, damp, dense to medium dense.	
30	9/12/12		S				
	12/12/15		S			@30' - Yellowish brown silty fine grained SAND with infrequent coarse grains, damp, dense to medium dense, grading to medium brown sandy CLAY with frequent medium to coarse grained SAND, moist, very stiff.	
35	12/11/13		S			@31' - Dark yellowish brown slightly clayey silty SAND with frequent coarse grains, damp, dense to medium dense.	
	12/17/25		S			@35' - Medium brown clayey silty SAND, moist, dense/stiff; LL=26.5, PL=14.1, PI=13; 54% sand, 34% silt, 12% clay (0.002).	
40	12/12/14		S			@37.5' - Pale yellow brown slightly silty medium to coarse grained SAND with frequent small gravels, dry, dense.	
						@40' - Medium brown clayey silty SAND with frequent coare grains, moist, dense to medium dense.	
45							

ADDITIONAL COMMENTS:

C = California Split Barrel Sampler
S = Standard Penetration Test (SPT)
Blows per 6"

SURFACE DATA

LOG OF BORING B1

CLIENT: John Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 529'	DATE: 7/2/0
RIG TYPE: 8" HSA	HAMMER WEIGHTS: 140 lbs.	DROP: 30"

	N	U	B	M	DD	DESCRIPTION	ATTITUDES
40							
	10/12/15		S			@42.5' - Medium brown slightly clayey silty SAND with frequent coarse sand grains, damp to moist, dense to medium dense.	
45	8/12/15		S			@45' - Medium brown slightly clayey silty SAND with frequent coarse sand grains, damp to moist, medium dense, to dense.	
	6/7/8		S			@47.5' - Medium brown slightly clayey silty SAND with frequent medium to coarse sand grains, moist, medium dense, non-plastic; 1% gravel, 63% sand, 27% silt, 9% clay (0.002).	
50	13/17/22		S			@50' - Dark brown silty sandy CLAY to silty clayey SAND (paleosol?) with frequent medium to coarse sand grains, moist, very stiff to dense.	
55						Total Depth - 51.5' No groundwater No caving Backfilled	
60							
65							
70							
75							
80							
85							

ADDITIONAL COMMENTS: C = California Split Barrel Sampler
S = Standard Penetration Test (SPT)
Blows per 6"

Logged by: CW

CLIENT: John Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 543'	DATE: 7/2/04
RIG TYPE: 8" HSA	HAMMER WEIGHTS: 140 lbs.	DROP: 30"

	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Alluvium: Medium brown slightly clayey silty SAND with frequent coarse grains, dry to moist, loose to medium dense.	
			X X				
5						Gravels at 5'.	
	14/23/ 50-5"	C		3.0	112.4	@5' - Yellowish brown fine, well sorted fine grained SAND with infrequent coarse grains, dry to damp, dense. Occasional silt stringers.	
			X X				
10	12/27/30		S			@10' - Yellowish brown SAND with graded sequences consisting of fine to coarse sands, dry, dense to very dense.	
						Subrounded gravels at 14'.	
15	100-6" 50-6"	C	S	2.0	---	@15' - Yellowish brown slightly clayey silty SAND and gravels, damp, dense to very dense. @16' - Yellowish gray brown GRAVEL with coarse sand, subrounded to rounded, dry to damp, tightly packed, very dense, occasional calcium carbonate coating of gravels.	
20							
25						Total Depth - 16.5' (refusal on gravels). No groundwater No caving Backfilled.	
30							
35							
40							
45							

ADDITIONAL COMMENTS:

C = California Split Barrel Sampler
S = Standard Penetration Test (SPT)
Blows per 6"

CLIENT: John Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 535'	DATE: 7/2/04
RIG TYPE: 8" HSA	HAMMER WEIGHTS: 140 lbs.	DROP: 30"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Alluvium: Medium to dark brown silty SAND with frequent coarse grains, dry, porous, loose, abundant roots and rootlets.	
			X				
			X				
5	6/10/12	C		3.3	103.7		
10	13/14/15		S			@7' - Yellowish brown slightly clayey silty SAND, frequent medium grains, occasional coarse grains, damp, medium dense to dense.	
15	5/30/30	C		2.7	107.0	@15' - Yellowish brown fine to coarse grained SAND, frequent graded sequences (fine to coarse), damp to dry, dense to very dense, occasional white calcium carbonate coatings on coarse grains.	
			X			@17' - Yellowish brown clayey SILT to silty CLAY and interfingers of SAND, occasional coarse grains, moist, dense/very stiff, finer grained sequences speckled with small white calcium carbonate veinlets.	
			X				
			X				
20	25-5"		S				
25	100-6"	C		8.0	114.0	@25' - Medium brown clayey SAND with frequent coarse grains, damp, very dense, occasional pores. @26.5' - Very tight drilling, near refusal. Saugus Formation:	
30	16/18/24		S			@30' - Pale yellow fine grained SANDSTONE, dry, dense, well sorted.	
35	100-11"	C		23.6	98.4	@34' - Dark yellow brown to medium brown CLAYSTONE, massive, frequent black manganese staining, damp, very stiff, grades to light brown SILTSTONE, damp, very stiff.	
40	40/50-5"		S			@40' - Pale yellow fine grained SANDSTONE, well sorted, friable, dry, very dense. Total Depth - 40' No groundwater No caving Backfilled	
45							

ADDITIONAL COMMENTS:

C = California Split Barrel Sampler
S = Standard Penetration Test (SPT)
Blows per 6"

CLIENT: John Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 540'	DATE: 7/2/04
RIG TYPE: 8" HSA	HAMMER WEIGHTS: 140 lbs.	DROP: 30"

	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Alluvium:	
5	6/10/18		S			@5' - Medium brown silty SAND with frequent coarse grains, damp, medium dense.	
10	100-10"	C		6.0	122.5	@10' - Medium brown silty SAND with frequent coarse grains, damp, dense.	
15	7/18/20		S			@15' - Medium brown slightly clayey silty SAND with occasional coarse grains, damp, dense.	
20	12/15/25		S			@20' - Pale yellow fine to coarse grained SAND and subrounded subangular gravels, dry, dense, friable.	
			X				
			X				
25	8/16/50	C		10.0	115.3	@25' - Medium olive brown clayey SILT stringers over pale yellow very fine grained SAND, finer materials, moist, stiff, coarse grained materials dry, dense.	
30	12/20/25		S			@30' - Pale yellow silty SAND, damp, dense, with infrequent thin medium brown clayey stringers.	
35						Total Depth - 30' No groundwater No caving Backfilled	
40							
45							

ADDITIONAL COMMENTS:

C = California Split Barrel Sampler
S = Standard Penetration Test (SPT)
Blows per 6"

CLIENT: John Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 542'	DATE: 11/9/04
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 18"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Alluvium: Dark brown silty SAND with frequent coarse subangular grains and occasional subangular to subrounded gravels, moist, moderately loose, porous.	
5							
						@6' - Dry, increasing coarse grains, increasing clay content, decreasing porosity, stiff to dense.	
10							
						@12' - Clayey SAND with frequent coarse subangular to subrounded grains, moist, dense.	
						@14.5' - Medium brown sandy CLAY, moist, frequent coarse grains, stiff.	
15						@15' - Medium brown silty SAND with frequent subangular coarse grains, moist to damp.	
						@19' - Coarsening at lower contact to friable SAND with subangular gravels, dry.	
20						@21' - Irregular scoured contact of dark brown silty SAND with occasional friable SAND lenses with abundant coarse grains, dry.	
						@23-38' - Casing.	
						@24' - Yellowish brown fine to coarse grained SAND with frequent small gravels, moderately friable, massive.	
25						@26' - 4" SILT lens, discontinuous.	
						@27' - SILT with frequent subrounded to rounded coarse grains and small gravels.	@28' Approx.
						@28' - Caved - light gray fine to coarse grained well graded SAND and small gravel, dry, friable.	N35W/20NE
30						@30-38' - Not logged due to caving and casing. Saugus Formation contact estimated at ±35'.	
35							
							@38.5' - B
						@38' - Saugus Formation: Olive tan SILTSTONE, wavy, weakly bedded, interbedded with fine grained SANDSTONE, moist, stiff to dense.	N66W/25NE N52W/14NE
40							@39.5' B
	3/6/8			25.0	93.5	Becoming slightly clayey SILTSTONE, massive, moist, stiff.	N65E/25NW
	NR			18.5	109.9		@40' B N36W/25NE
45							@40.5' B N42W/16NE

ADDITIONAL COMMENTS: Kelly Bar Weights: 0 - 25', 2800 lbs.
25 - 47', 1600 lbs.
47'+, 1000 lbs.

NR = Not Recorded

CLIENT: John Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 542'	DATE: 11/9/04
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 18"

Depth	N	U	B	M	DD
40					
45					
50	7/15			4.4	120.7
55					
60					
65					
70					
72					
80					
85					

DESCRIPTION

@43.5' - Sharply truncated (faulted?) by sandy SILTSTONE bed.
@44' - Medium brown SANDSTONE with abundant coarse sand, occasional subrounded to rounded gravels, moist.
@46' - 4" clay bed, smooth bedding plane, no striations, no evidence of movement, underlain by olive brown SILTSTONE and light olive gray fine grained SANDSTONE interbeds.
@48' - Fine to medium grained SANDSTONE with fine grained SILTSTONE interbedded lenses.
@49' - Olive brown SILTSTONE, massive, damp to moist.
@51' - Light gray fine grained massive SANDSTONE with 1" gray, massive, stiff, clay lens at base.
@52' - Yellow brown fine to medium grained SANDSTONE with frequent subrounded to rounded coarse grained lenses and gravels, moderately friable, damp, poorly bedded, moderate well graded, coarsening downward sequences to include rounded cobbles, cross-bedded by coarser grained sequences.
@57' - Dark brown silty SANDSTONE with abundant coarse grains and subrounded to subangular gravels, damp.
@61' - Olive brown slightly silty fine to medium grained SANDSTONE with abundant coarse grains and subrounded gravels.
@62' - Cobbles up to 10".

Total Depth - 70'
No groundwater
Caving from 28-38'
Casing placed from 23-38'
Backfilled

ATTITUDES

@42.5' B
N55W/13NE
@43' - B
N38W/20NE
@43.5' Fault
N27W/37SW
@44' B
N33E/34SE
@46' B
N25W/14SW
@48' - B
N45W/15SW
@52' Approx.
BN45W/6SW
@53' B
N80W/5SW

ADDITIONAL COMMENTS: Kelly Bar Weights: 0 - 25', 2800 lbs.
25 - 47', 1600 lbs.
47'+, 1000 lbs.

CLIENT: Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 549'±	DATE: 2/7/05
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 12"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Fill: Dark brown clayey SAND with gravel, dense, moist, sparse roots to 8'. @1.5' - Alluvium: Tan brown fine to coarse grained SAND, dense, damp. Tan brown fine to coarse grained SAND with occasional subrounded to round gravel, very friable, dense, damp.	
5	3			2.2	104.7	Belling below 4'. Cross bedded friable SAND.	@7' B N42E/19NW (x-bedding)
10	9			1.8	112.9	Subrounded to round gravel to cobble in friable yellow brown fine to coarse grained SAND, well graded, dense, damp. @9' - Interbedded 12" thick subrounded to round gravel to cobble in fine to coarse grained SAND, well graded, and fine to coarse grained SAND, friable, dense, damp.	@12' B N38E/7NW (lamination)
15	9			2.6	116.8	@12' - Laminated light olive gray fine to coarse grained SAND, friable, dense, damp. @12.5' - Subrounded to round gravel to coarse cobble in fine to coarse grained SAND, dense, damp, well graded, occasionally poorly graded gravel, heavily scoured.	
20	8			3.7	114.1	Saugus Formation: Scoured contact with discontinuous pale yellow brown SILTSTONE and light olive gray cross bedded friable fine to coarse grained SANDSTONE. Massive silty fine grained SANDSTONE, dense, moist.	@19' Contact N25E/27NW; @22' B N25E/8NW
25	17			14.5	108.1	Interbedded 3-6" pale yellow brown clayey SILTSTONE, hard, moist, and silty SANDSTONE, dense, damp.	@26' B N22E/8NW
30	9			6.0	102.8	@30.5' - Slightly scoured contact between friable light olive brown fine to medium grained SANDSTONE and pale yellow brown friable fine grained SANDSTONE with silt.	@30.5' Contact N-S/39W
35	7			2.4	98.2	Thinly bedded brown clayey SILTSTONE within pale yellow brown silty SANDSTONE.	@34' B N16W/16SW
40	13			2.7	102.4	Thinly bedded (1-3" thick) yellow brown silty fine grained SANDSTONE within yellow brown to light live gray friable fine to coarse grained SANDSTONE, cross bedded, dense, damp, caving sands below 39'.	@39' B N21E/3NW
45							

ADDITIONAL COMMENTS: Blows per 12"
Kelly Bar Weights: 0 - 24', 3800 lbs.
24 - 47', 2800 lbs.
47 - 74', 1800 lbs.

SURFACE DATA

LOG OF BORING B6

CLIENT: Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 549'±	DATE: 2/7/05
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 12"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
40							
45	18			3.3	104.1	Yellow brown cross bedded fine to coarse grained SANDSTONE.	@44' B N40W/19SW (x-bedding)
50	61			13.2	106.3	@48.5' - Yellow brown friable gravelly coarse grained SANDSTONE to brown clayey SILTSTONE, massive, hard, moist. 12" thick brown clayey SILTSTONE itnerbedded within friable fine to coarse grained SANDSTONE, dense, damp. @51' - Tan brown friable fine to coarse grained SANDSTONE, dense, damp.	@48.5' B N12W/6SW @50' B N10W/6SW
55	36			2.5	101.0	Total Depth - 57' No grounwater Caving-very friable from 1.5-19' and 39-57'	
60							
65							
70							
75							
80							
85							

ADDITIONAL COMMENTS: Blows per 12"

Kelly Bar Weights: 0 - 24', 3800 lbs.
24 - 47', 2800 lbs.
47 - 74', 1800 lbs.

Logged by: SBS

SUBSURFACE DATA

LOG OF BORING B7

CLIENT: Newton	PROJECT: Everett Terrace	W.O.: 8953
LOCATION: Moorpark	ELEVATION: 582'±	DATE: 3/8/05
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 15"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						<u>Fill</u>: Dark brown fine to coarse grained SAND with clay and silt, abundant roots, trace paper debris, medium dense to dense, moist.	
						<u>Saugus Formation</u>: Irregular contact, light brown medium to coarse grained	
5	36	C		10.7	123.8	SANDSTONE with fine gravel, very dense, moist, massive, poorly cemented, trace rootlets.	
						12" thick brown silty fine grained SANDSTONE, dense, moist. 3-4" diameter tree root.	@8' B N30E/10SE
10	27	C		10.0	115.5		
						Light brown medium to coarse grained SANDSTONE, well graded, bedded, slightly friable, dense, moist, over fine to medium grained SANDSTONE with clay and gravel, well graded, poorly cemented, dense, moist. @11.5' - Grades to moderately cemented fine to coarse grained	@11' B N21W/5NE
15	47	C		5.4	118.1	SANDSTONE with clay and subangular to subrounded gravel to cobble (5-15%), massive, dense, moist. @16' - Horizontal contact with tan brown thinly bedded horizontal medium to coarse grained SANDSTONE interbed. @17' - 3" thick gravel CONGLOMERATE channel, slightly friable, dense,	@16' Contact B Horizontal
20	53	C		4.8	107.7	moist. @19' - Scoured contact with moderately cemented light brown silty fine grained SANDSTONE, very dense, moist. @20' - Light brown medium grained SANDSTONE, thinly bedded, friable, over 12" thick light brown moderately cemented SILTSTONE, hard, moist.	@20' scoured upper contact N80W/14SW @21' sharp contact
25	47	C		1.9	117.2	@21' - Sharp contact with 3" thick gravelly SANDSTONE, slightly friable, to brown clayey fine grained SANDSTONE, very dense, moist, massive. @24' - Grades to damp, slightly friable light brown fine to coarse grained SANDSTONE with subangular to subrounded gravel, dense, massive. @26' - Thin undulating Paleosols within light brown slightly friable fine to	N24W/3SW @27' Paleosol N55E/17SE N75W/8SW
30	60	C		2.1	117.1	coarse grained SANDSTONE with gravel, dense, damp, massive. Light brown thin bedded fine to coarse grained SANDSTONE with fine gravel, damp to massive, slightly friable, dense.	@29' B N20W/2SW @31' B N23W/4SW
35	68	C		4.0	109.8	@35.5' - 12" thick light brown fine grained silty SANDSTONE, hard, moist to dry, friable, medium to coarse grained SANDSTONE, caving.	
40						Total Depth - 37' No groundwater Caving at 36.5' Blows per 12"	
45							

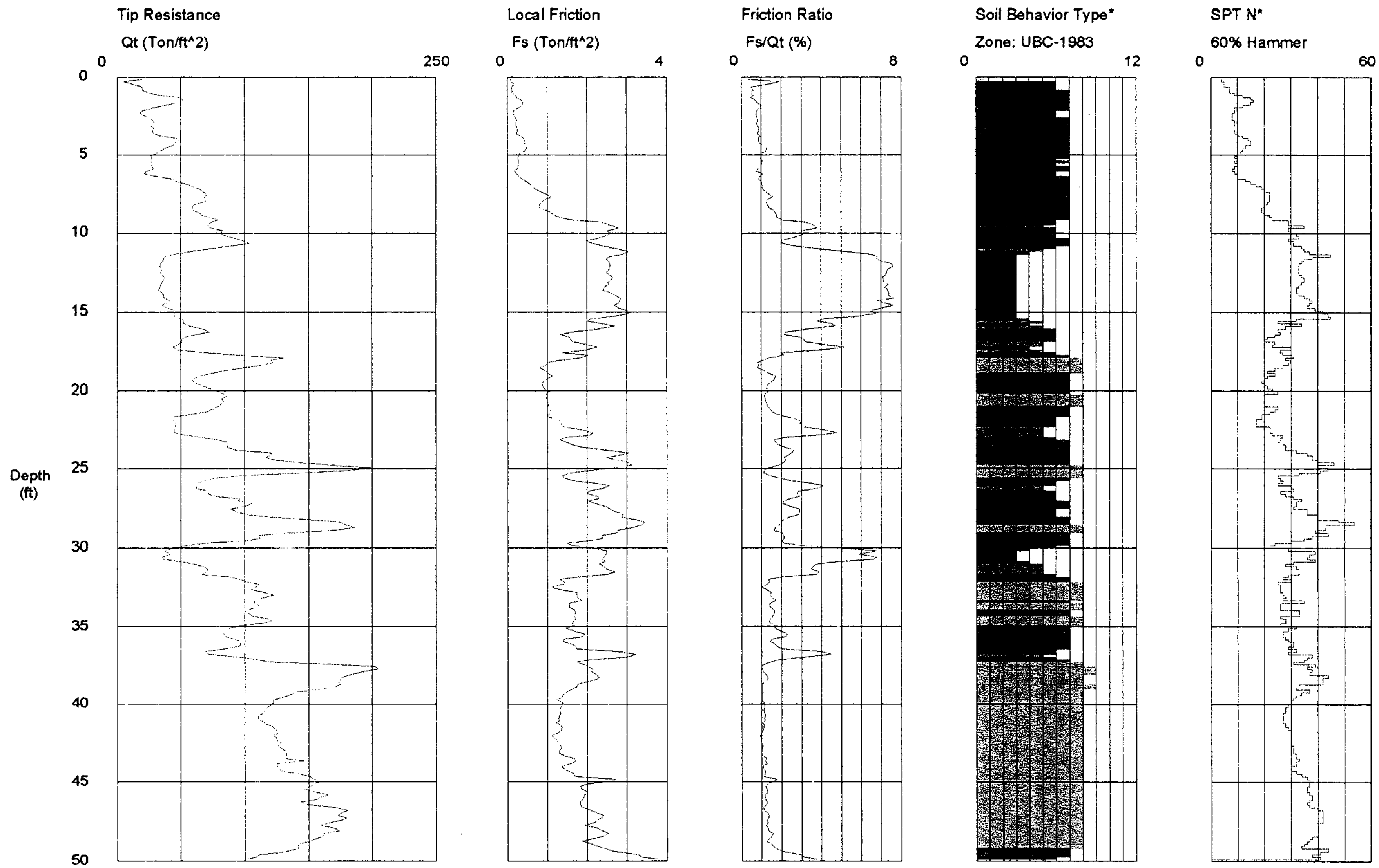
ADDITIONAL COMMENTS:

Kelly Bar Weights:				
	0-8'	866 lbs.	30.5-38'	326 lbs.
	8-15.5'	703 lbs.	38-45.5'	245 lbs.
	15.5-23'	556 lbs.	45.5-52'	172 lbs.
	23-30.5'	430 lbs.		

Geolabs Westlake Village

Operator: VO/ML
Sounding: CPT-01
Cone Used: DSA0472

CPT Date/Time: 6/29/2004 9:14:
Location: Everest Terrace
Job Number: 8953



Maximum Depth = 50.20 feet

Depth Increment = 0.164 feet

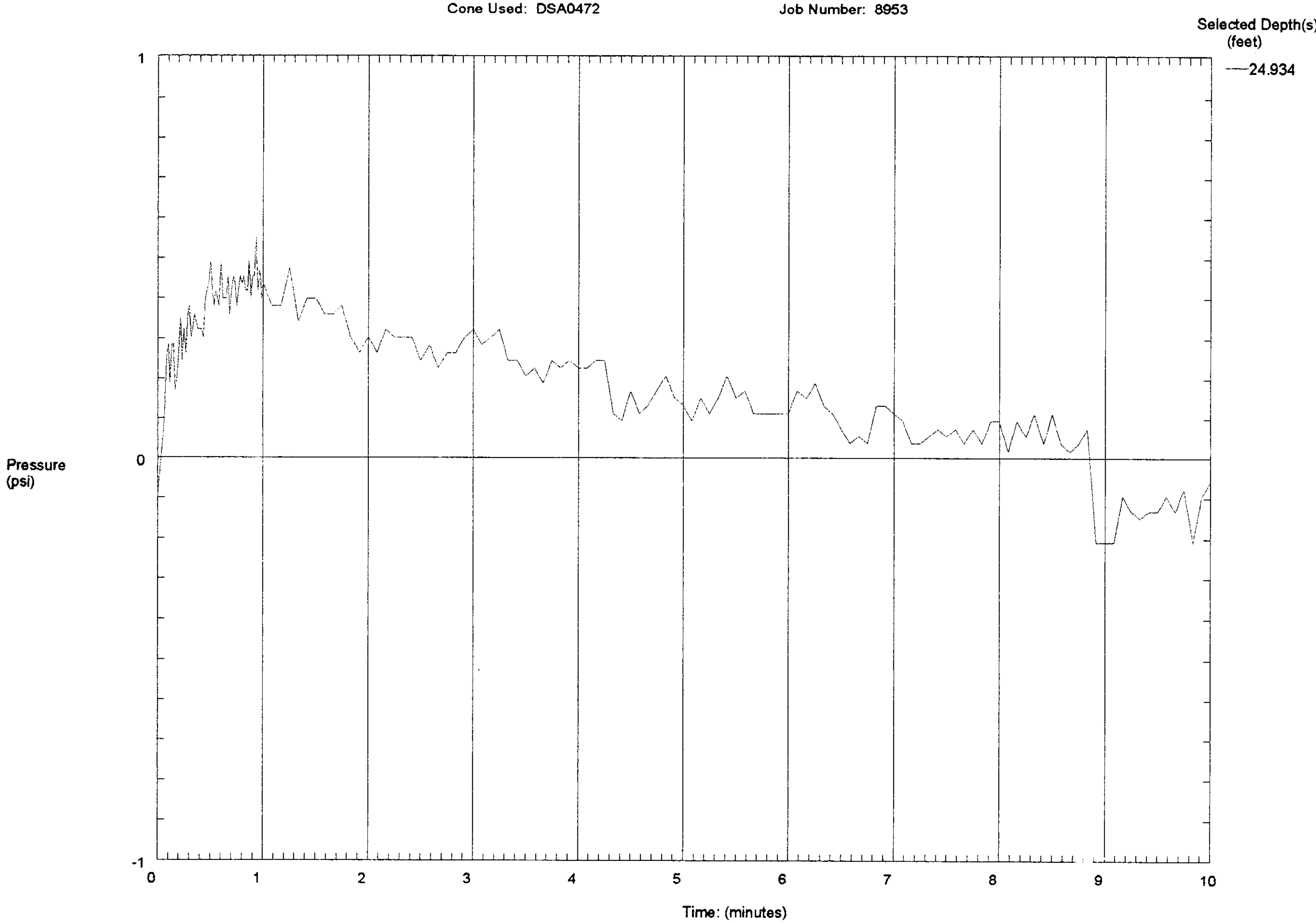
- 1 sensitive fine grained
- 2 organic material
- 3 clay
- 4 silty clay to clay
- 5 clayey silt to silty clay
- 6 sandy silt to clayey silt
- 7 silty sand to sandy silt
- 8 sand to silty sand
- 9 sand
- 10 gravelly sand to sand
- 11 very stiff fine grained (*)
- 12 sand to clayey sand (*)

Soil behavior type and SF ,d on data from UBC-1983

Geolabs Westlake Village

Operator VO/ML
Sounding: CPT-01
Cone Used: DSA0472

CPT Date/Time: 6/29/2004 9:14:00 AM
Location: Everest Terrace
Job Number: 8953

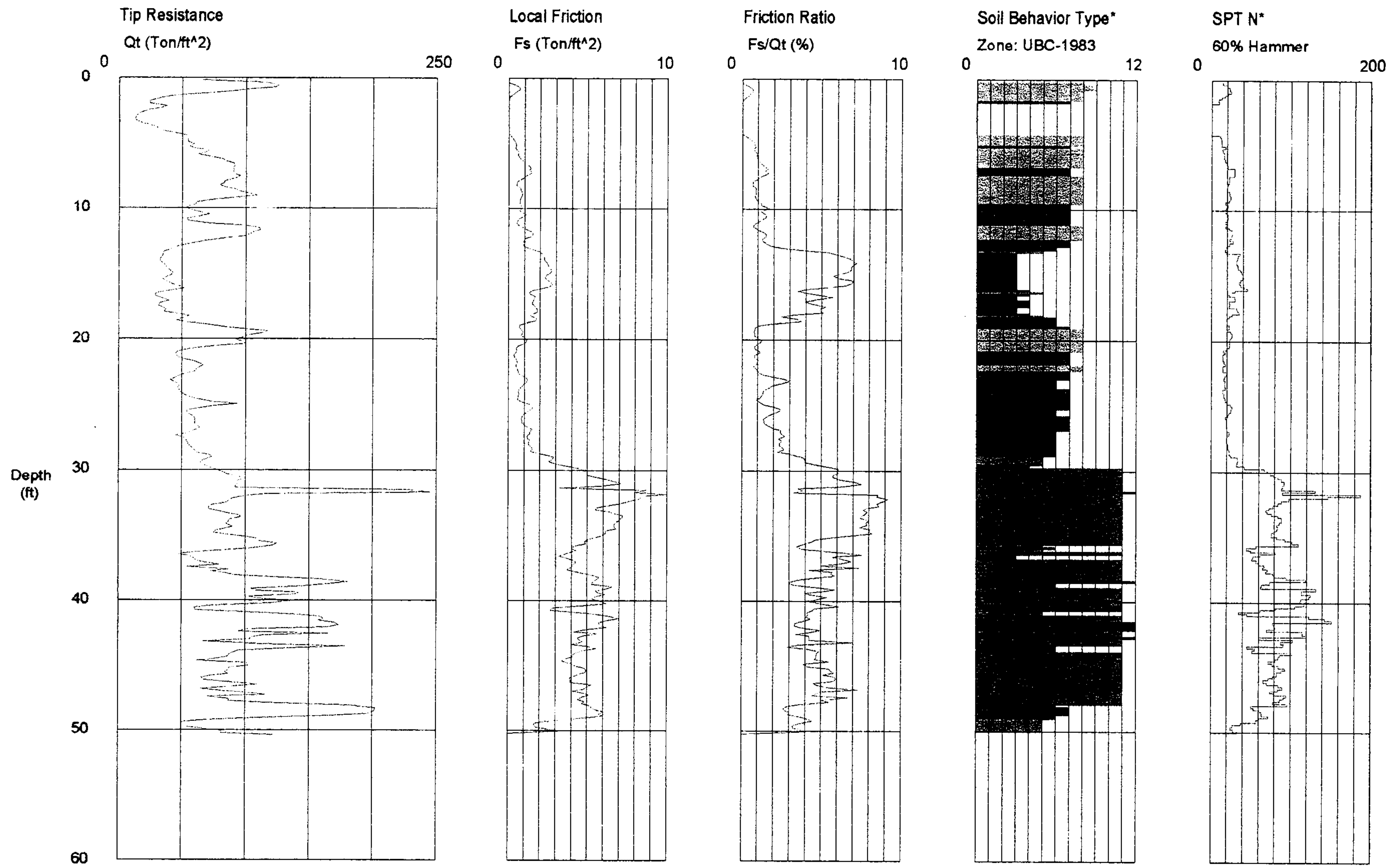


Maximum Pressure = 0.55 psi
Hydrostatic Pressure = 10.821 psi

Geolabs Westlake Village

Operator: VO/ML
Sounding: CPT-02
Cone Used: DSA0472

CPT Date/Time: 6/29/2004 10:33
Location: Everest Terrace
Job Number: 8953



Maximum Depth = 50.36 feet

Depth Increment = 0.164 feet

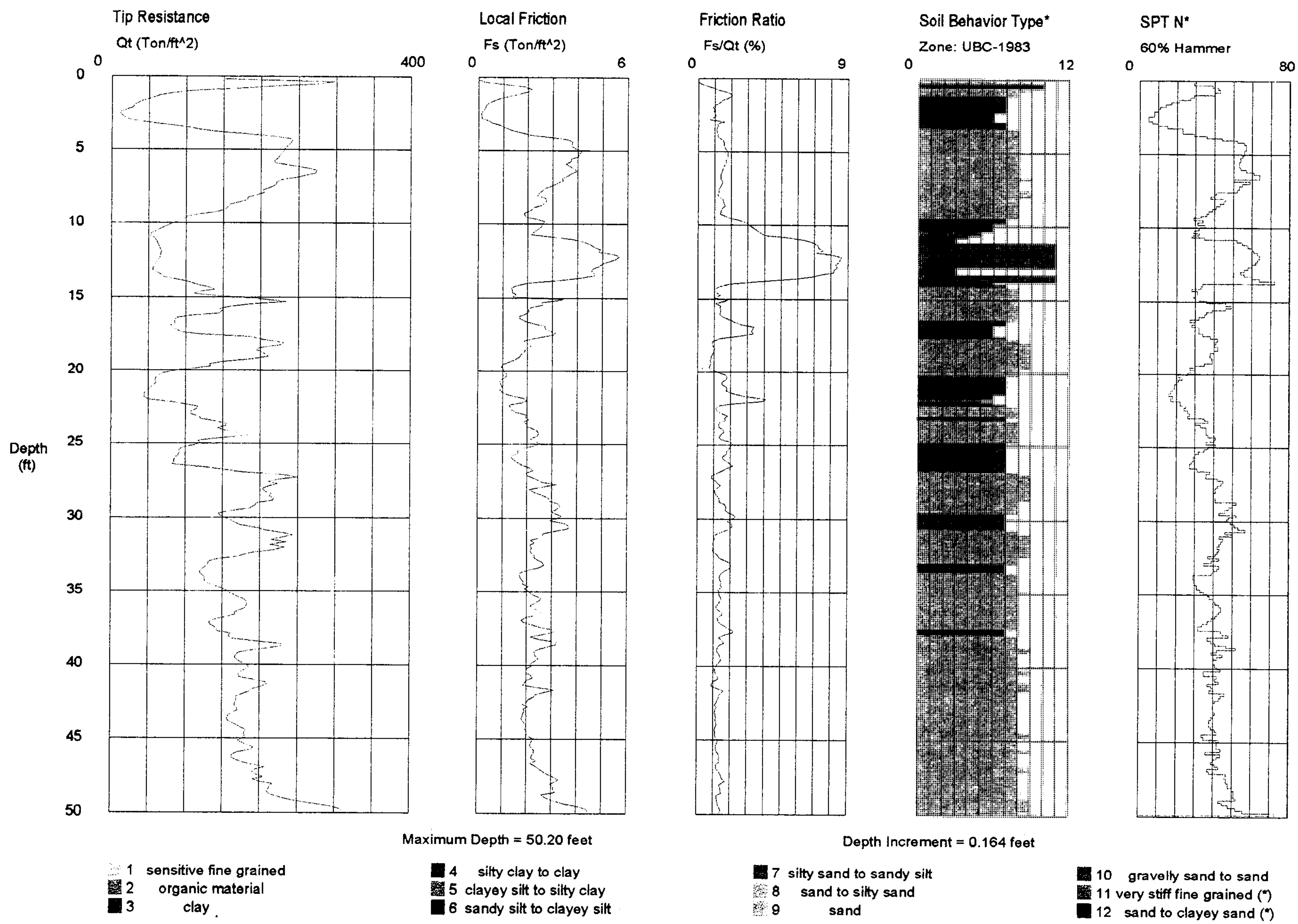
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Soil behavior type and SF d on data from UBC-1983

Geolabs Westlake Village

Operator: VO/ML
Sounding: CPT-03
Cone Used: DSA0472

CPT Date/Time: 6/29/2004 11:25
Location: Everest Terrace
Job Number: 8953



Soil behavior type and SF d on data from UBC-1983

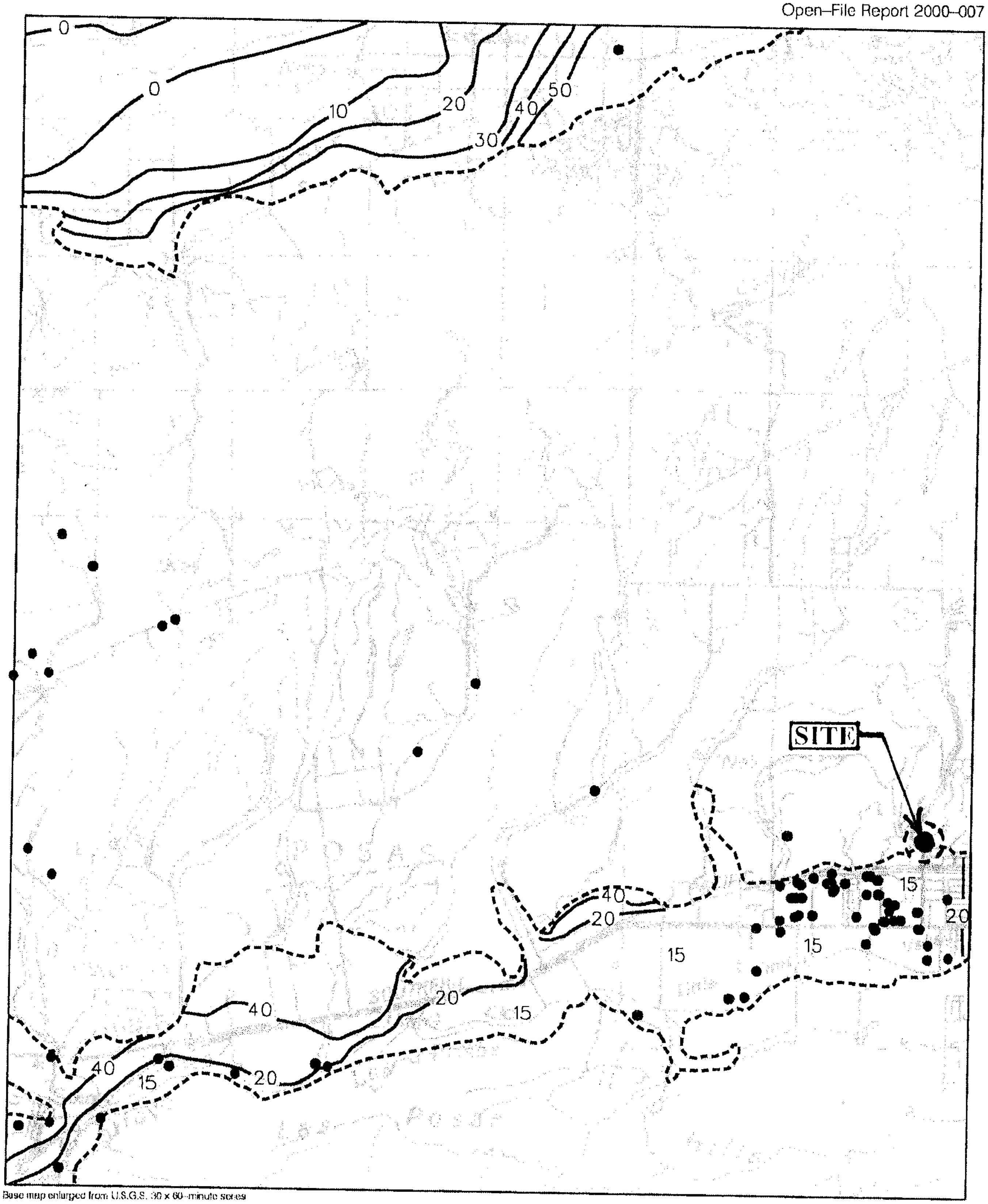


Plate 1.2 Historically shallow ground–water depths and borehole data points in alluviated valley areas of the Moorpark Quadrangle.

Alluviated Valley

20 Historically shallow ground–water depth contours (in feet)

Borehole Site

15 Historically shallow ground–water depth where same value occurs over a broad area (in feet)

ONE MILE

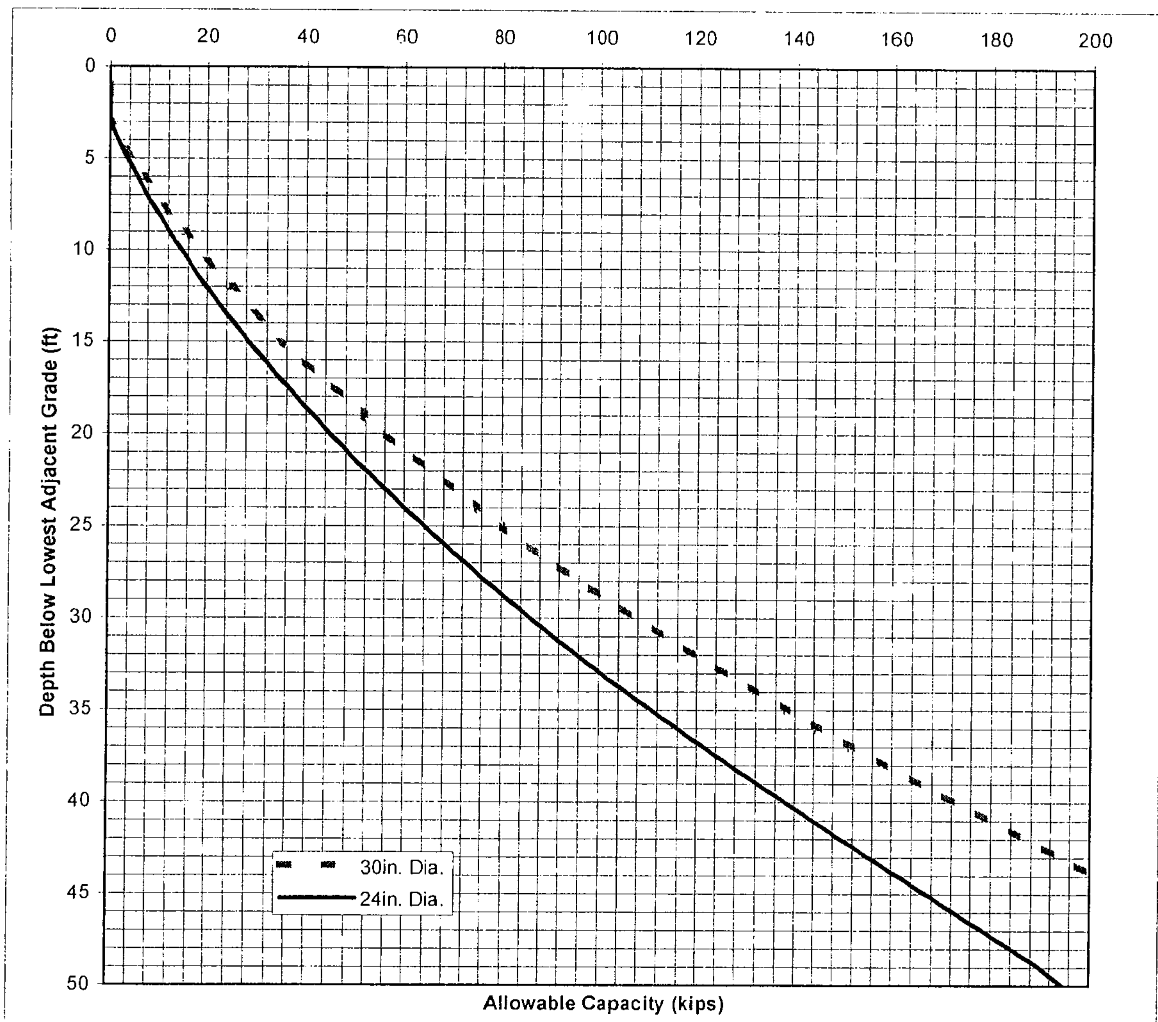
Scale

FRICTION CIDH PILE CAPACITIES (drilled,cast-in-place)

Soil Data: No.	Density	Phi (deg)	Cohesion (psf)	Bottom of Layer
Saugus Form. 1	125	27	500	60
2				
3				
4				
5				

Pile Configuration:

Non-Bearing Depth=	3.00	ft.
Min. Diameter=	2.00	ft.
Min. Embeddment=		ft.
Depth Increment=	1.00	ft.
Depth to Water=	50.00	ft.
Factor of Safety=	2.00	



APPENDIX A

LABORATORY TEST RESULTS

Laboratory Test Summary

W.O. 8953

Depth	Geology	Sample Description	ST	w	DD	S	Max	Opt	EI	LL	PI	e	n	WD	SD	BD	Consol	Shear
Excavation: B1 (TD= 51.5 ft, No GW)																		
0	Engineered Fill	Silty Sand	(B)				129	8	0									S-B1.0
2.5	Alluvium	Silty Sand	(U)	4.4	104	19						0.62	0.38	108	127	64.7		
5	Alluvium	Silty Sand	(U)	6.6	107	31						0.57	0.36	114	129	66.7		
7.5	Alluvium	Silty Sand	(U)	6.2	119	41						0.41	0.29	127	137	74.6		
10	Alluvium	Silty Sand	(U)	8.2	113	46						0.48	0.32	122	133	70.9	C-B1.10	
12.5	Alluvium	Silty Clay with Sand	(U)	18.2	112	100						0.49	0.33	133	133	70.5		
15	Alluvium	Clay	(U)	17.8	109	88				56	39	0.54	0.35	128	130	68	C-B1.15	
25	Alluvium	Silt with Clay	(S)							NP	NP							
35	Alluvium	Clayey Silty Sand	(S)							26	13							
47.5	Alluvium	Clayey Silty Sand	(S)							NP	NP							
Excavation: B2 (TD= 16.5 ft, No GW)																		
6	Alluvium	Sand	(U)	3	112	16						0.50	0.34	116	132	69.7		S-B2.6
15	Alluvium	Silty Sand	(U)	2														
Excavation: B3 (TD= 40 ft, No GW)																		
5	Alluvium	Silty Sand	(U)	3.3	104	14						0.63	0.39	107	127	64.2	C-B3.5	
15	Alluvium	Sand	(U)	2.7	107	13						0.56	0.36	110	130	67.3	C-B3.15	
25	Alluvium	Clayey Sand	(U)	8	114	46						0.47	0.32	123	134	71.5		
35	Saugus Formation	Claystone	(U)	23.6	98.4	90						0.70	0.41	122	124	61.6		
Excavation: B4 (TD= 30 ft, No GW)																		
10	Alluvium	Silty Sand	(U)	6	123	44						0.37	0.27	130	139	76.8	C-B4.10	
25	Alluvium	Silt	(U)	10	115	60						0.45	0.31	127	135	72.4		
Excavation: B5 (TD= 70 ft, No GW)																		
41	Saugus Formation	clayey SILTSTONE	(U)	25	93.5	85						0.79	0.44	117	121	58.6		
42	Saugus Formation	clayey SILTSTONE	(U)	18.5	110	95						0.52	0.34	130	131	68.9		S-B5.42
43	Saugus Formation	clayey SILTSTONE	(U)							63	37							
46	Saugus Formation	CLAY	(U)							67	39							
50	Saugus Formation	SILTSTONE	(U)	4.4	121	31						0.38	0.28	126	138	76		

For abbreviation explanation see Legend on PLATE LS 2

GEOLABS-WESTLAKE VILLAGE

Depth	Geology	Sample Description	ST	w	DD	S	Max	Opt	EI	LL	PI	e	n	WD	SD	BD	Consol	Shear
Excavation: B6 (TD= 57 ft, No GW)																		
5	Alluvium	Sand	(U)	2.2	105	10						0.59	0.37	107	128	65.9		
10	Alluvium	Sand	(U)	1.8	113	10						0.48	0.32	115	133	70.7		
15	Alluvium	Sand	(U)	2.6	117	16						0.44	0.30	120	135	73.0		
20	Saugus Formation	SANDSTONE	(U)	3.7	114	21						0.47	0.32	118	134	71.2	C-B6.20	
25	Saugus Formation	clayey SILTSTONE and silty SANDSTONE	(U)	14.5	108	71						0.55	0.35	124	130	67.8		S-B6.25
30	Saugus Formation	SANDSTONE	(U)	6	103	26						0.62	0.38	109	127	64.8	C-B6.30	
35	Saugus Formation	clayey SILTSONE	(U)	2.4	98.2	9						0.71	0.42	101	124	61.1		
40	Saugus Formation	silty SANDSTONE	(U)	2.7	102	11						0.66	0.4	105	126	63.2		
45	Saugus Formation	SANDSTONE	(U)	3.3	104	15						0.59	0.37	108	128	65.9		
50	Saugus Formation	clayey SILTSTONE	(U)	15.2	106	71						0.57	0.37	123	129	66.6		S-B6.50
55	Saugus Formation	SANDSTONE	(U)	2.5	101	10						0.67	0.40	104	125	62.8		
Excavation: B7 (TD= 37 ft, No GW)																		
5	Saugus Formation	SANDSTONE	(U)	10.7	124	82						0.35	0.26	137	144	81.5		
10	Saugus Formation	SANDSTONE	(U)	10	116	60						0.45	0.31	127	145	82.7		
15	Saugus Formation	SANDSTONE	(U)	5.4	118	35						0.41	0.29	125	154	91.6		
20	Saugus Formation	SANDSTONE	(U)	4.8	108	23						0.56	0.36	113	155	92.9		
25	Saugus Formation	SANDSTONE	(U)	1.9	117	12						0.42	0.3	119	162	99.8		
30	Saugus Formation	SANDSTONE	(U)	2.1	117	13						0.43	0.30	120	162	99.2		
35	Saugus Formation	silty SANDSTONE	(U)	4	110	20						0.54	0.35	114	157	94.7		

LEGEND

Depth = Sample Depth (ft) below ground surface
ST = Sample Type*
w = Initial Moisture Content (%)
DD = Initial Dry Unit Weight (pcf)
Max = Maximum Dry Unit Weight (pcf)
Opt = Optimum Moisture Content (%)
EI = Expansion Index
S = Degree of Saturation (%)

LL = Liquid Limit
PI = Plasticity Index
e = Void Ratio
n = Porosity
WD = Initial Wet Unit Weight (pcf)
SD = Saturated Unit Weight (pcf)
BD = Bouyant (Submerged) Unit Weight (pcf) - Assuming water unit weight of 62.4 pcf

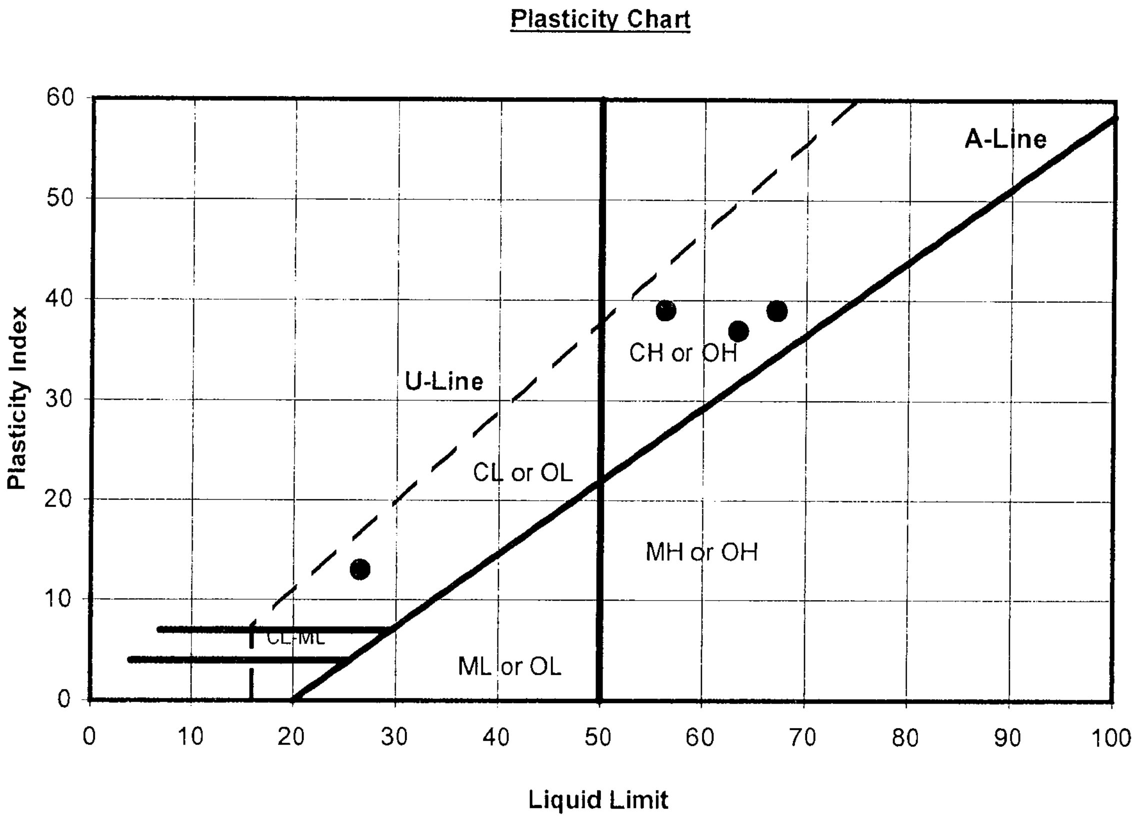
Consol = Consolidation Test Diagram (Plate No.)
Shear = Shear Test Diagram (Plate No.)

* Sample Types: (U) = relatively Undisturbed; (S) = SPT; (B) = Bulk

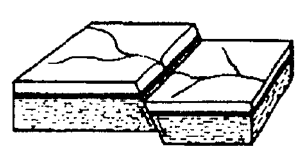
GEOLABS-WESTLAKE VILLAGE

PLATE LS. 2

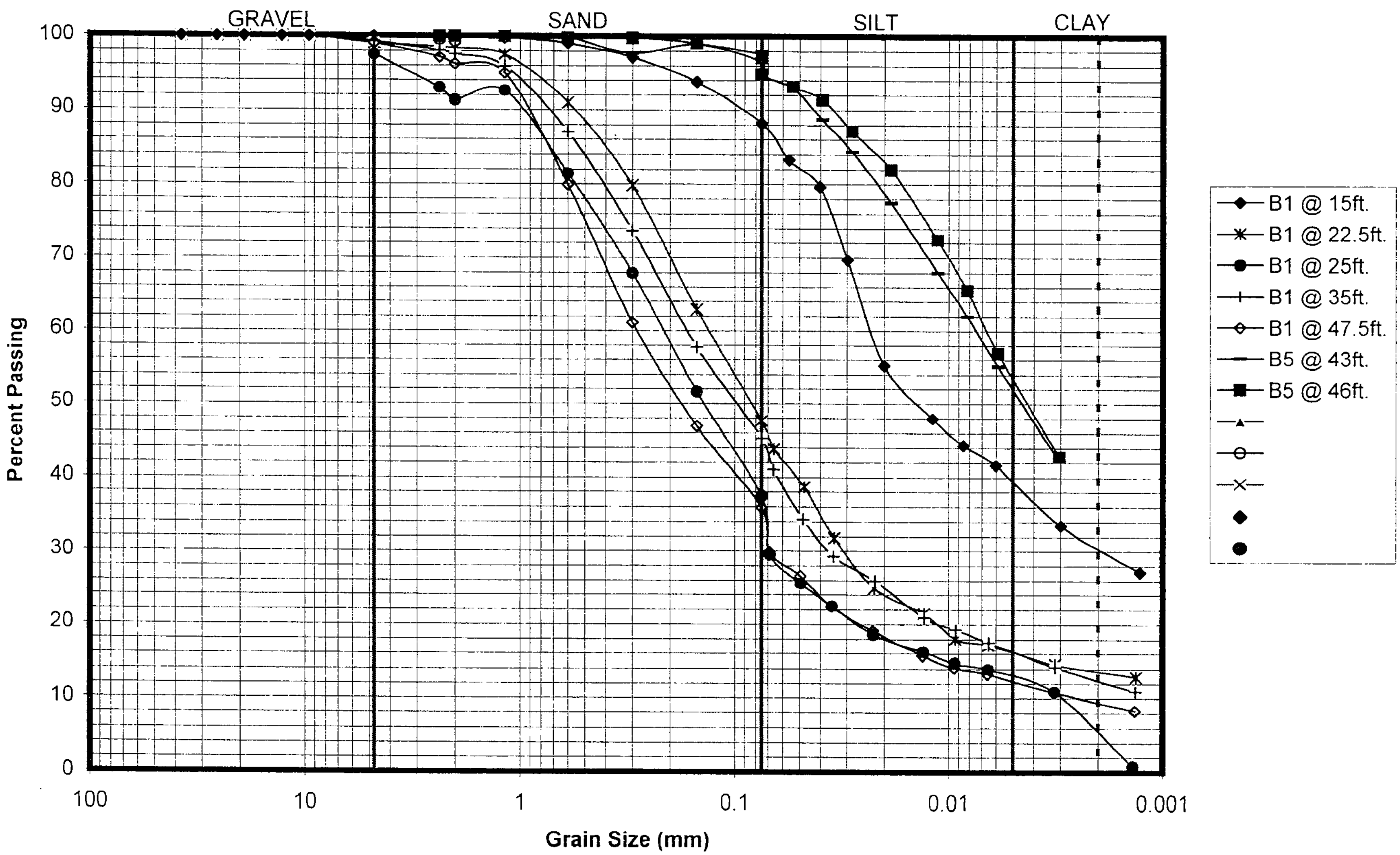
Atterberg Limits Test Results



Location	Depth (ft)	LL	PI	Classification
B1	15	56.3	39	CH
B1	35	26.5	13	CL
B5	43	63.4	37	CH
B5	46	67.2	39	CH
B1	25	-	-	non-plastic
B1	47.5	-	-	non-plastic



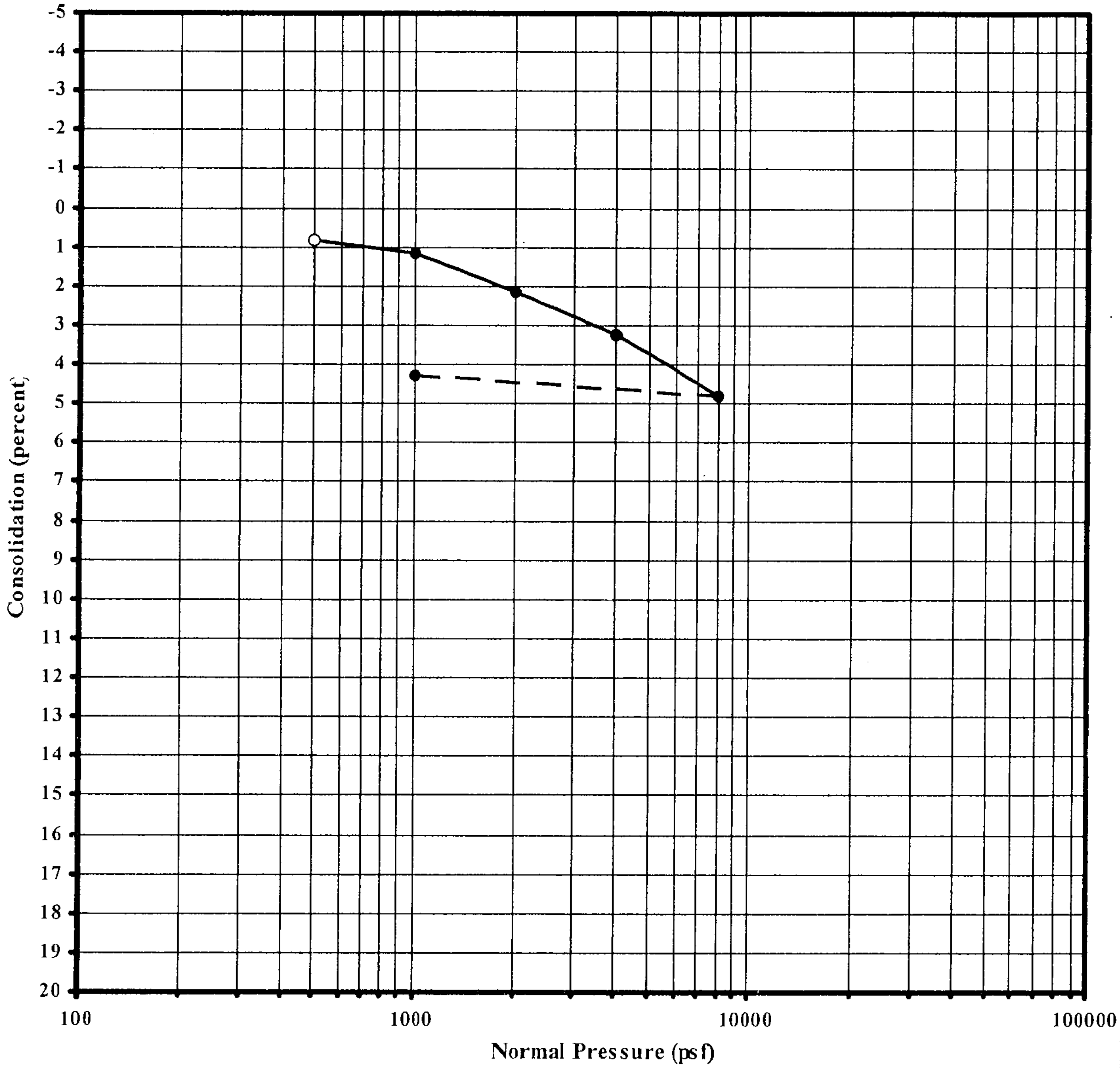
PARTICLE SIZE ANALYSIS



GEOLABS-WESTLAKE VILLAGE

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: B1

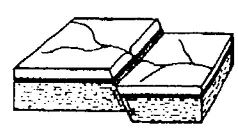
Sample Depth: 10 ft.

Initial Moisture: 8.2 %

Init. Dry Density: 113.1 pcf

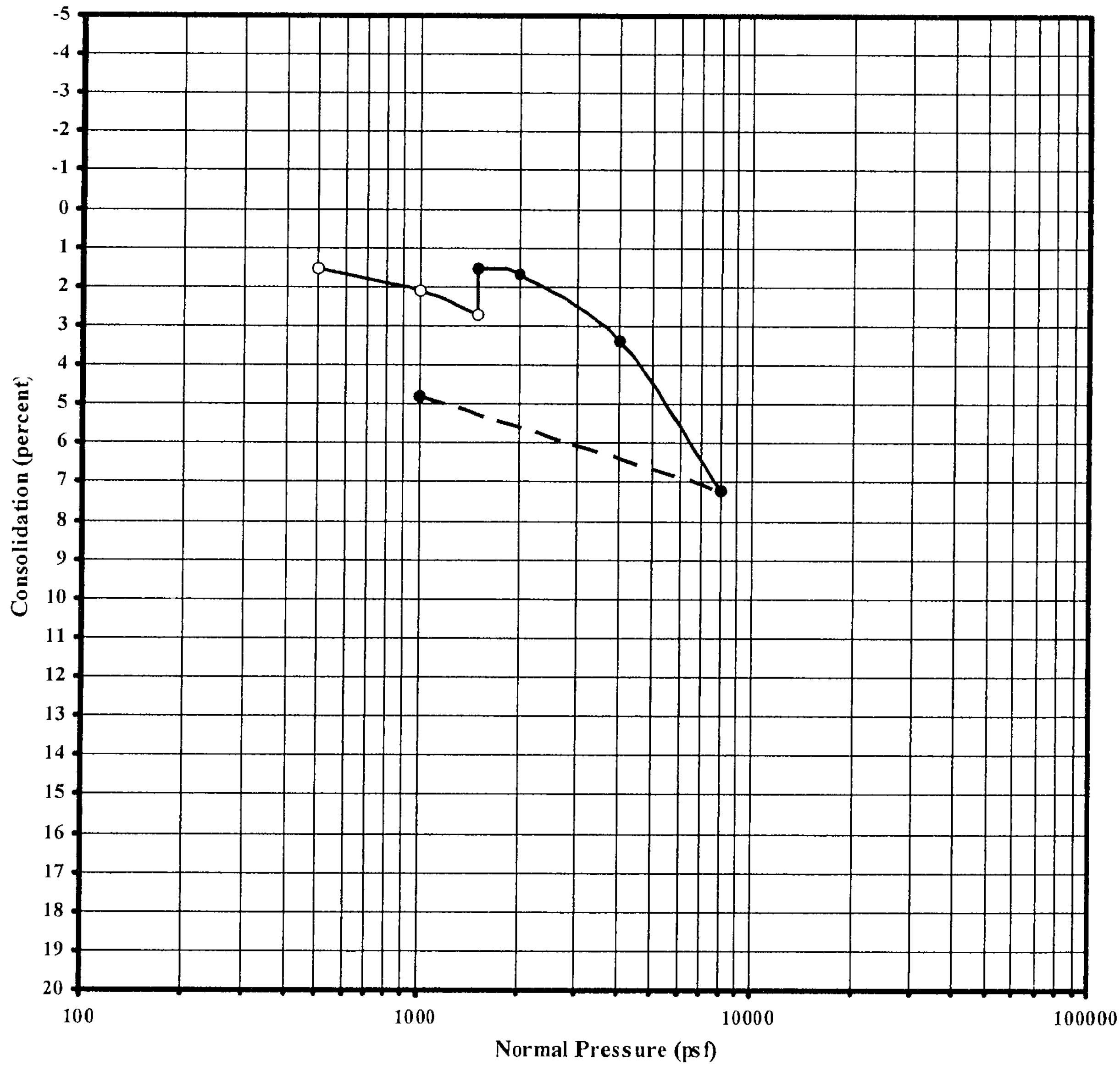
Geologic Unit: Alluvium

Material: Silty Sand



CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1500 psf

Sample Location: B1

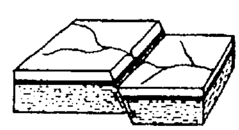
Sample Depth: 15 ft.

Initial Moisture: 17.8 %

Init. Dry Density: 108.5 pcf

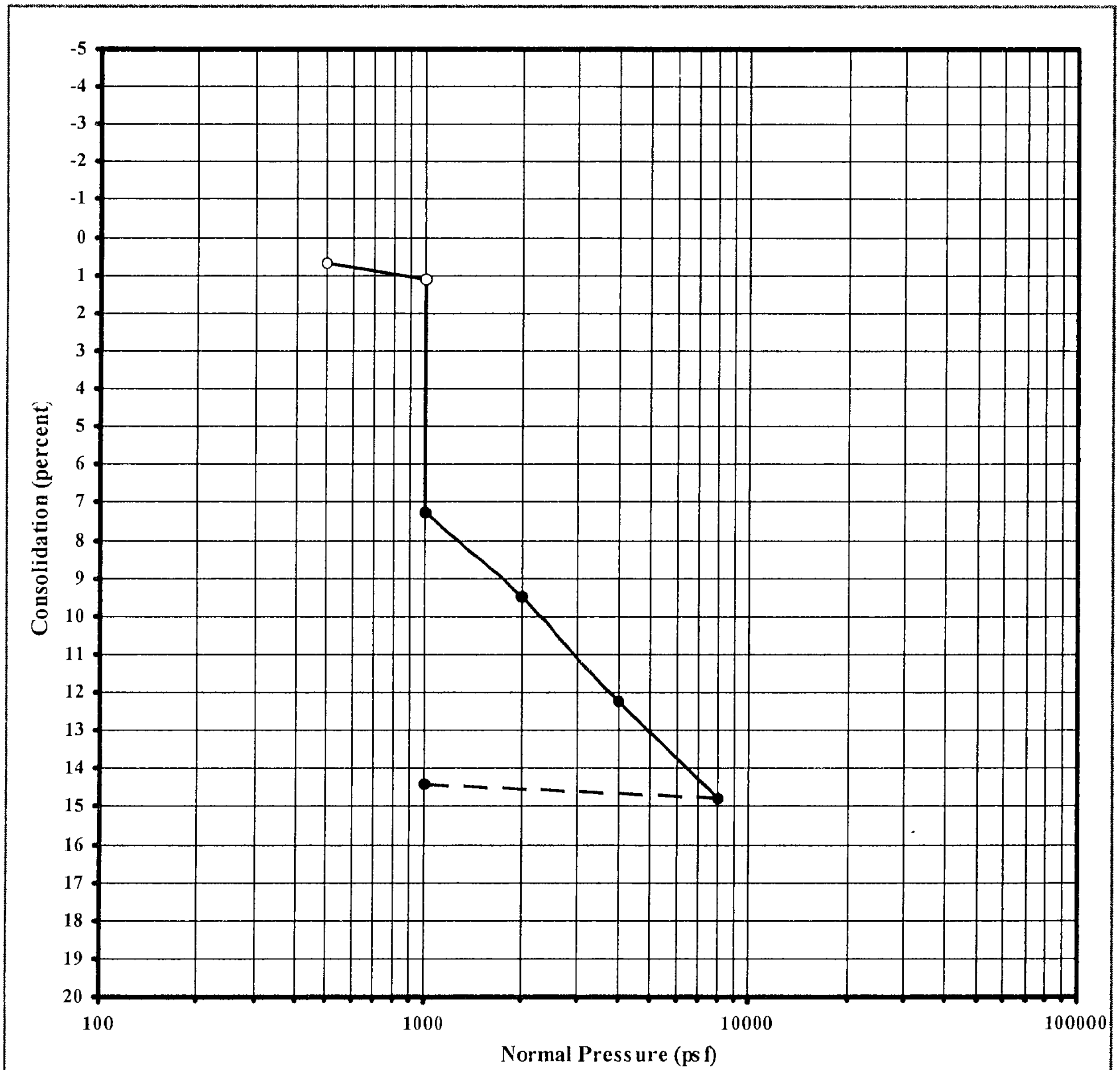
Geologic Unit: Alluvium

Material: Clay



CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: B3

Sample Depth: 5 ft.

Initial Moisture: 3.3 %

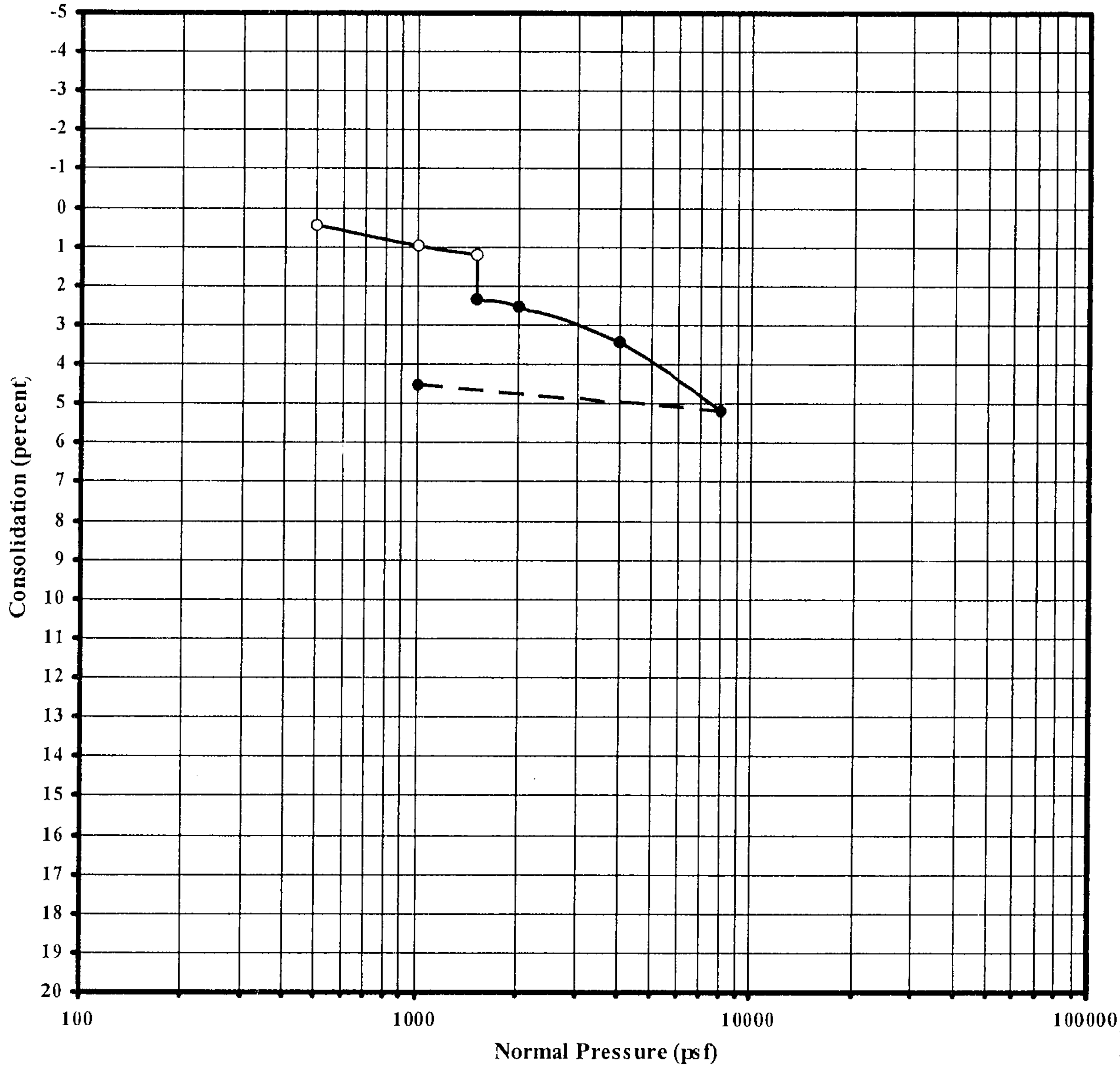
Init. Dry Density: 103.7 pcf

Geologic Unit: Alluvium

Material: Silty Sand

CONSOLIDATION RESULTS

Undisturbed Sample



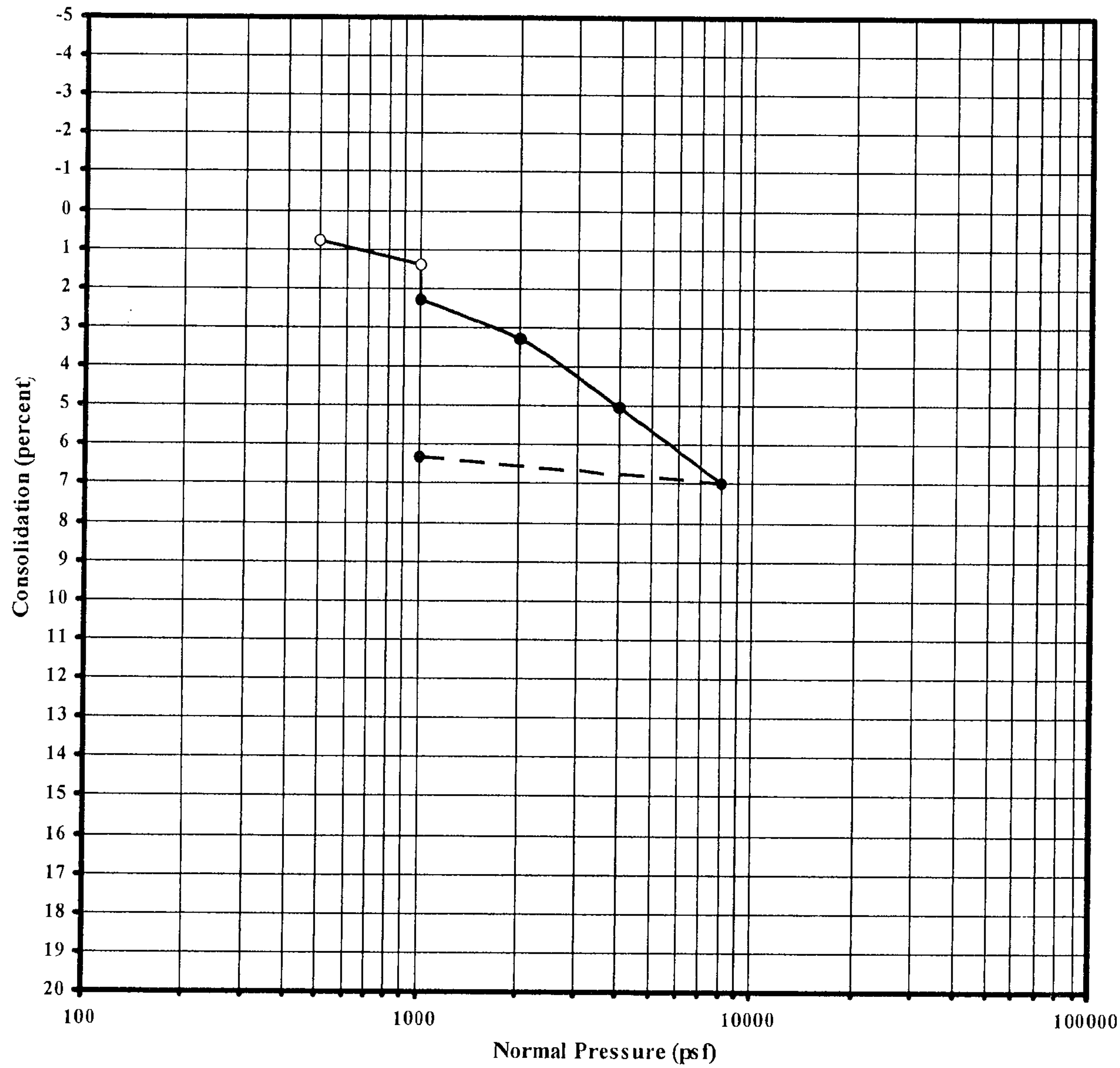
Sample Inundated At Normal Pressure of 1500 psf

Sample Location: B3
Sample Depth: 15 ft.
Initial Moisture: 2.7 %
Init. Dry Density: 107 pcf

Geologic Unit: Alluvium
Material: Sand

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: B4

Sample Depth: 10 ft.

Initial Moisture: 6 %

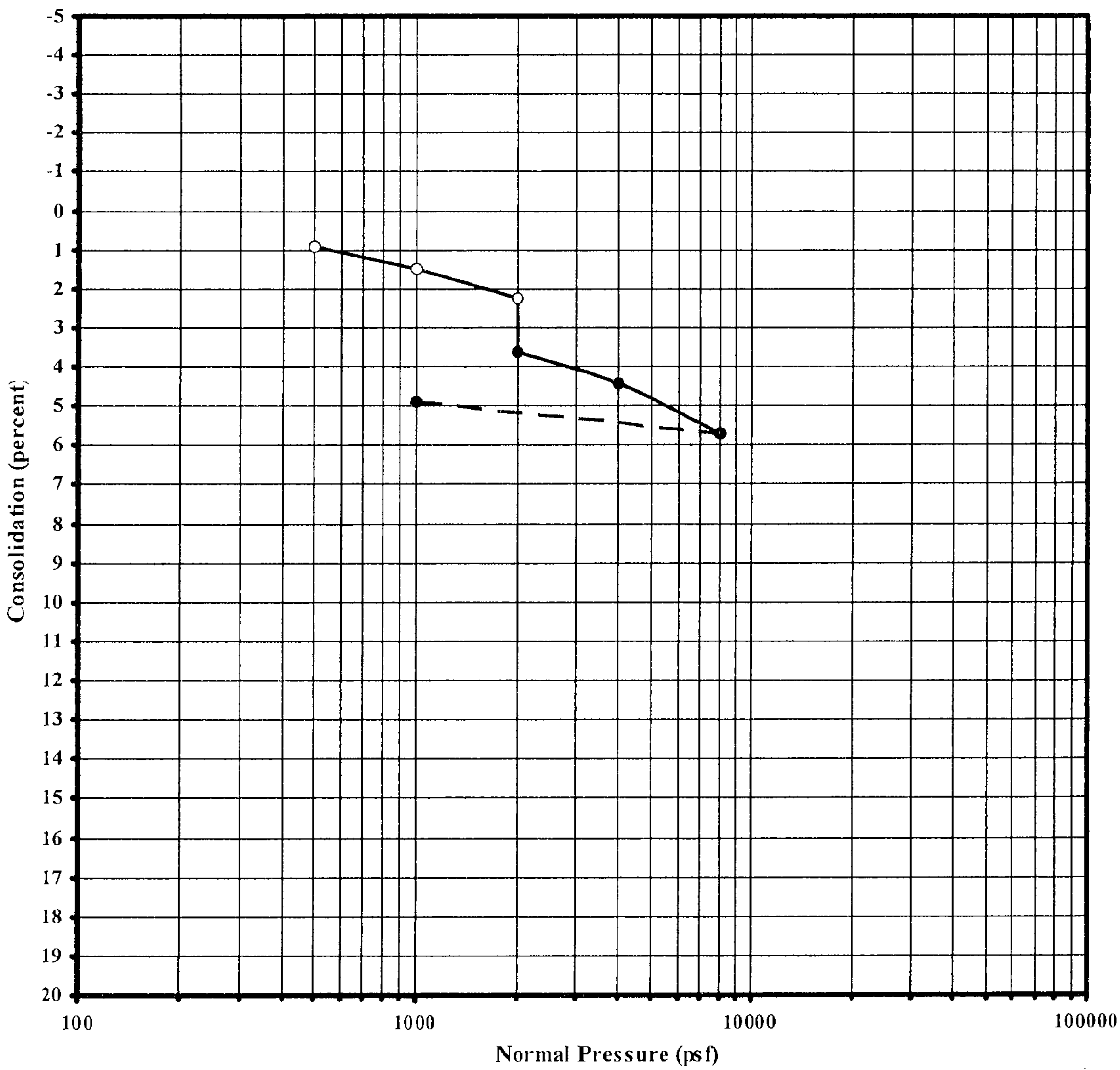
Init. Dry Density: 122.5 pcf

Geologic Unit: Alluvium

Material: Silty Sand

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 2000 psf

Sample Location: B6

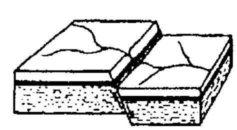
Sample Depth: 20 ft.

Initial Moisture: 3.7 %

Init. Dry Density: 114.1 pcf

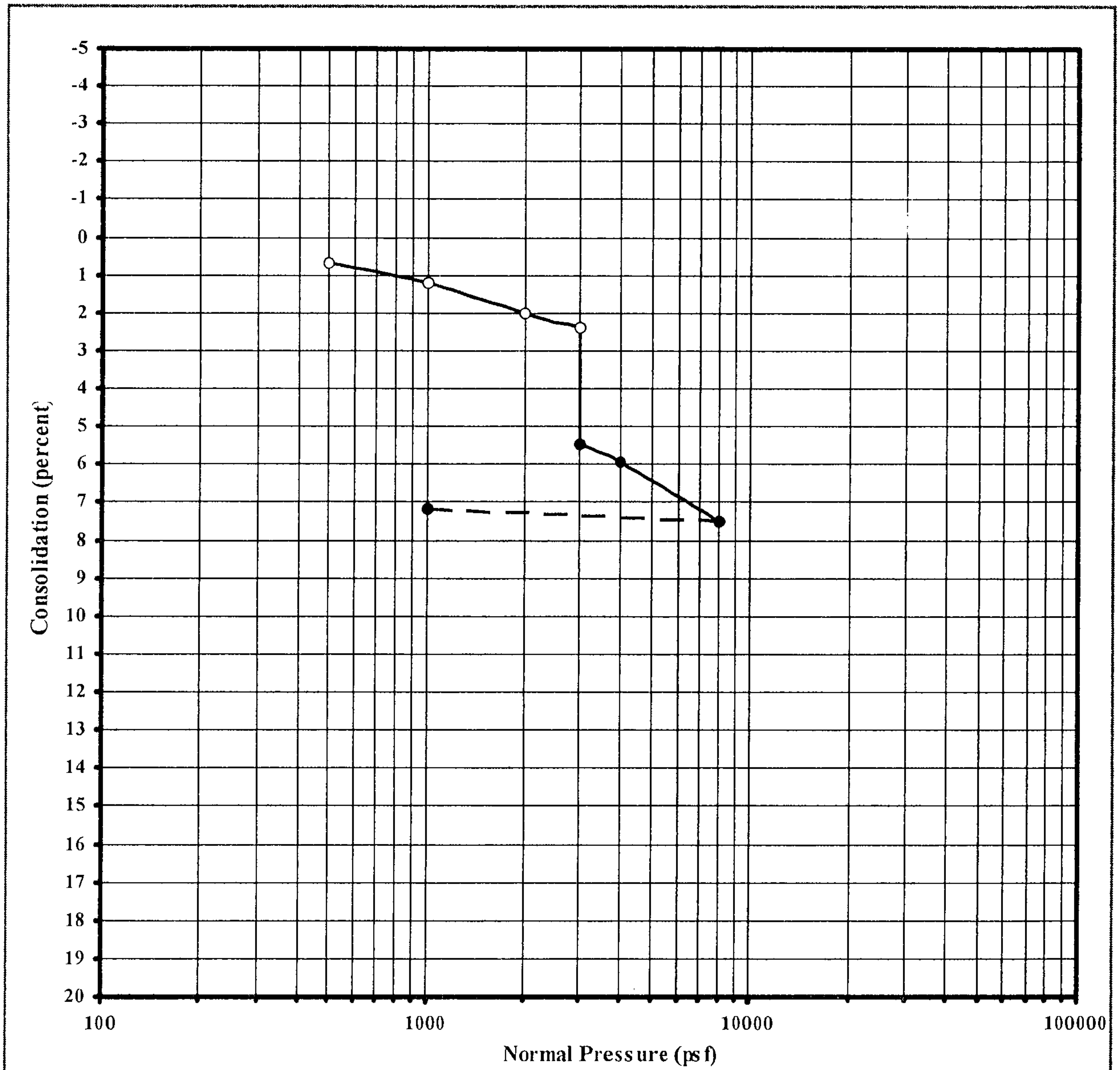
Geologic Unit: Saugus Formation

Material: SANDSTONE



CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 3000 psf

Sample Location: B6

Sample Depth: 30 ft.

Initial Moisture: 6 %

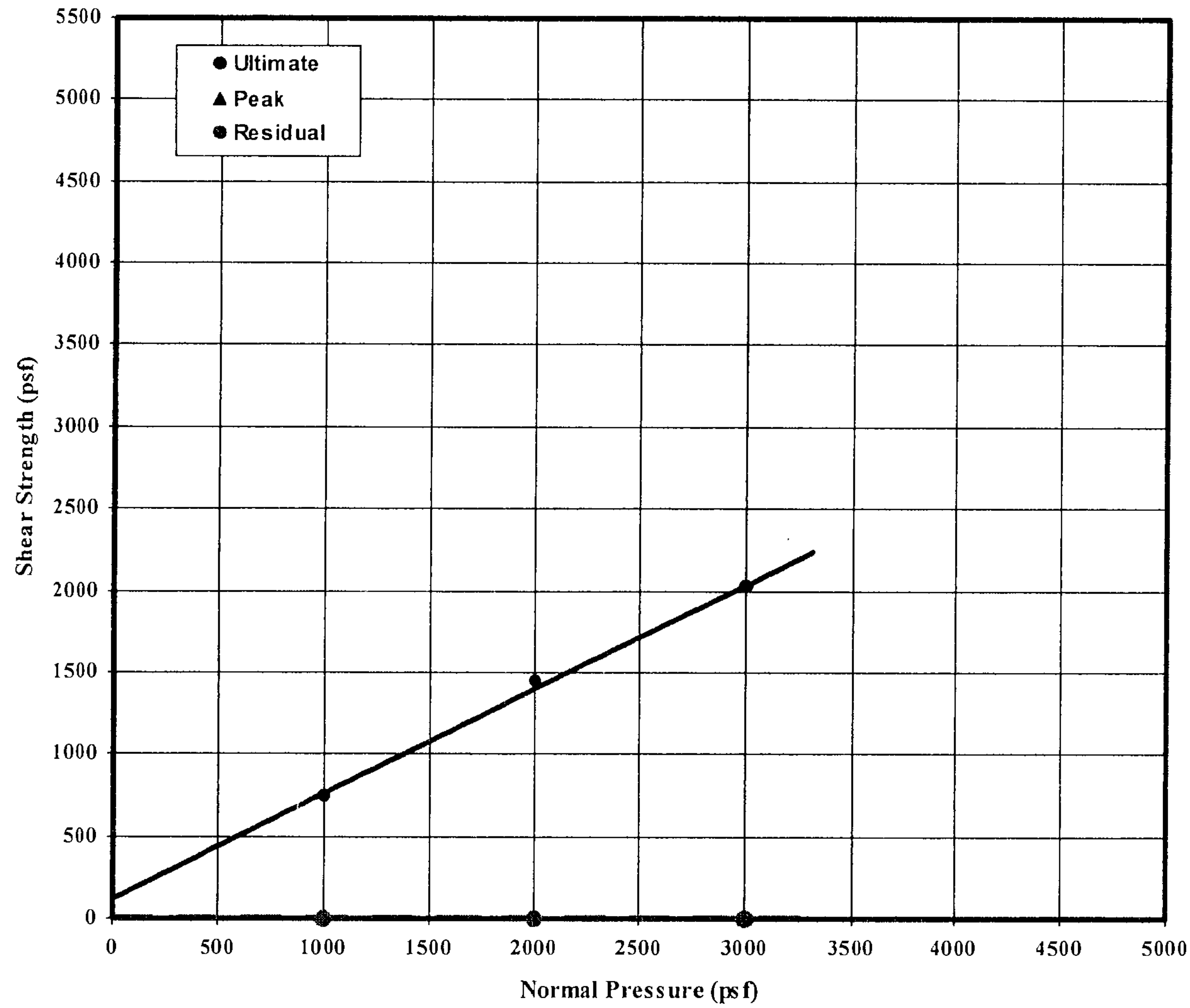
Init. Dry Density: 102.8 pcf

Geologic Unit: Saugus Formation

Material: SANDSTONE

SHEAR TEST RESULTS

Bulk Sample Remolded to 90 Percent Relative Compaction



	Friction Angle	Cohesion
Ultimate Shear Strength:	32 deg	125 psf
Peak Shear Strength:		
Residual Shear Strength:		

Displacement Rate: 0.01 in/min

Sample Location: B1
Sample Depth: 0 ft.
Geologic Unit: Engineered Fill
Material: Silty Sand

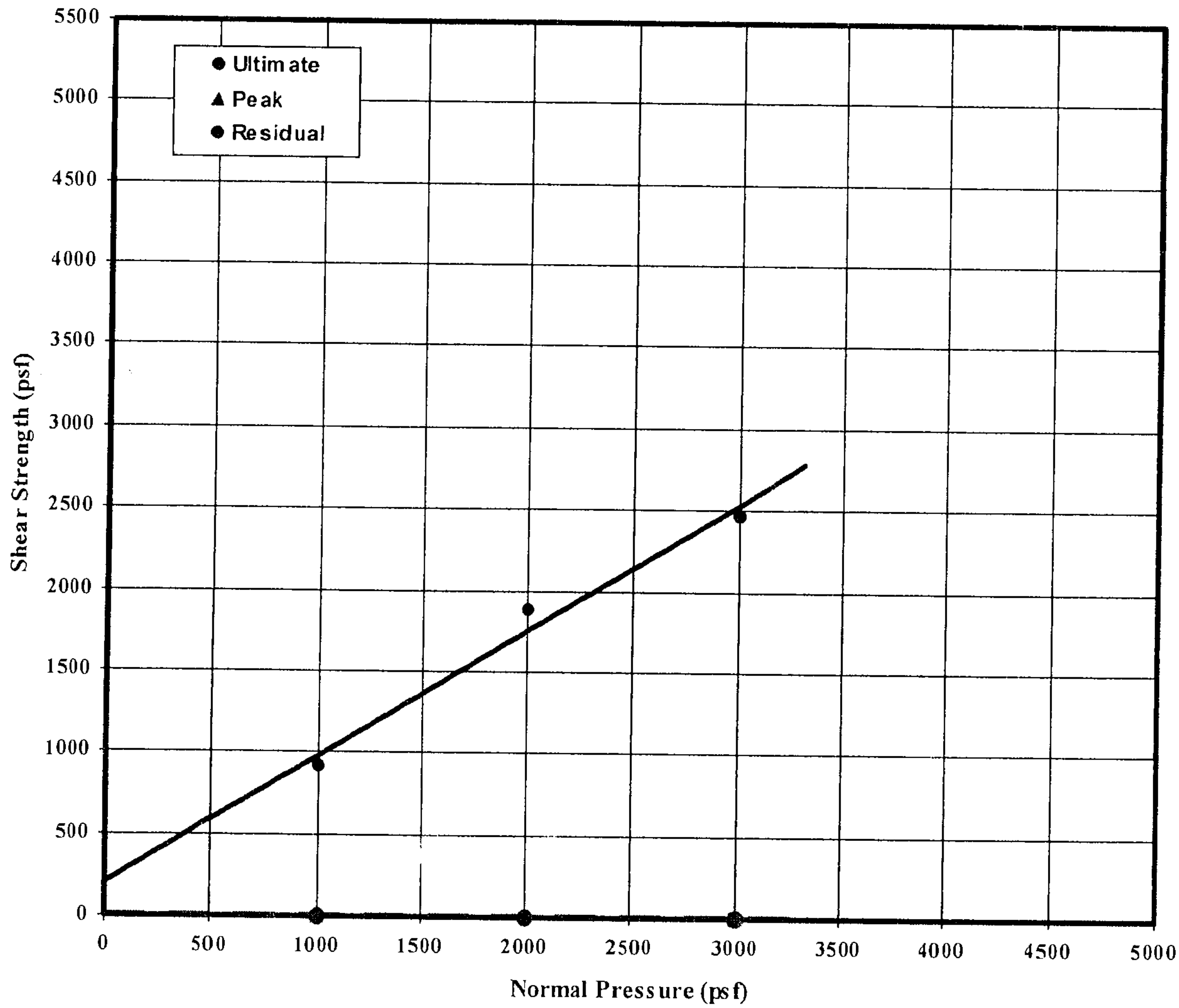
Dry Density:	116 pcf
Moisture:	16.3 %

GEOLABS-WESTLAKE VILLAGE

PLATE S-B1.0

SHEAR TEST RESULTS

Undisturbed Sample



	Friction Angle	Cohesion
Ultimate Shear Strength:	38 deg	200 psf
Peak Shear Strength:		
Residual Shear Strength:		

Displacement Rate: 0.01 in/min

Sample Location: B2
Sample Depth: 6 ft.
Geologic Unit: Alluvium
Material: Sand

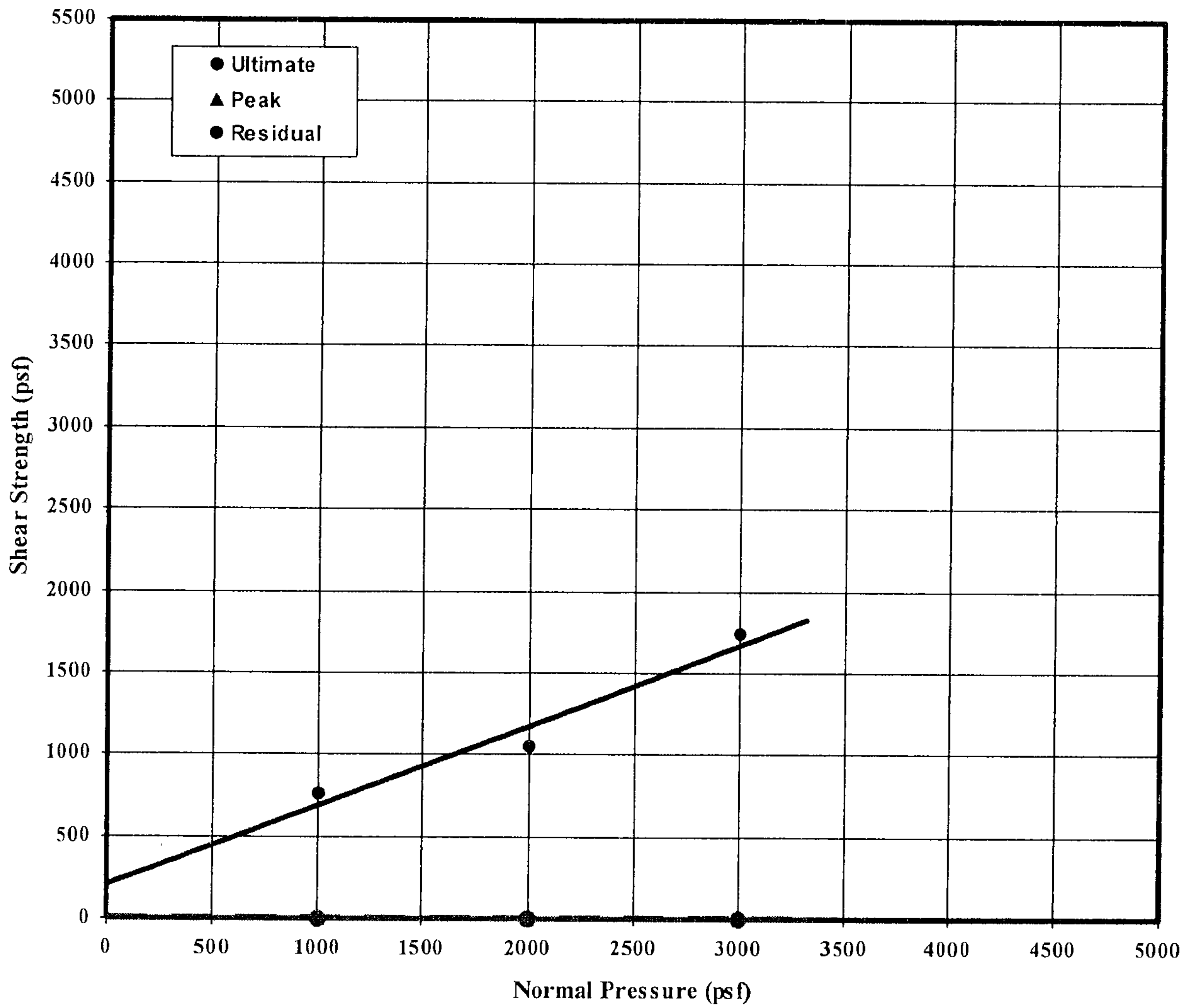
Dry Density:	112.4 pcf
Moisture:	19.3 %

GEOLABS-WESTLAKE VILLAGE

PLATE S-B2.6

SHEAR TEST RESULTS

Undisturbed Sample



	Friction Angle	Cohesion
Ultimate Shear Strength:	26 deg	200 psf
Peak Shear Strength:		
Residual Shear Strength:		

Displacement Rate: 0.01 in/min

Sample Location: B5
Sample Depth: 42 ft.
Geologic Unit: Saugus Formation
Material: clayey SILTSTONE

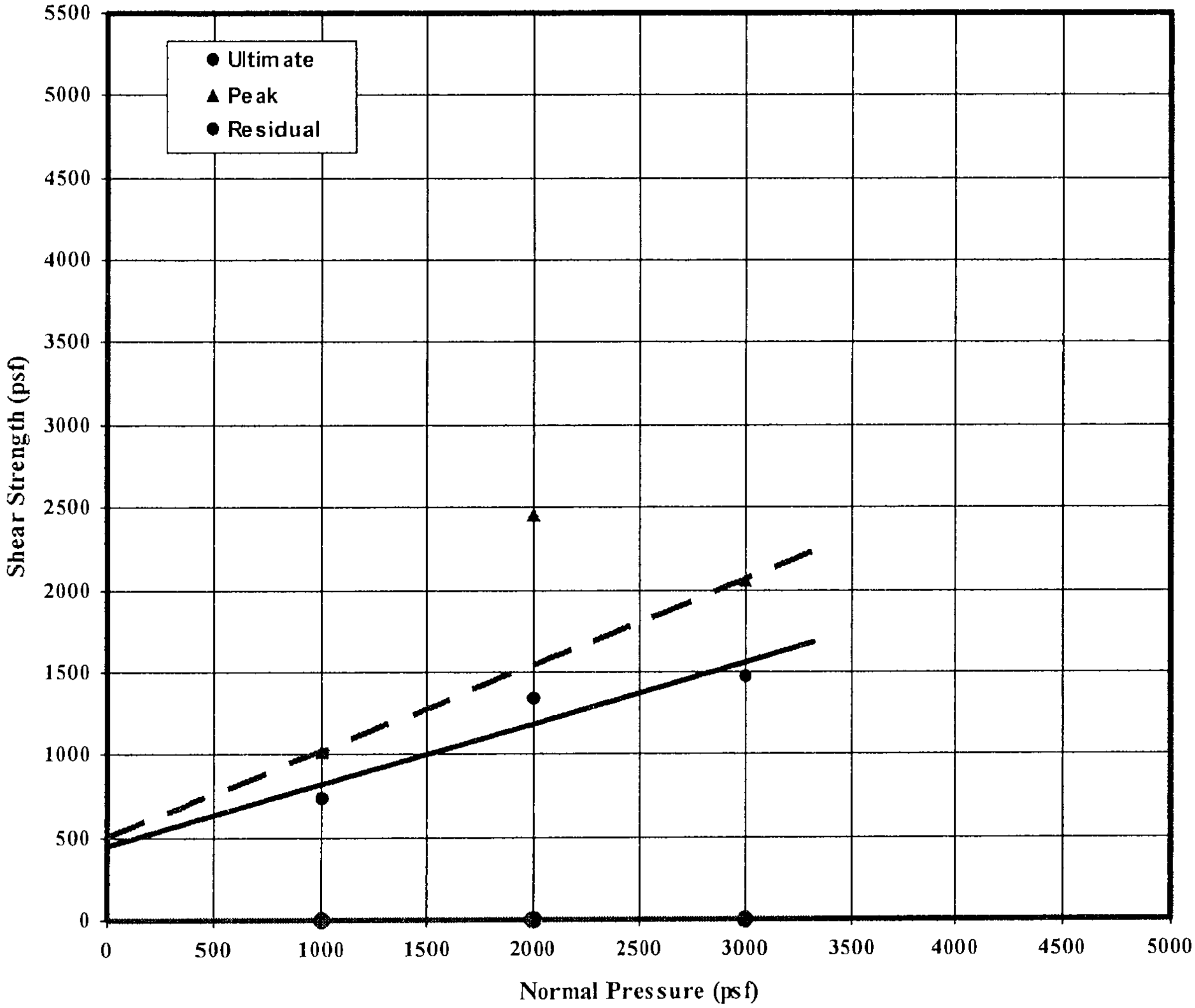
Dry Density:	109.9 pcf
Moisture:	22.8 %

GEOLABS-WESTLAKE VILLAGE

PLATE S-B5.42

SHEAR TEST RESULTS

Undisturbed Sample

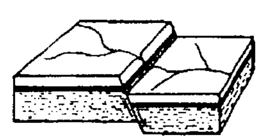


	Friction Angle	Cohesion
Ultimate Shear Strength:	20 deg	450 psf
Peak Shear Strength:	27 deg	500 psf
Residual Shear Strength:		

Displacement Rate: 0.01 in/min

Dry Density:	108.1 pcf
Moisture:	22.2 %

Sample Location: B6
Sample Depth: 25 ft.
Geologic Unit: Saugus Formation
Material: clayey SILTSTONE and silty SANDSTONE

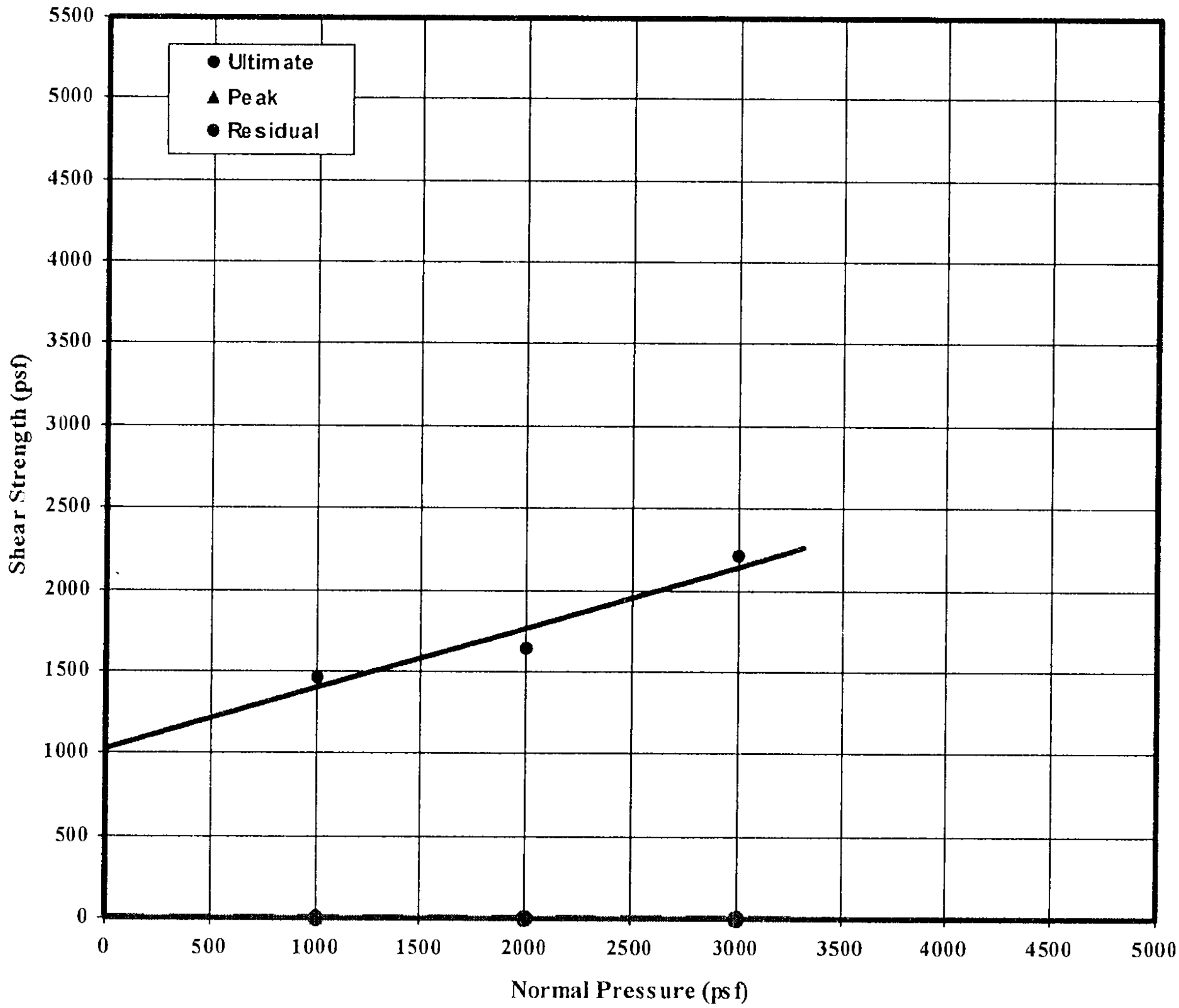


GEOLABS-WESTLAKE VILLAGE

PLATE S-B6.25

SHEAR TEST RESULTS

Undisturbed Sample



	Friction Angle	Cohesion
Ultimate Shear Strength:	20 deg	1025 psf
Peak Shear Strength:		
Residual Shear Strength:		

Displacement Rate: 0.01 in/min

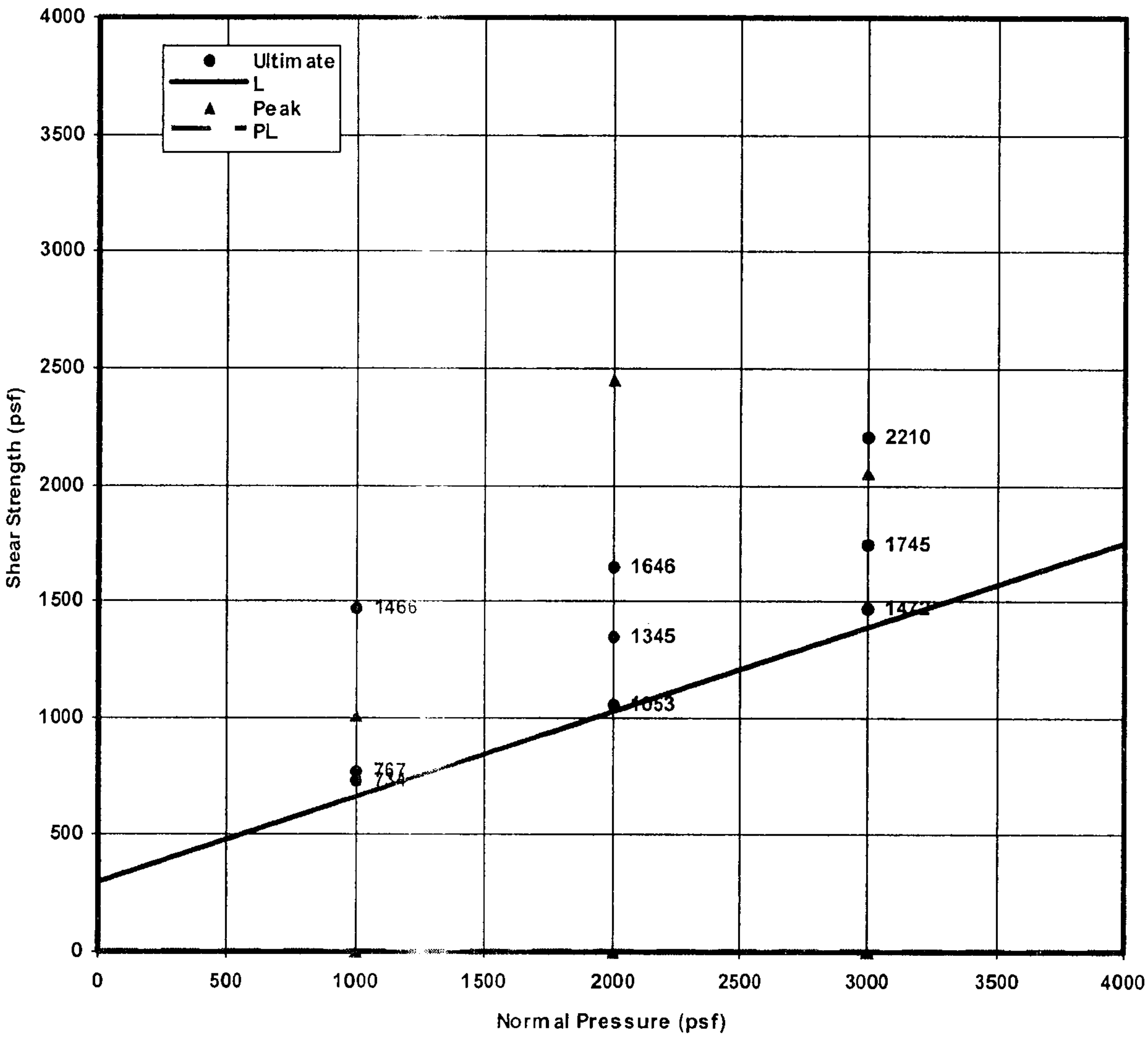
Sample Location: B6
Sample Depth: 50 ft.
Geologic Unit: Saugus Formation
Material: clayey SILTSTONE

Dry Density: 106.3 pcf
Moisture: 19 %

GEOLABS-WESTLAKE VILLAGE

PLATE S-B6.50

SHEAR TEST RESULTS



	Friction Angle	Cohesion
Ultimate Shear Strength:	20 deg	300 psf

Material: TQs

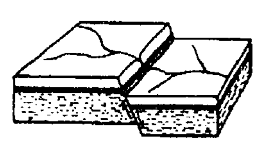
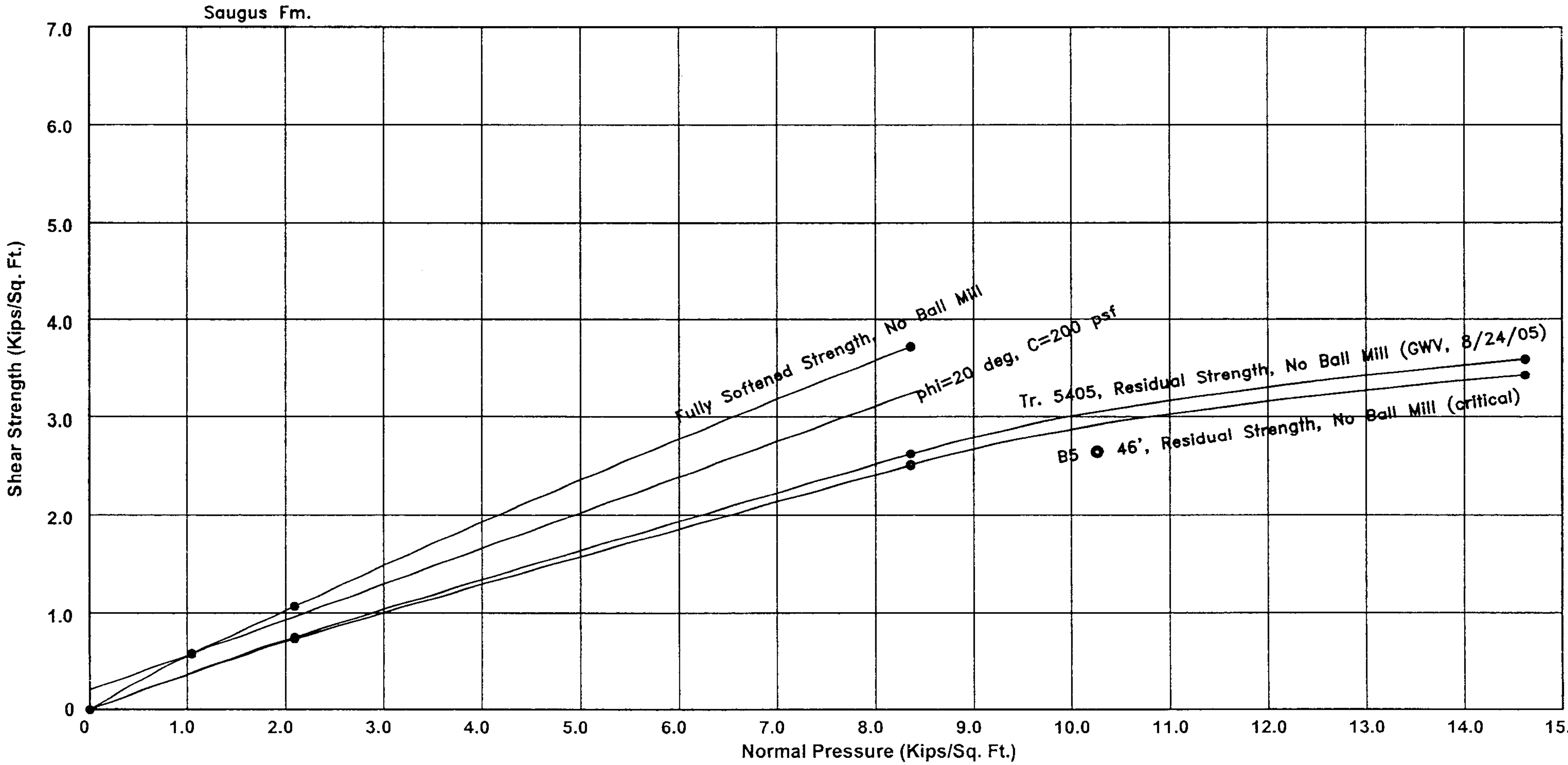


PLATE S-TQs

ESTIMATED SHEAR STRENGTH DIAGRAM



Emperical Correlation by Stark and McCone (2005)

Project Everett Terrace
Excavation B5
Depth 46 ft

Geolabs – Westlake Village
GEOLOGY AND SOIL ENGINEERING

DATE 11/05 BY
SCALE NTS W.O. 8993

PLATE

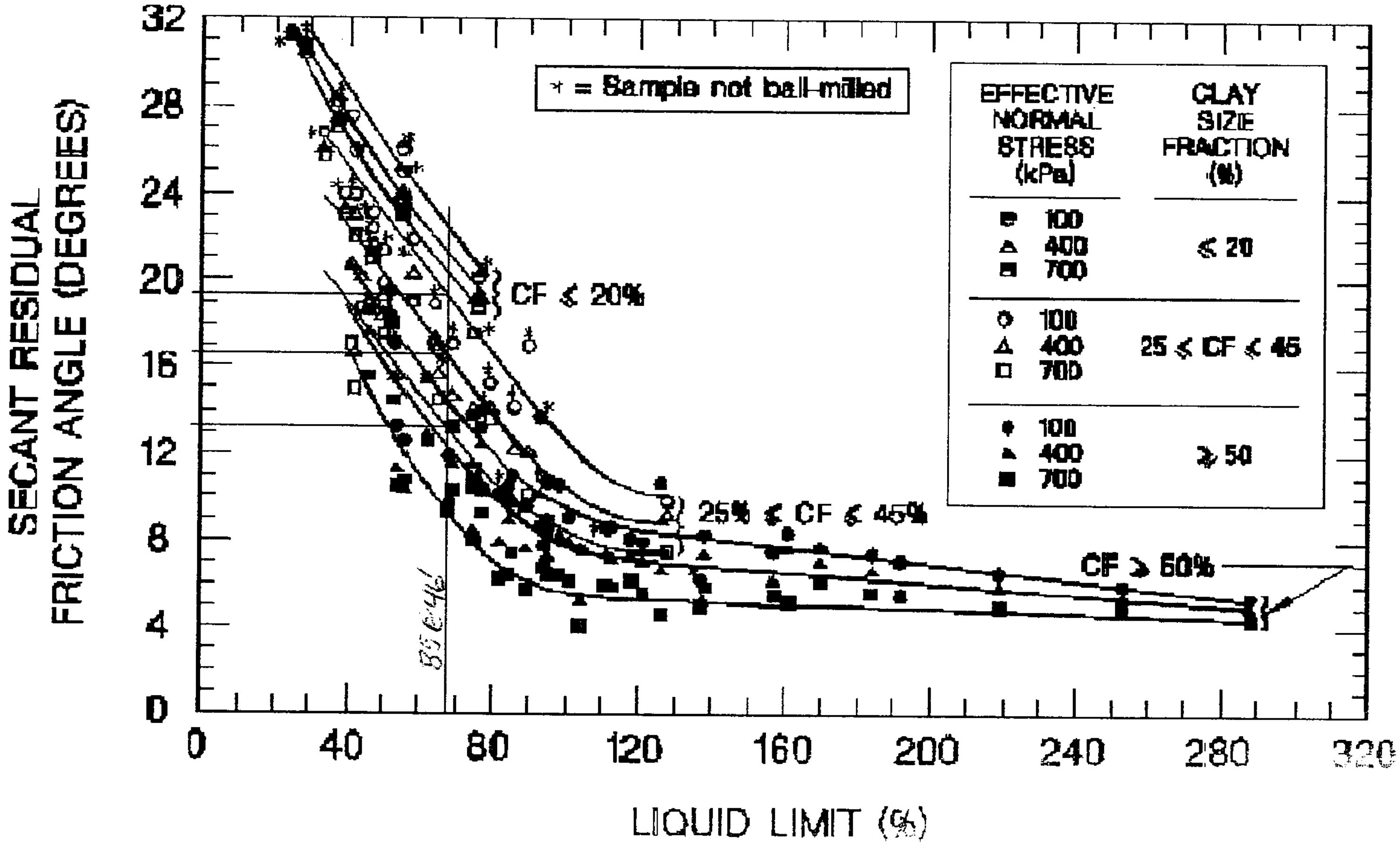


Fig. 1. Revised Secant Friction Angle Relationships with Liquid Limit, Clay-size Fraction, and Effective Normal Stress

	Geolabs – Westlake Village GEOLOGY AND SOIL ENGINEERING
	DATE 11/05 BY SCALE NTS W.O. 8963

PLATE

M. J. Schiff & Associates, Inc.
Consulting Corrosion Engineers - Since 1959
431 W. Baseline Road
Claremont, CA 91711
Phone: (909) 626-0967 Fax: (909) 626-3316
E-mail lab@mjschiff.com
website: mjschiff.com

Table 1 - Laboratory Tests on Soil Samples

Everett Terrace
Your #8953, MJS&A #04-1057LAB
5-Aug-04

Sample ID			B1 @ 0-3'
Resistivity		**Units**	
as-received		ohm-cm	52,000
saturated		ohm-cm	5,000
pH			7.3
Electrical Conductivity		mS/cm	0.11
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	64
magnesium	Mg^{2+}	mg/kg	27
sodium	Na^{1+}	mg/kg	5
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	342
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	na
nitrate	NO_3^{1-}	mg/kg	na
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

APPENDIX B

SEISMICITY ANALYSES

****
*   *
*   U B C S E I S   *
*   *
*   Version 1.00   *
*   *
****

COMPUTATION OF 1997
UNIFORM BUILDING CODE
SEISMIC DESIGN PARAMETERS

JOB NUMBER: 8953   DATE: 11-14-2005

JOB NAME: Everett Terrace

FAULT-DATA-FILE NAME: CDMGUBCR.DAT

SITE COORDINATES:
SITE LATITUDE: 34.2880
SITE LONGITUDE: 118.8821

UBC SEISMIC ZONE: 0.4

UBC SOIL PROFILE TYPE: SD

NEAREST TYPE A FAULT:
NAME: SAN ANDREAS - 1857 Rupture
DISTANCE: 54.8 km

NEAREST TYPE B FAULT:
NAME: SIMI-SANTA ROSA
DISTANCE: 0.0 km

NEAREST TYPE C FAULT:
NAME: [illegible]
DISTANCE: 99999.0 km

SELECTED UBC SEISMIC COEFFICIENTS:
Na: 1.3
Nv: 1.6
Ca: 0.57
Cv: 1.02
Ts: 0.716
To: 0.143

****
* CAUTION: The digitized data points used to model faults are *
* limited in number and have been digitized from small- *
* scale maps (e.g., 1:750,000 scale). Consequently, *
* the estimated fault-site-distances may be in error by *
* several kilometers. Therefore, it is important that *
* the distances be carefully checked for accuracy and *
* adjusted as needed, before they are used in design. *
****

SUMMARY OF FAULT PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
SIMI-SANTA ROSA	0.0	B	6.7	1.00	DS
OAK RIDGE (Onshore)	5.8	B	6.9	4.00	DS
SANTA SUSANA	13.2	B	6.6	5.00	DS
SAN CAYETANO	13.6	B	6.8	6.00	DS
HOLSER	16.9	B	6.5	0.40	DS
MALIBU COAST	23.8	B	6.7	0.30	DS
VENTURA - PITAS POINT	24.6	B	6.8	1.00	DS
ANACAPA-DUME	27.4	B	7.3	3.00	DS
SANTA YNEZ (East)	30.5	B	7.0	2.00	SS
SAN GABRIEL	32.7	B	7.0	1.00	SS
M.RIDGE-ARROYO PARIDA-SANTA ANA	33.0	B	6.7	0.40	DS
SANTA MONICA	35.2	B	6.6	1.00	DS
SIERRA MADRE (San Fernando)	37.1	B	6.7	2.00	DS
RED MOUNTAIN	38.7	B	6.8	2.00	DS
VERDUGO	42.5	B	6.7	0.50	DS
PALOS VERDES	46.2	B	7.1	3.00	SS
HOLLYWOOD	46.3	B	6.5	1.00	DS
NEWPORT-INGLEWOOD (L.A.Basin)	53.6	B	6.9	1.00	SS
SIERRA MADRE (Central)	53.8	B	7.0	3.00	DS
SAN ANDREAS - 1857 Rupture	54.8	A	7.8	34.00	SS
BIG PINE	56.8	B	6.7	0.80	SS
GARLOCK (West)	60.3	A	7.1	6.00	SS
PLEITO THRUST	62.2	B	6.8	2.00	DS
RAYMOND	62.4	B	6.5	0.50	DS
SANTA CRUZ ISLAND	67.0	B	6.8	1.00	DS
SANTA YNEZ (West)	72.4	B	6.9	2.00	SS
CLAMSHELL-SAWPIT	75.6	B	6.5	0.50	DS
WHITE WOLF	85.1	B	7.2	2.00	DS
ELSINORE-WHITTIER	86.2	B	6.8	2.50	SS
SAN JOSE	94.2	B	6.5	0.50	DS
SANTA ROSA ISLAND	98.6	B	6.9	1.00	DS
CUCAMONGA	102.1	A	7.0	5.00	DS
CHINO-CENTRAL AVE. (Elsinore)	105.3	B	6.7	1.00	DS
LOS ALAMOS-W. BASELINE	115.2	B	6.8	0.70	DS
NEWPORT-INGLEWOOD (Offshore)	117.9	B	6.9	1.50	SS
ELSINORE-GLEN IVY	124.2	B	6.8	5.00	SS
SAN ANDREAS - Southern	124.6	A	7.4	24.00	SS
SAN JACINTO-SAN BERNARDINO	126.3	B	6.7	12.00	SS
CLEGHORN	130.3	B	6.5	3.00	SS
LIONS HEAD	132.4	B	6.6	0.02	DS
GARLOCK (East)	136.1	A	7.3	7.00	SS
SAN JUAN	137.7	B	7.0	1.00	SS
SAN LUIS RANGE (S. Margin)	139.2	B	7.0	0.20	DS
CORONADO BANK	143.3	B	7.4	3.00	SS
LENWOOD-LOCKHART-OLD WOMAN SPRGS	146.1	B	7.3	0.60	SS
NORTH FRONTAL FAULT ZONE (West)	147.1	B	7.0	1.00	DS

SUMMARY OF FAULT PARAMETERS

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
CASMALIA (Orcutt Frontal Fault)	149.0	B	6.5	0.25	DS
HELENDALE - S. LOCKHARDT	150.7	B	7.1	0.60	SS
SAN JACINTO-SAN JACINTO VALLEY	154.1	B	6.9	12.00	SS
ELSINORE-TEMECULA	158.1	B	6.8	5.00	SS
So. SIERRA NEVADA	161.4	B	7.1	0.10	DS
LOS OSOS	168.4	B	6.8	0.50	DS
GRAVEL HILLS - HARPER LAKE	169.7	B	6.9	0.60	SS
HOSGRI	178.2	B	7.3	2.50	SS
LITTLE LAKE	185.3	B	6.7	0.70	SS
BLACKWATER	185.5	B	6.9	0.60	SS
ROSE CANYON	185.8	B	6.9	1.50	SS
RINCONADA	187.6	B	7.3	1.00	SS
SAN JACINTO-ANZA	190.5	A	7.2	12.00	SS
NORTH FRONTAL FAULT ZONE (East)	190.6	B	6.7	0.50	DS
LANDERS	195.3	B	7.3	0.60	SS
CALICO - HIDALGO	196.5	B	7.1	0.60	SS
ELSINORE-JULIAN	199.4	A	7.1	5.00	SS
PINTO MOUNTAIN	200.0	B	7.0	2.50	SS
JOHNSON VALLEY (Northern)	203.2	B	6.7	0.60	SS
TANK CANYON	211.6	B	6.5	1.00	DS
EMERSON So. - COPPER MTN.	217.1	B	6.9	0.60	SS
OWENS VALLEY	227.0	B	7.6	1.50	SS
PISGAH-BULLION MTN.-MESQUITE LK	227.9	B	7.1	0.60	SS
BURNT MTN.	228.3	B	6.5	0.60	SS
PANAMINT VALLEY	229.2	B	7.2	2.50	SS
EUREKA PEAK	229.3	B	6.5	0.60	SS
OWL LAKE	235.4	B	6.5	2.00	SS
SAN JACINTO-COYOTE CREEK	236.9	B	6.8	4.00	SS
EARTHQUAKE VALLEY	244.5	B	6.5	2.00	SS
SAN ANDREAS (Creeping)	245.1	B	5.0	34.00	SS
INDEPENDENCE	256.6	B	6.9	0.20	DS
DEATH VALLEY (South)	268.5	B	6.9	4.00	SS
DEATH VALLEY (Graben)	274.0	B	6.9	4.00	DS
ELSINORE-COYOTE MOUNTAIN	274.3	B	6.8	4.00	SS
SAN JACINTO - BORREGO	275.0	B	6.6	4.00	SS
HUNTER MTN. - SALINE VALLEY	277.0	B	7.0	2.50	SS
BIRCH CREEK	303.1	B	6.5	0.70	DS
SUPERSTITION MTN. (San Jacinto)	307.7	B	6.6	5.00	SS
BRAWLEY SEISMIC ZONE	309.9	B	6.5	25.00	SS
DEATH VALLEY (Northern)	311.3	A	7.2	5.00	SS
ELMORE RANCH	311.4	B	6.6	1.00	SS
SUPERSTITION HILLS (San Jacinto)	313.5	B	6.6	4.00	SS
WHITE MOUNTAINS	313.5	B	7.1	1.00	SS
ELSINORE-LAGUNA SALADA	325.8	B	7.0	3.50	SS
ROUND VALLEY (E. of S.N.Mtns.)	330.0	B	6.8	1.00	DS
ORTIGALITA	331.7	B	6.9	1.00	SS

SUMMARY OF FAULT PARAMETERS

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
CALAVERAS (So.of Calaveras Res)	334.7	B	6.2	15.00	SS
MONTEREY BAY - TULARCITOS	335.4	B	7.1	0.50	DS
DEEP SPRINGS	335.8	B	6.6	0.80	DS
PALO COLORADO - SUR	336.1	B	7.0	3.00	SS
IMPERIAL	340.4	A	7.0	20.00	SS
FISH SLOUGH	343.8	B	6.6	0.20	DS
QUIEN SABE	348.4	B	6.5	1.00	SS
HILTON CREEK	353.2	B	6.7	2.50	DS
DEATH VALLEY (N. of Cucamongo)	354.3	A	7.0	5.00	SS
ZAYANTE-VERGELES	365.8	B	6.8	0.10	SS
SAN ANDREAS (1906)	371.0	A	7.9	24.00	SS
SARGENT	371.5	B	6.8	3.00	SS
HARTLEY SPRINGS	372.7	B	6.6	0.50	DS
MONO LAKE	406.0	B	6.6	2.50	DS
SAN GREGORIO	410.2	A	7.3	5.00	SS
MONTE VISTA - SHANNON	421.0	B	6.5	0.40	DS
HAYWARD (SE Extension)	422.1	B	6.5	3.00	SS
GREENVILLE	423.6	B	6.9	2.00	SS
ROBINSON CREEK	435.1	B	6.5	0.50	DS
CALAVERAS (No.of Calaveras Res)	442.1	B	6.8	6.00	SS
HAYWARD (Total Length)	442.1	A	7.1	9.00	SS
ANTELOPE VALLEY	472.6	B	6.7	0.80	DS
CONCORD - GREEN VALLEY	490.9	B	6.9	6.00	SS
GENOA	494.0	B	6.9	1.00	DS
RODGERS CREEK	528.6	A	7.0	9.00	SS
WEST NAPA	530.3	B	6.5	1.00	SS
POINT REYES	545.0	B	6.8	0.30	DS
HUNTING CREEK - BERRYESSA	554.3	B	6.9	6.00	SS
MAACAMA (South)	591.5	B	6.9	9.00	SS
COLLAYOMI	609.3	B	6.5	0.60	SS
BARTLETT SPRINGS	614.4	A	7.1	6.00	SS
MAACAMA (Central)	632.7	A	7.1	9.00	SS
MAACAMA (North)	692.6	A	7.1	9.00	SS
ROUND VALLEY (N. S.F.Bay)	700.9	B	6.8	6.00	SS
BATTLE CREEK	737.0	B	6.5	0.50	DS
LAKE MOUNTAIN	758.8	B	6.7	6.00	SS
GARBERVILLE-BRICELAND	774.7	B	6.9	9.00	SS
MENDOCINO FAULT ZONE	829.1	A	7.4	35.00	DS
LITTLE SALMON (Onshore)	838.2	A	7.0	5.00	DS
CASCADIA SUBDUCTION ZONE	841.6	A	8.3	35.00	DS
MAD RIVER	842.5	B	7.1	0.70	DS
McKINLEYVILLE	852.6	B	7.0	0.60	DS
FICKLE HILL	854.3	B	6.9	0.60	DS
TRINIDAD	854.6	B	7.3	2.50	DS
TABLE BLUFF	858.3	B	7.0	0.60	DS
LITTLE SALMON (Offshore)	871.8	B	7.1	1.00	DS
BIG LAGOON - BALD MTN.FLT.ZONE	891.8	B	7.3	0.50	DS

MOORPARK 7.5 MINUTE QUADRANGLE AND PORTIONS OF ADJACENT QUADRANGLES

10% EXCEEDANCE IN 50 YEARS PEAK GROUND ACCELERATION (g)

1998

ALLUVIUM CONDITIONS

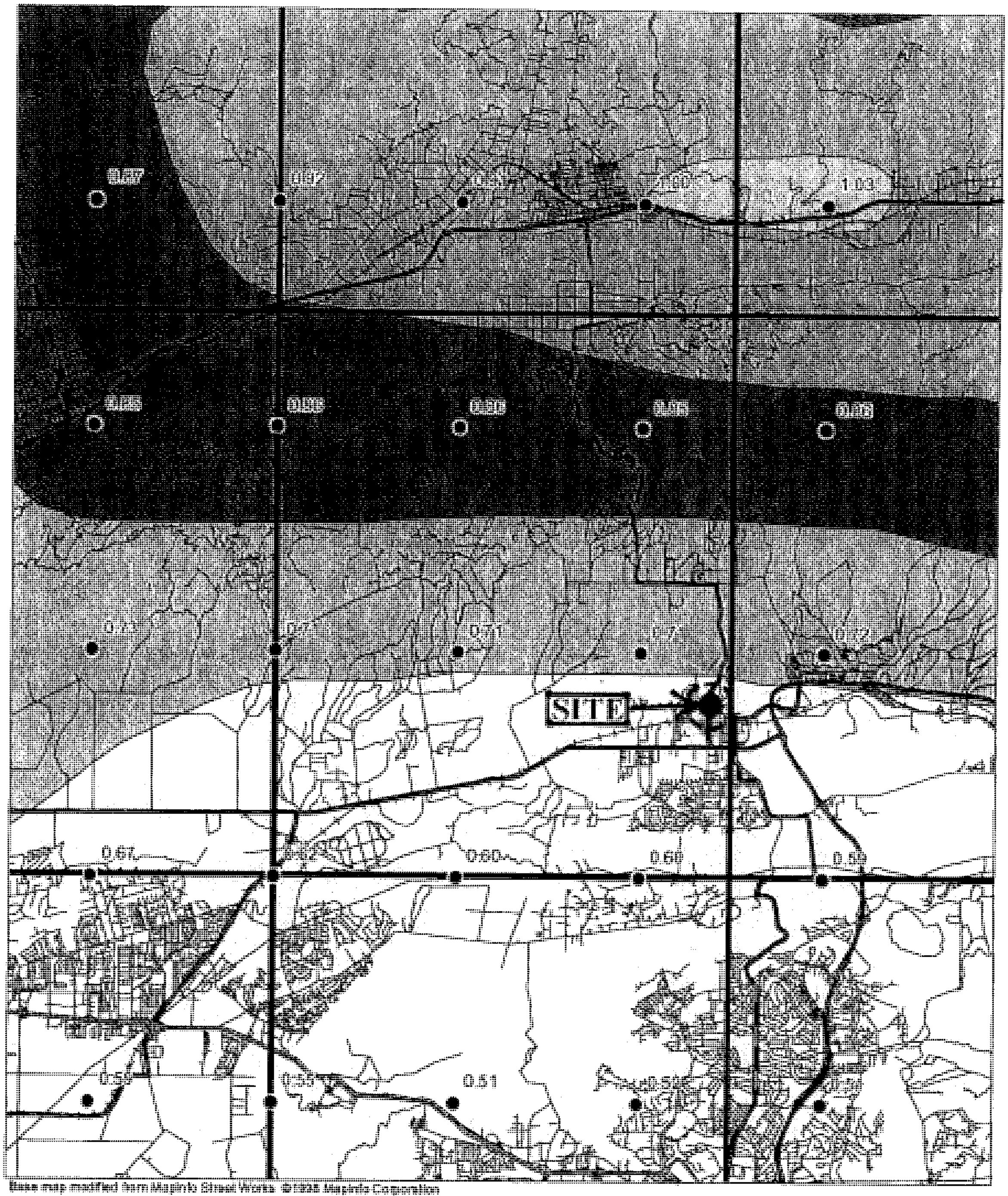


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Figure 3.3

MOORPARK 7.5 MINUTE QUADRANGLE AND PORTIONS OF ADJACENT QUADRANGLES

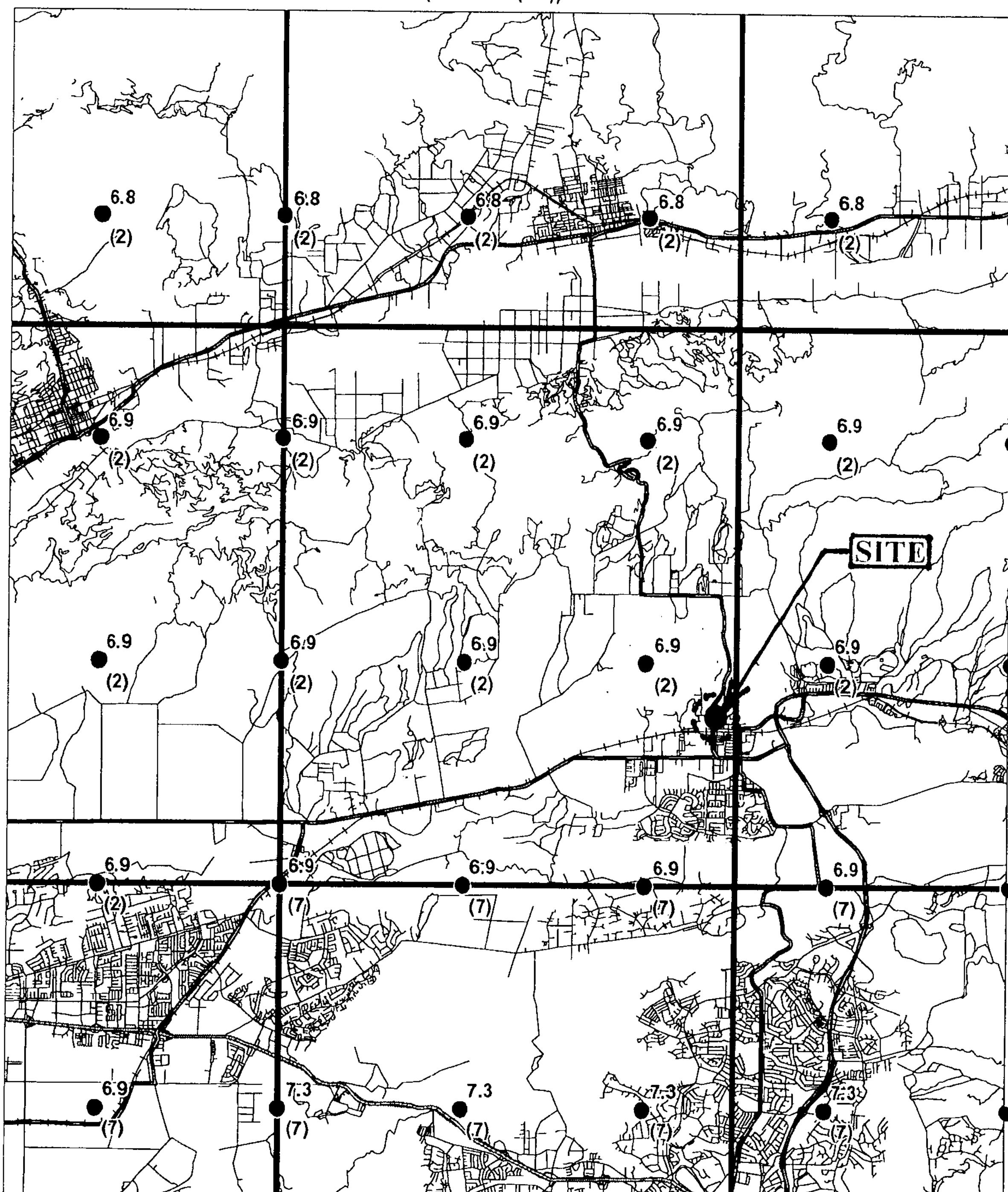
10% EXCEEDANCE IN 50 YEARS PEAK GROUND ACCELERATION

1998

PREDOMINANT EARTHQUAKE

Magnitude (Mw)

(Distance (km))



Base map modified from MapInfo StreetWorks ©1998 MapInfo Corporation

0 2.5 5

Kilometers

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Figure 3.4

APPENDIX C

LIQUEFACTION / SEISMIC SETTLEMENT ANALYSES

APPENDIX C
LIQUEFACTION AND SEISMIC SETTLEMENT ANALYSIS

This geotechnical investigation included analysis of the liquefaction potential and potential seismically induced settlement at the subject site. The liquefaction analysis addressed the alluvium below the shallow groundwater. The analysis of seismic settlement encompassed both saturated and unsaturated soils.

Field Investigation

These analyses used data retrieved from Standard Penetration Tests (SPT) in borings drilled using a hollow-stem auger and from the Cone Penetrometer Test (CPT) soundings. Samples were driven with a 140 lb. Cathead and winch hammer lifted 30 inches. The estimated efficiency of the hammer is approximately 60 percent. Drilling rod was used to allow the hammer to remain above the auger. The boring diameter was approximately 6 inches (outer diameter). The samplers consisted of both a SPT split spoon sampler and a lined Modified California split spoon sampler (2.375 inch i.d.). The borings for this investigation used water and drilling mud to prohibit soil from sluicing up the auger.

The CPT rig used during the field investigation was a 23-ton truck-mounted rig provided by Holguin, Fahan & Associates, Inc. The cone tip has a cross-sectional area of 10 square centimeters. The CPT is capable of obtaining tip pressure and side friction data at 2 inch (0.05 meter) intervals.

Data Analyses

The data obtained from the CPT and SPT tests were processed using the procedures proposed from the 1996 NCEER (Youd, 1997) and 1998 NCEER/NSF Workshops (Youd, 2001), and the SCEC implementation document (Martin, 1999). The analyses were performed using procedures programmed in the computer using Microsoft Visual Basic in conjunction with a Microsoft Excel spreadsheet.

SPT Analysis

The data from the SPT tests were processed according to the procedures proposed by NCEER, 2001. The field blowcounts were corrected for overburden, hammer energy, rod length, percent fines, and sampler liner. Tests performed using the lined California sampler (with 3 inch outer diameter and 2.37 inch inner diameter) were converted to SPT blowcounts using the procedures proposed by Lowe and Zaccheo (Fang, 1991). The cyclic resistance of the soils is compared to the cyclic stress ratio. Ratios less than 1.3 are considered to have a potential for liquefaction. The following correlations were used in these analyses.

Cyclic Stress Ratio, CSR:

$$CSR = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

Stress Reduction Coeff, r_d:

$$r_d = 1.0 - 0.00765z \quad \text{for} \quad z \le 9.15m$$

$$r_d = 1.174 - 0.0267z \quad \text{for} \quad 9.15m < z \le 23m$$

Cyclic Resistance Ratio, $CRR_{7.5}$:

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10\times(N_1)_{60}+45]^2} - \frac{1}{200}$$

$$for: (N_1)_{60} < 30$$

Fines Content Correction: $(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$

Where:	$\alpha = 0$	for	$FC \leq 5\%$
	$\alpha = e^{(1.76-(190/FC^2))}$	for	$5\% < FC < 35\%$
	$\alpha = 5.0$	for	$FC \geq 35\%$
	$\beta = 1.0$	for	$FC \leq 5\%$
	$\beta = [0.99 + (FC^{1.5}/1000)]$	for	$5\% < FC < 35\%$
	$\beta = 1.2$	for	$FC \geq 35\%$
	FC = Fines Content		

Corrections to SPT N value: $(N_1)_{60} = N_{field} C_n C_e C_b C_r C_s$

Where: $C_n = 2.2/(1.2 + \sigma'_{vo}/P_a)$ for overburden normalization

Other Correction factors per Table 2 (Youd, 2001)

Liquefaction Safety Factor: $FS = (CRR_{7.5}/CSR)MSF$

Where: MSF is Magnitude Scaling Factor (Revised Idriss)

$$MSF = 10^{2.24}/M_w^{\ 2.56}$$

<u>CPT Analysis</u>

The reduction of CPT data consisted of interpreting the soil behavior types encountered and assigning stratagraphic layers to the different soils. The depth ranges for the layers were assigned based upon material type differences such as grain size distribution and penetration resistance. Once soil layers were assigned to the sounding profile, thin sand layers were evaluated for the applicability of a correction for thin sand layers between soft clay layers. After applying a thin layer correction (if necessary), the profile data is normalized to approximately one atmosphere, evaluated, and material types and engineering characteristics are determined. The following correlations were used within these analyses.

Thin Layer
Correction, K_H: $K_H = \frac{1}{4} \times [((\frac{H}{d_c})/17) - 1.77]^2 + 1.0$

Overburden
Normalization: iterative procedure proposed by Robertson and Wride (1997).

$q_{c1N} = C_Q(q_c/P_a)$ where: $C_Q = (P_a/\sigma'_{vo})^n$ but not greater than 1.7

n varies from 0.5 to 1 by soil type.
this requires an iterative process

SPT Blowcounts: R.S. Olsen (1997) (see attached graph)

Soil Behavior Index, Ic:

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^n$$

Soil Type: Soil Behavior Chart, Robertson & Wride (1997)

Where: $Q = [(q_c - \sigma_{vo}) / P_a][(P_a / \sigma'_{vo})^n]$

And $F = [f_s / (q_c - \sigma_{vo})] \times 100\%$

Percent Fines: Robertson & Wride (1997)

Where:	if	$Ic < 2.6$	$FC(\%) = 0$
	if	$1.26 \leq Ic \leq 3.5$	$FC(\%) = 1.75 Ic^{3.25} - 3.7$
	if	$Ic > 3.5$	$FC(\%) = 100$

Grain characteristic corr. factor, K_c: $K_c = 1.0$ for $I_c \leq 1.64$

$$K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \quad \text{for } I_c > 1.64$$

Overburden corr. factor, K_σ: $K_\sigma = (\sigma_v')^{f-1}$ where: f ranges from 0.8 to 0.6 inversely to Dr

Equivalent Clean Sand Normalized Penetration Resistance:

$$(q_{c1N})_{cs} = K_c q_{c1N}$$

Cyclic Resistance Ratio, $CRR_{7.5}$:

$$CRR_{7.5} = 0.833[(q_{c1N})_{cs} / 1000] + 0.05 \quad \text{for } (q_{c1N})_{cs} < 50$$

$$CRR_{7.5} = 93[(q_{c1N})_{cs} / 1000]^3 + 0.08 \quad \text{for } 50 \leq (q_{c1N})_{cs} < 160$$

Cyclic Stress Ratio, CSR:

$$\mathrm{CSR} = 0.65(\mathrm{a}_{\max} / g)(\sigma_{vo} / \sigma'_{vo}) r_d$$

Magnitude Scaling Factor, MSF:

$$MSF = 10^{2.24} / M_w^{2.56}$$ Magnitude Scaling Factor (Revised Idriss)

Liquefaction Safety Factor: $FS = (CRR_{7.5} / CSR) MSF$

Relative Density $D_r = -98 + 66\log_{10}\frac{q_c}{[\sigma'_{vo}]^{0.5}}$ Jamiolkowski et al (1985), Units of 10kPa

$D_r = \frac{1}{2.41} * \ln[q_c/(157 * \sigma'^{0.55}_{vo})]$ Baldi et al (1986), Units of kPa

Undrained Shear Strength: $S_u = \frac{q_c - \sigma_o}{N_K}$ Robertson & Campanella (1989), N_k=15

Effective Internal Friction: $\phi' = \tan^{-1}[0.1 + 0.38\log\frac{q_c}{\sigma'_{vo}}]$ Roberston & Campanella (1983)

$\phi' = 53.881 - 27.6034e^{(-0.0147N)}$ Peck et al., (1974) after Coyle (1985)

Overconsolidation Ratio: programmed from chart Schmertmann (1978)

The results of our analyses are presented on the attached graphs. Layers of materials in which the CSR exceeds the $CRR_{7.5}$ are considered liquefiable. The data from boring B1 and CPT sounding CPT1 were compared on a plot located at the end of the CPT1 output series. In our opinion the CPT data compares well with the SPT data.

Liquefaction Induced Settlement

The analyses of the potential liquefaction induced settlement are performed using the same electronic spreadsheet used to perform the liquefaction analyses. The spreadsheet is programmed to perform the analyses proposed by Tokimatsu and Seed (1987).

For the settlement analyses, the normalized, fines corrected SPT blowcounts are compared to digitized files of chart data (SPT blowcounts vs. Volumetric strain for clean sands) provided in Tokimatsu and Seed. The fines correction to the SPT blowcounts consider a liquefied soil. Therefore, this fines correction produces smaller corrected blowcounts than the corrected blowcounts used to estimate the potential for triggering of liquefaction. This fines correction is based on the recommended procedures for implementation of Publication 117 (Martin and Lew, 1999). The spreadsheet estimates the percent volumetric strain for each layer assuming the lateral strain is minor so the volumetric strain is equivalent to settlement. The estimated settlement for each soil layer and a summation of all the soil layers below the design groundwater level are then reported.

Seismic Settlement of Dry Sands

For coarse-grained materials above the design groundwater level the potential for settlement related to ground shaking is analyzed using the methods proposed by Tokimatsu and Seed (1987). Like in the liquefaction-induced-settlement analyses, the SPT blowcounts and/or CPT derived SPT blowcounts are corrected to an equivalent blowcount for clean sand. Clayey soils with 15 percent clay or more, and soils with an Ic of 2.6 or greater are discarded from the analyses. The following equations are used in the analysis to enter into charts provided with the methodology.

Effective Shear Strain: $$\gamma_{eff} = \frac{\tau_{av}}{G_{max} \times \frac{G_{eff}}{G_{max}}}$$

Average Cyclic Shear Stress: $\tau_{av} = 0.65 \bullet \frac{a_{max}}{g} \bullet \sigma_o \bullet r_d$

Shear Modulus at Low Strain:

$$G_{max} = 1000 \bullet (K_2)_{max} \bullet (\sigma'_m)^{1/2} \text{ in psf units}$$

$$(K_2)_{max} \cong 20(N_1)^{1/3} \qquad \text{(Ohta and Goto, 1976)}$$

Using the referenced methodology, the volumetric strain is estimated for each soil layer or layer portion above the design water level. This methodology is considered applicable to dry or moist sands (unsaturated). Again the lateral strains are considered insignificant so the volumetric strain is considered as settlement. The estimated settlement for each soil layer and a summation of all the soil layers above the groundwater level are then reported by incorporation into the attached settlement graphs.

Currently the practice of soils engineering lacks accurate means or knowledge to estimate the potential seismic settlement for fine-grained soils. The use of this methodology for soils with significant amounts of fine grain sizes is believed to be conservative. This methodology is used with the understanding of its limitations and for lack of better simplified means of estimating the potential for seismic settlement.

The estimate of potential differential settlement is typically taken to be half of the total seismic settlement (Martin and Lew, 1999).

Surface Manifestations

Consideration of the potential for surface manifestations used the procedure proposed by Ishihara, 1985. The potential is considered a function of the relative density (SPT blowcounts), depth and thickness of liquefiable material, and thickness of overlying non-liquefiable material. Surface manifestations are not considered probable during a design level earthquake.

Liquefaction-Induced Lateral Spread

The potential for liquefaction-induced lateral spread was analyzed using the procedures proposed by Barlett and Youd (1995) as modified in 1999. The potential is considered a function of earthquake distance and magnitude, thickness and grain-size distribution in the liquefiable layers, and ground slope or nearness of an open face. It should be noted that this procedure was developed using a historical database of large displacement events. The database includes few points with movement magnitudes of small value, on the order that is of interest for engineering purposes. The procedure is also applicable only for earthquake sources greater than 10 kilometers from the subject site (though we have evaluated our data using smaller source distances). These two elements of the analyses are cause to suspect the output when faults are near source events and when the magnitude of movement is relatively small (a few meters). Therefore, for our purposes we have used this analyses as an indicator whether lateral-spread may be possible; however, the magnitudes of movement output from the analyses are considered suspect.

Considering the blow counts and estimated blow counts obtained during the investigation and the depth of design groundwater, lateral spreading during a design level earthquake is considered unlikely.

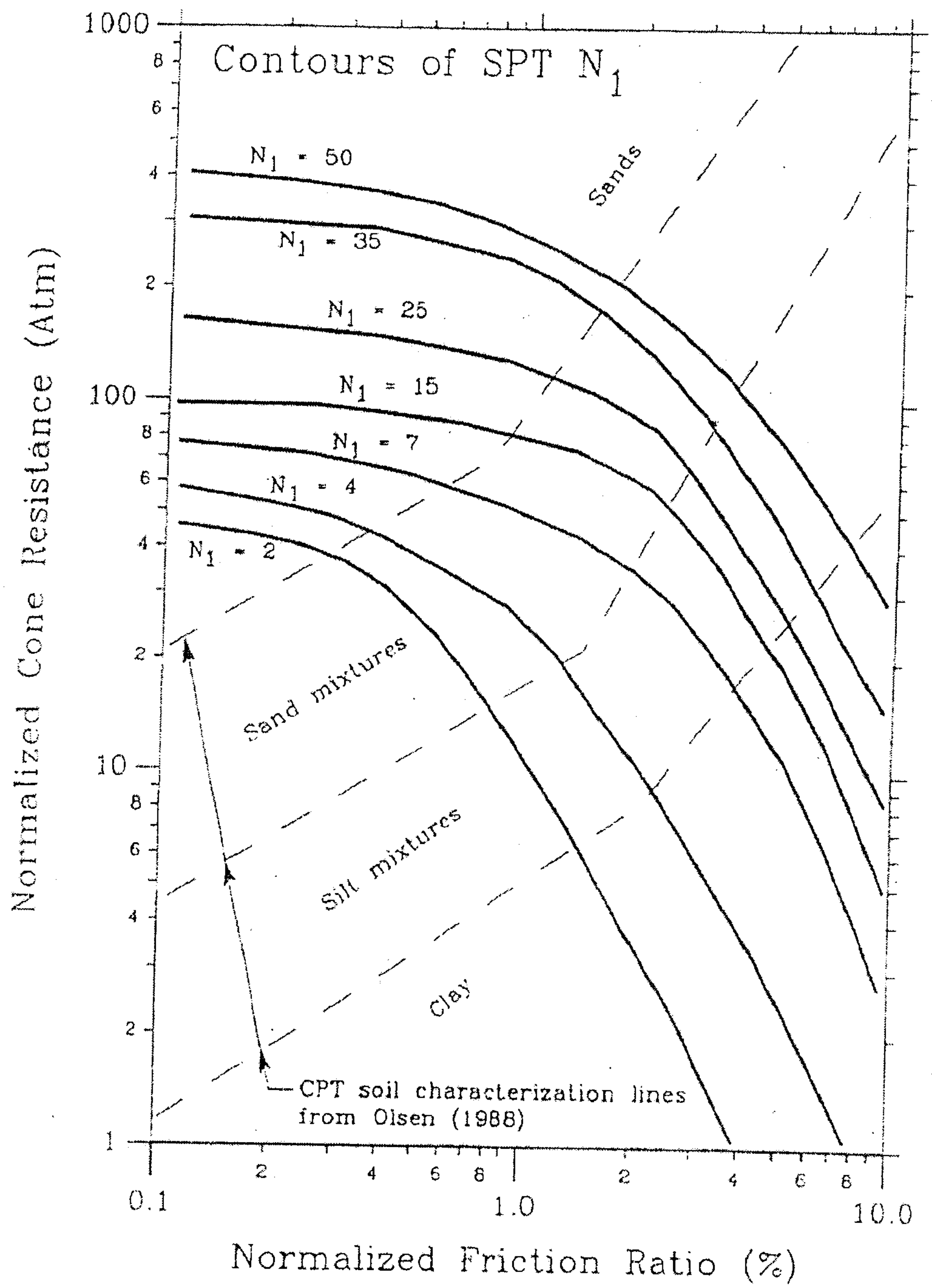

Figure 11 CPT estimation of SPT N_1 using both CPT measurements (Olsen 1994, 1988 1986, 1984).

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CPT ANALYSIS
CPT 1

GEOLABS-WESTLAKE VILLAGE

Summary of Analysis of CPT Data

CPT: CPT1
G.W. Depth: 60 ft
Design G.W. Depth: 60 ft
Removal: 0 ft

W.O.: 8953
Elev.: 0
Ic: 2.6

C:\JOBS\8953_Everest Terrace\6-29-04\CPT-01.cpd

Fill Height: 0.0 ft 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Est. Wet Den. (pcf)	Soil Behavior Type	O.B. (tsf)	Eff. O.B. (tsf)	Norm. Exp. n	Ic Range	Kc	Fines Content (%)	Phi (deg)	Dr (%)	Su (tsf)	SPT N1(60) (bpf)	SPT N1(60)cs (bpf)
1	0.82	0.49	17.6	0.13	28.7	0.8	120	Sand mixtures - silty sand to sandy silt	0.031	0.031	0.5	2.17 to 2.55	2.09	23.2	49	75.0	—	3.9	8.2
2	1.80	0.98	41.4	0.28	67.5	0.7	125	Sands - clean sand to silty sand	0.080	0.080	0.5	1.8 to 2.07	1.26	11.8	48	87.0	—	10.6	12.4
3	3.77	1.97	26.6	0.22	43.3	0.8	120	Sand mixtures - silty sand to sandy silt	0.170	0.170	0.5	2.09 to 2.29	1.60	18.0	43	52.0	—	6.1	9.7
4	4.43	0.66	46.3	0.43	75.4	0.9	125	Sands - clean sand to silty sand	0.249	0.249	0.5	1.95 to 2.04	1.28	12.6	44	66.0	—	14.1	16.3
5	6.40	1.97	28.4	0.29	46.1	1.0	120	Sand mixtures - silty sand to sandy silt	0.329	0.329	0.5	2.15 to 2.32	1.65	18.7	40	39.0	—	7.2	11.1
6	7.55	1.15	57.9	0.65	86.8	1.1	125	Sands - clean sand to silty sand	0.424	0.424	0.5	1.94 to 2.04	1.29	12.7	42	63.0	—	18.1	20.5
7	7.71	0.16	68.6	1.10	98.5	1.6	120	Sand mixtures - silty sand to sandy silt	0.465	0.465	0.5	2.06	1.38	14.5	43	68.0	—	24.8	28.3
8	8.20	0.49	66.8	0.87	94.0	1.3	125	Sands - clean sand to silty sand	0.485	0.485	0.5	1.99 to 2.05	1.31	13.2	42	66.0	—	21.3	24.1
9	9.51	1.31	68.4	1.50	91.0	2.2	120	Sand mixtures - silty sand to sandy silt	0.540	0.540	0.5	2.07 to 2.34	1.59	17.7	42	65.0	—	27.9	32.9
10	9.68	0.16	73.3	2.81	93.9	3.9	120	Silt mixtures - clayey silt to silty clay	0.584	0.584	0.5	2.35	2.11	24.3	—	—	4.9	48.1	57.6
11	10.83	1.15	90.7	2.34	112.4	2.6	120	Sand mixtures - silty sand to sandy silt	0.623	0.623	0.5	2.06 to 2.26	1.60	18.0	43	73.0	—	37.6	43.3
12	10.99	0.16	73.1	2.74	87.9	3.8	120	Silt mixtures - clayey silt to silty clay	0.663	0.663	0.5	2.36	2.15	24.8	—	—	4.8	44.6	53.9
13	11.32	0.33	51.3	2.96	66.7	5.9	125	Very stiff, fine grained*	0.678	0.678	0.5 to 1	2.53 to 2.63	3.24	34.6	—	—	3.4	50.0	64.2
14	15.26	3.94	37.2	2.65	45.3	7.3	120	Clays - silty clay to clay	0.806	0.806	1	2.68 to 2.83	4.48	43.9	—	—	2.4	49.7	64.6
15	15.91	0.66	53.1	2.28	53.5	4.4	120	Silt mixtures - clayey silt to silty clay	0.944	0.944	0.5	2.51 to 2.57	3.02	32.9	—	—	3.5	35.3	46.5
16	16.57	0.66	63.8	1.68	63.0	2.7	120	Sand mixtures - silty sand to sandy silt	0.983	0.983	0.5	2.25 to 2.44	2.13	24.4	38	48.0	—	23.1	29.9
17	17.06	0.49	50.0	1.74	48.5	3.6	120	Silt mixtures - clayey silt to silty clay	1.018	1.018	0.5	2.48 to 2.58	2.84	31.2	—	—	3.3	26.3	35.5
18	17.22	0.16	43.6	2.24	41.0	5.3	120	Clays - silty clay to clay	1.038	1.038	1	2.69	3.90	39.8	—	—	2.8	36.3	48.6
19	17.39	0.16	48.4	2.00	46.3	4.2	120	Silt mixtures - clayey silt to silty clay	1.047	1.047	0.5	2.58	3.21	34.4	—	—	3.2	30.6	41.4
20	17.72	0.33	82.5	1.70	78.4	2.1	120	Sand mixtures - silty sand to sandy silt	1.062	1.062	0.5	2.17 to 2.24	1.69	19.2	39	57.0	—	21.9	27
21	18.70	0.98	111.9	1.14	104.4	1.0	125	Sands - clean sand to silty sand	1.103	1.103	0.5	1.81 to 2.04	1.20	10.6	41	68.0	—	21.6	23.2
22	20.01	1.31	67.6	0.98	61.1	1.5	120	Sand mixtures - silty sand to sandy silt	1.173	1.173	0.5	2.12 to 2.26	1.64	18.5	38	46.0	—	13.0	17.2
23	20.51	0.49	84.1	0.97	74.3	1.2	125	Sands - clean sand to silty sand	1.228	1.228	0.5	2.04 to 2.07	1.38	14.5	39	55.0	—	15.2	18.2
24	21.65	1.15	70.6	1.04	61.3	1.6	120	Sand mixtures - silty sand to sandy silt	1.277	1.277	0.5	2.07 to 2.46	1.72	19.2	37	46.0	—	13.3	17.6
25	22.47	0.82	45.1	1.41	35.9	3.2	120	Silt mixtures - clayey silt to silty clay	1.337	1.337	0.5 to 1	2.53 to 2.69	3.22	34.4	—	—	2.9	15.4	23.1
26	22.64	0.16	45.0	2.14	31.9	4.9	120	Clays - silty clay to clay	1.366	1.366	1	2.74	4.31	42.7	—	—	2.9	28.1	38.7
27	22.80	0.16	53.9	2.08	38.2	4.0	120	Silt mixtures - clayey silt to silty clay	1.376	1.376	1	2.62	3.45	36.4	—	—	3.5	25.2	35.2
28	24.61	1.80	101.7	2.20	83.0	2.2	120	Sand mixtures - silty sand to sandy silt	1.435	1.435	0.5	2.11 to 2.32	1.68	19.1	39	58.0	—	24.0	29.2
29	25.26	0.66	171.7	2.38	136.8	1.4	125	Sands - clean sand to silty sand	1.510	1.510	0.5	1.83 to 2.01	1.20	10.6	41	79.0	—	29.5	31.3
30	25.75	0.49	80.7	1.47	63.6	1.9	120	Sand mixtures - silty sand to sandy silt	1.545	1.545	0.5	2.1 to 2.38	1.82	20.6	37	47.0	—	16.1	21.1
31	26.25	0.49	62.7	2.38	42.2	3.9	120	Silt mixtures - clayey silt to silty clay	1.574	1.574	0.5 to 1	2.5 to 2.63	3.26	34.8	—	—	4.1	26.3	36.1
32	28.38	2.13	104.1	2.54	79.1	2.6	120	Sand mixtures - silty sand to sandy silt	1.653	1.653	0.5	2.03 to 2.41	1.91	21.7	38	55.0	—	26.8	33
33	28.87	0.49	178.6	3.10	132.8	1.8	125	Sands - clean sand to silty sand	1.732	1.732	0.5	1.97 to 2.02	1.29	12.7	41	78.0	—	31.4	34.3
34	29.69	0.82	106.3	2.21	78.2	2.1	120	Sand mixtures - silty sand to sandy silt	1.772	1.772	0.5	2.12 to 2.34	1.73	19.6	38	55.0	—	22.3	27.5
35	29.86	0.16	54.4	1.64	29.2	3.1	120	Silt mixtures - clayey silt to silty clay	1.802	1.802	1	2.64	3.57	37.3	—	—	3.5	10.9	18
36	30.84	0.98	40.7	2.40	21.1	6.3	120	Clays - silty clay to clay	1.836	1.836	1	2.82 to 3.03	6.19	55.0	—	—	2.6	27.7	38.3
37	31.66	0.82	66.7	2.48	34.3	3.8	120	Silt mixtures - clayey silt to silty clay	1.891	1.891	1	2.6 to 2.69	3.62	37.6	—	—	4.3	20.3	29.4
38	32.15	0.49	95.5	1.51	67.3	1.6	120	Sand mixtures - silty sand to sandy silt	1.930	1.930	0.5	2.12 to 2.27	1.62	18.2	37	49.0	—	15.5	19.8
39	32.48	0.33	110.1	1.25	77.1	1.2	125	Sands - clean sand to silty sand	1.955	1.955	0.5	2.01 to 2.06	1.35	14.0	37	55.0	—	15.8	18.7
40	36.42	3.94	102.5	1.65	69.6	1.7	120	Sand mixtures - silty sand to sandy silt	2.083	2.083	0.5	2.07 to 2.35	1.63	18.3	37	50.0	—	16.4	20.7
41	36.75	0.33	70.7	2.99	31.0	4.4	120	Silt mixtures - clayey silt to silty clay	2.211	2.211	1	2.7 to 2.73	4.11	41.3	—	—	4.6	22.9	32.5
42	37.24	0.49	108.5	2.27	71.0	2.2	120	Sand mixtures - silty sand to sandy silt	2.236	2.236	0.5	2.11 to 2.4	1.82	20.5	36	51.0	—	21.6	27.2
43	40.52	3.28	154.5	1.70	98.8	1.1	125	Sands - clean sand to silty sand	2.353	2.353	0.5	1.82 to 2.06	1.25	11.8	38	64.0	—	20.8	22.9
44	41.01	0.49	111.6	1.29	69.5	1.2	120	Sand mixtures - silty sand to sandy silt	2.470	2.470	0.5	2.08 to 2.08	1.42	15.2	36	50.0	—	13.9	17.2
45	43.64	2.62	127.5	1.34	77.9	1.1	125	Sands - clean sand to silty sand	2.567	2.567	0.5	1.98 to 2.06	1.32	13.3	37	55.0	—	15.6	18.1
46	43.80	0.16	125.6	1.61	75.5	1.3	120	Sand mixtures - silty sand to sandy silt	2.654	2.654	0.5	2.08	1.42	15.2	36	54.0	—	16.2	19.5
47	44.62	0.82	134.6	1.52	80.4	1.2	125	Sands - clean sand to silty sand	2.685	2.685	0.5	1.98 to 2.05	1.33	13.6	37	56.0	—	16.7	19.4
48	44.95	0.33	156.8	2.61	93.1	1.7	120	Sand mixtures - silty sand to sandy silt	2.720	2.720	0.5	2.06 to 2.11	1.43	15.4	38	62.0	—	23.8	27.6
49	48.06	3.12	163.5	2.05	95.1	1.3	125	Sands - clean sand to silty sand	2.827	2.827	0.5	1.94 to 2.04	1.30	12.9	38	63.0	—	21.4	24
50	48.39	0.33	161.5	2.46	92.2	1.6	120	Sand mixtures - silty sand to sandy silt	2.935	2.935	0.5	2.06 to 2.07	1.39	14.7	37	62.0	—	22.4	25.9
51	48.72	0.33	153.0	1.99	87.1	1.3	125	Sands - clean sand to silty sand	2.955	2.955	0.5	2.03 to 2.04	1.35	13.9	37	59.0	—	19.5	22.5
52	49.70	0.98	130.0	2.67	73.5	2.2	120	Sand mixtures - silty sand to sandy silt	2.995	2.995	0.5	2.09 to 2.41	1.79	20.1	36	52.0	—	22.3	27.8

Evaluation of Soil C[illegible]racteristics Using CPT Data

CPT: CPT1
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Elev.: 0

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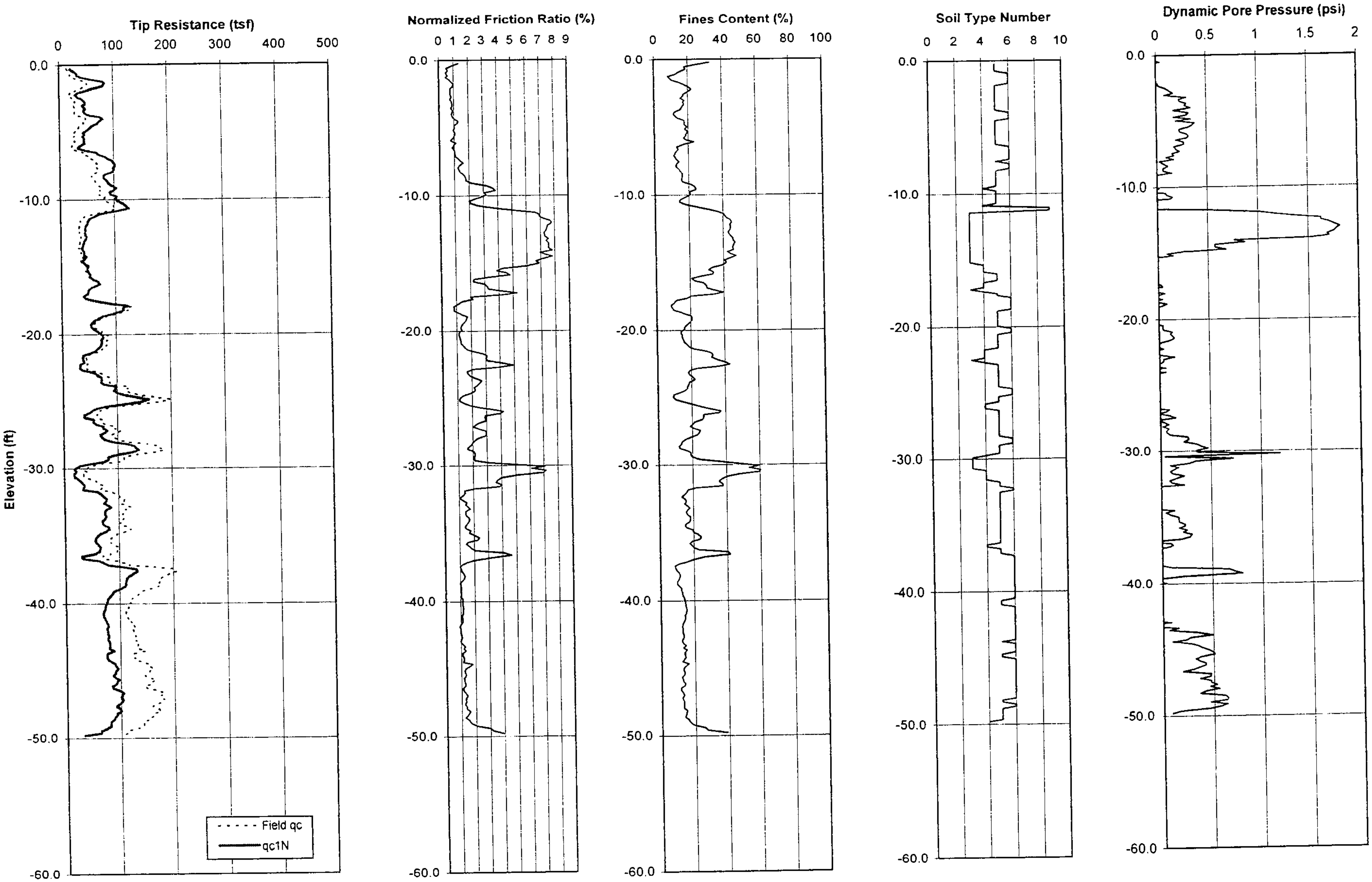

GEOLABS-WESTLAKE VILLAGE

CPT-Based Soil Behavior Type

CPT: **CPT1**
G.W. Depth: 60 ft
Ic: 2.6

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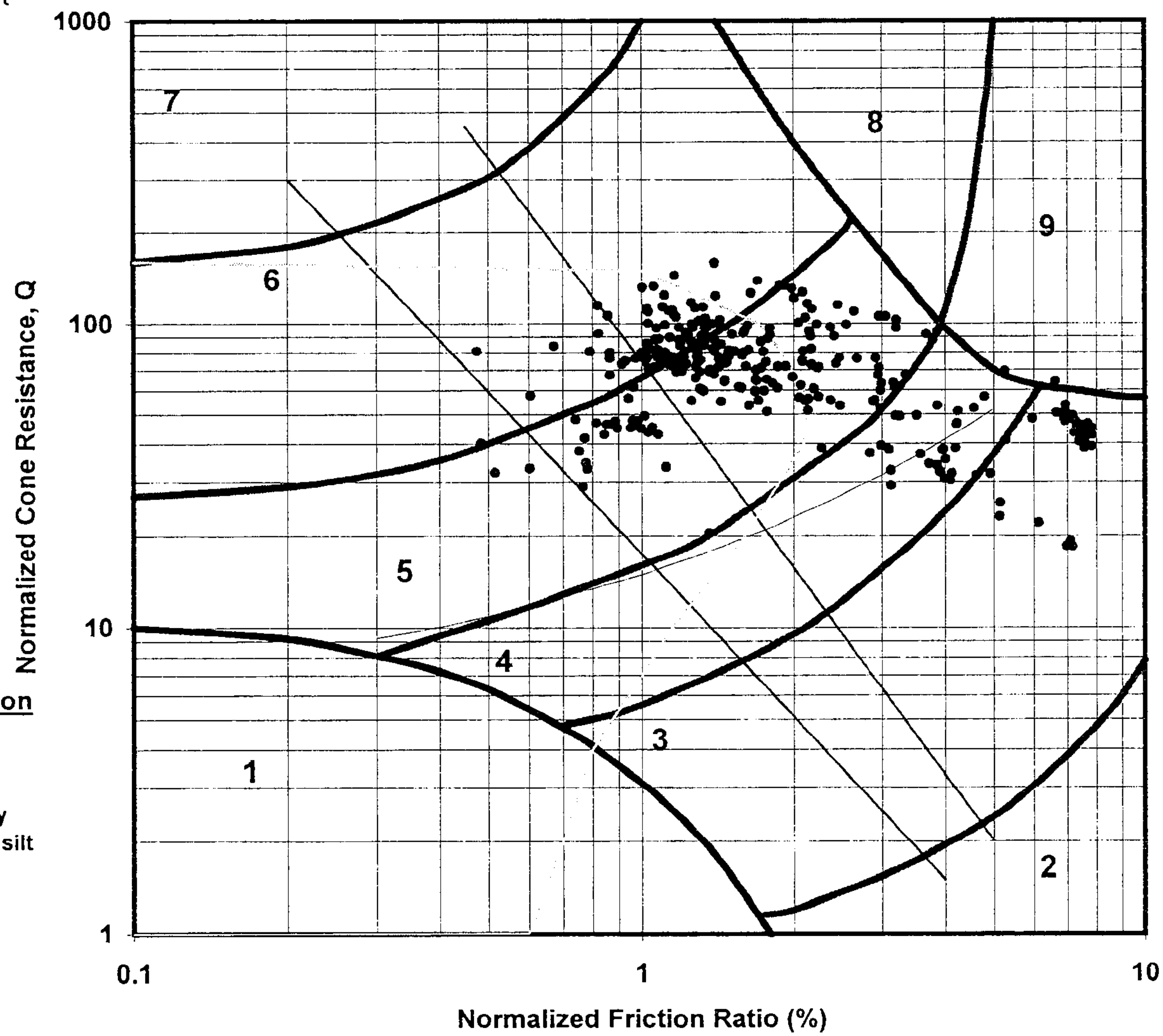
Soil Consistency Number

(Sand / Clay)

1 = very loose / very soft
2 = loose / soft
3 = medium dense / medium stiff
4 = dense / stiff
5 = very dense / very stiff
6 = — / hard



Soil Behavior Type Classification

1. Sensitive Fine Grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

*Heavily overconsolidated or cemented

GEOLABS-WESTLAKE VILLAGE

Soil Characteristics and Engineering Characteristics
Using CPT Data

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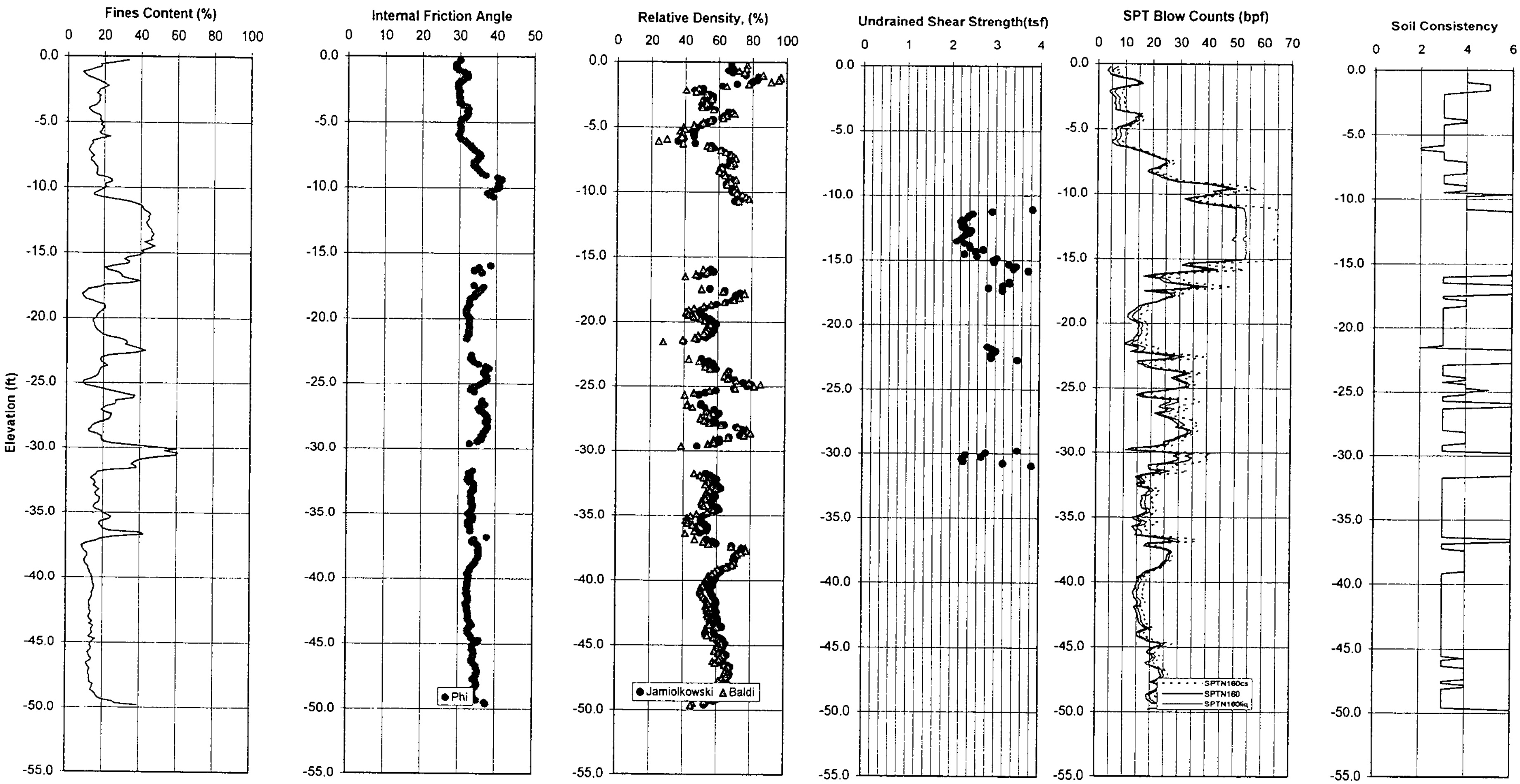
CPT: CPT1
G.W. Depth: 60 ft. Elev.: 0
Design G.W. Depth: 60 ft.



CPT ANALYSIS
CPT 1
NO DESIGN GROUNDWATER

GEOLABS-WESTLAKE VILLAGE

"Dry" Sand Seismic Settlement Using CPT Data

CPT: CPT1
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 0.88

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Fines Content (%)	Avg. Dr (%)	Spt N160cs (bpf)	Cyclic Shear Stress τav (psf)	Gmax (ksf)	γeff / (Geff/Gmax)	γeff	Vol. Strain (%)	"Dry" Settle. (in)
1	0.82	0.49	17.6	0.13	28.7	0.8	0.031	Sand mixtures - silty sand to sandy silt (5)	1.00	2.09	1	23.2	75	8.2	28	2.16E+02	1.24E-04	4.21E-04	2.70E-01	1.85E-02
2	1.80	0.98	41.4	0.28	67.5	0.7	0.080	Sands - clean sand to silty sand (6)	1.00	1.26	1	11.8	87	12.4	72	4.12E+02	1.73E-04	1.20E-03	4.65E-01	5.50E-02
3	3.77	1.97	26.6	0.22	43.3	0.8	0.170	Sand mixtures - silty sand to sandy silt (5)	1.00	1.60	1	18.0	52	9.7	152	5.77E+02	2.60E-04	5.31E-03	2.23E+00	5.27E-01
4	4.43	0.66	46.3	0.43	75.4	0.9	0.249	Sands - clean sand to silty sand (6)	0.99	1.28	1	12.6	66	16.3	222	8.30E+02	2.67E-04	1.42E-02	2.64E+00	2.08E-01
5	6.40	1.97	28.4	0.29	46.1	1.0	0.329	Sand mixtures - silty sand to sandy silt (5)	0.99	1.65	1	18.7	39	11.1	292	8.61E+02	3.39E-04	3.69E-02	3.56E-01	8.40E-02
6	7.55	1.15	57.9	0.65	86.8	1.1	0.424	Sands - clean sand to silty sand (6)	0.99	1.29	1	12.7	63	20.5	375	1.18E+03	3.19E-04	5.99E-03	9.65E-01	1.33E-01
7	7.71	0.16	68.6	1.10	98.5	1.6	0.465	Sand mixtures - silty sand to sandy silt (5)	0.98	1.38	1	14.5	68	28.3	410	1.37E+03	2.98E-04	2.29E-03	2.62E-01	5.15E-03
8	8.20	0.49	66.8	0.87	94.0	1.3	0.485	Sands - clean sand to silty sand (6)	0.98	1.31	1	13.2	66	24.1	428	1.33E+03	3.21E-04	3.18E-03	4.66E-01	2.75E-02
9	9.51	1.31	68.4	1.50	91.0	2.2	0.540	Sand mixtures - silty sand to sandy silt (5)	0.98	1.59	1	17.7	65	32.9	475	1.55E+03	3.09E-04	2.31E-03	2.83E-01	4.46E-02
10	9.68	0.16	73.3	2.81	93.9	3.9	0.584	Silt mixtures - clayey silt to silty clay (4)	0.98	2.11	1	24.3	---	57.6	513	2.79E+03	1.84E-04	3.01E-04	1.62E-02	3.19E-04
11	10.83	1.15	90.7	2.34	112.4	2.6	0.623	Sand mixtures - silty sand to sandy silt (5)	0.98	1.60	1	18.0	73	43.3	547	1.83E+03	2.99E-04	1.49E-03	9.12E-02	1.26E-02
12	10.99	0.16	73.1	2.74	87.9	3.8	0.663	Silt mixtures - clayey silt to silty clay (4)	0.98	2.15	1	24.8	---	53.9	581	2.90E+03	2.00E-04	3.39E-04	1.83E-02	3.60E-04
15	15.91	0.66	53.1	2.28	53.5	4.4	0.944	Silt mixtures - clayey silt to silty clay (4)	0.97	3.02	1	32.9	---	46.5	819	3.30E+03	2.49E-04	4.44E-04	2.39E-02	1.89E-03
16	16.57	0.66	63.8	1.68	63.0	2.7	0.983	Sand mixtures - silty sand to sandy silt (5)	0.97	2.13	1	24.4	48	29.9	852	2.10E+03	4.08E-04	2.21E-03	2.61E-01	2.05E-02
17	17.06	0.49	50.0	1.74	48.5	3.6	1.018	Silt mixtures - clayey silt to silty clay (4)	0.96	2.84	1	31.2	---	35.5	881	3.13E+03	2.82E-04	5.26E-04	4.17E-02	2.47E-03
19	17.39	0.16	48.4	2.00	46.3	4.2	1.047	Silt mixtures - clayey silt to silty clay (4)	0.96	3.21	0.99	34.4	---	41.4	905	3.35E+03	2.70E-04	4.82E-04	2.61E-02	5.13E-04
20	17.72	0.33	82.5	1.70	78.4	2.1	1.062	Sand mixtures - silty sand to sandy silt (5)	0.96	1.69	0.99	19.2	57	27.0	917	2.10E+03	4.39E-04	2.66E-03	3.51E-01	1.38E-02
21	18.70	0.98	111.9	1.14	104.4	1.0	1.103	Sands - clean sand to silty sand (6)	0.96	1.20	0.97	10.6	68	23.2	951	2.01E+03	4.75E-04	3.59E-03	5.77E-01	6.82E-02
22	20.01	1.31	67.6	0.98	61.1	1.5	1.173	Sand mixtures - silty sand to sandy silt (5)	0.96	1.64	0.96	18.5	46	17.2	1008	1.92E+03	5.24E-04	4.69E-03	1.01E+00	1.60E-01
23	20.51	0.49	84.1	0.97	74.3	1.2	1.228	Sands - clean sand to silty sand (6)	0.96	1.38	0.94	14.5	55	18.2	1053	1.99E+03	5.28E-04	4.69E-03	9.44E-01	5.58E-02
24	21.65	1.15	70.6	1.04	61.3	1.6	1.277	Sand mixtures - silty sand to sandy silt (5)	0.95	1.72	0.94	19.2	46	17.6	1093	2.03E+03	5.38E-04	4.74E-03	9.96E-01	1.37E-01
25	22.47	0.82	45.1	1.41	35.9	3.2	1.337	Silt mixtures - clayey silt to silty clay (4)	0.95	3.22	0.94	34.4	---	23.1	679	1.81E+03	2.25E-04	4.39E-04	7.23E-02	7.11E-03
28	24.61	1.80	101.7	2.20	83.0	2.2	1.435	Sand mixtures - silty sand to sandy silt (5)	0.95	1.68	0.9	19.1	58	29.2	1217	2.51E+03	4.89E-04	2.89E-03	3.68E-01	7.97E-02
29	25.26	0.66	171.7	2.38	136.8	1.4	1.510	Sands - clean sand to silty sand (6)	0.94	1.20	0.86	10.6	79	31.3	1275	2.59E+03	4.94E-04	2.88E-03	2.94E-01	2.32E-02
30	25.75	0.49	80.7	1.47	63.6	1.9	1.545	Sand mixtures - silty sand to sandy silt (5)	0.94	1.82	0.88	20.6	47	21.1	1302	2.38E+03	5.48E-04	3.71E-03	6.29E-01	3.71E-02
31	26.25	0.49	62.7	2.38	42.2	3.9	1.574	Silt mixtures - clayey silt to silty clay (4)	0.94	3.26	0.91	34.8	---	36.1	439	1.30E+03	1.13E-04	1.98E-04	1.48E-02	8.73E-04
32	28.38	2.13	104.1	2.54	79.1	2.6	1.653	Sand mixtures - silty sand to sandy silt (5)	0.93	1.91	0.87	21.7	55	33.0	1383	2.83E+03	4.89E-04	2.18E-03	1.98E-01	5.07E-02
33	28.87	0.49	178.6	3.10	132.8	1.8	1.732	Sands - clean sand to silty sand (6)	0.93	1.29	0.81	12.7	78	34.3	1440	2.87E+03	5.02E-04	2.37E-03	1.96E-01	1.16E-02
34	29.69	0.82	106.3	2.21	78.2	2.1	1.772	Sand mixtures - silty sand to sandy silt (5)	0.92	1.73	0.83	19.6	55	27.5	1469	2.75E+03	5.35E-04	2.56E-03	3.29E-01	3.24E-02
38	32.15	0.49	95.5	1.51	67.3	1.6	1.930	Sand mixtures - silty sand to sandy silt (5)	0.91	1.62	0.85	18.2	49	19.8	1575	2.61E+03	6.04E-04	3.12E-03	5.75E-01	3.40E-02
39	32.48	0.33	110.1	1.25	77.1	1.2	1.955	Sands - clean sand to silty sand (6)	0.91	1.35	0.82	14.0	55	18.7	1591	2.56E+03	6.22E-04	3.46E-03	6.80E-01	2.68E-02
40	36.42	3.94	102.5	1.65	69.6	1.7	2.083	Sand mixtures - silty sand to sandy silt (5)	0.89	1.63	0.81	18.3	50	20.7	1669	3.E+03	6.08E-04	2.70E-03	4.68E-01	2.21E-01
42	37.24	0.49	108.5	2.27	71.0	2.2	2.236	Sand mixtures - silty sand to sandy silt (5)	0.88	1.82	0.82	20.5	51	27.2	1756	3.E+03	5.69E-04	2.01E-03	2.74E-01	1.62E-02
43	40.52	3.28	154.5	1.70	98.8	1.1	2.353	Sands - clean sand to silty sand (6)	0.86	1.25	0.75	11.8	64	22.9	1815	3.E+03	6.12E-04	2.34E-03	3.84E-01	1.51E-01
44	41.01	0.49	111.6	1.29	69.5	1.2	2.470	Sand mixtures - silty sand to sandy silt (5)	0.84	1.42	0.77	15.2	50	17.2	1869	3.E+03	6.61E-04	2.67E-03	5.94E-01	3.51E-02
45	43.64	2.62	127.5	1.34	77.9	1.1	2.567	Sands - clean sand to silty sand (6)	0.83	1.32	0.76	13.3	55	18.2	1909	3.E+03	6.54E-04	2.52E-03	5.25E-01	1.65E-01
46	43.80	0.16	125.6	1.61	75.5	1.3	2.654	Sand mixtures - silty sand to sandy silt (5)	0.82	1.42	0.73	15.2	54	19.6	1942	3052.360	6.36E-04	2.26E-03	4.11E-01	8.09E-03
47	44.62	0.82	134.6	1.52	80.4	1.2	2.685	Sands - clean sand to silty sand (6)	0.81	1.33	0.74	13.6	56	19.4	1952	3054.050	6.40E-04	0.0	0.43462	0.042778
48	44.95	0.33	156.8	2.61	93.1	1.7	2.720	Sand mixtures - silty sand to sandy silt (5)	0.80	1.43	0.73	15.4	62	27.6	1964	3433.130	5.72E-04	0.0	0.20157	0.007936
49	48.06	3.12	163.5	2.05	95.1	1.3	2.827	Sands - clean sand to silty sand (6)	0.79	1.30	0.71	12.9	63	24.0	1997	3339.520	5.98E-04	0.0	0.2749	0.102818
50	48.39	0.33	161.5	2.46	92.2	1.6	2.935	Sand mixtures - silty sand to sandy silt (5)	0.77	1.39	0.7	14.7	62	25.9	2027	3500.300	5.79E-04	0.0	0.22624	0.008907
51	48.72	0.33	153.0	1.99	87.1	1.3	2.955	Sands - clean sand to silty sand (6)	0.77	1.35	0.71	13.9	59	22.5	2032	3360.880	6.05E-04	0.0	0.30261	0.011914
52	49.70	0.98	130.0	2.67	73.5	2.2	2.995	Sand mixtures - silty sand to sandy silt (5)	0.76	1.79	0.72	20.1	52	27.8	2041	3648.220	5.62E-04	0.0	0.20888	0.024671

Evaluation of Lique[illegible]n Resistance of Soils Using [illegible]PT Data

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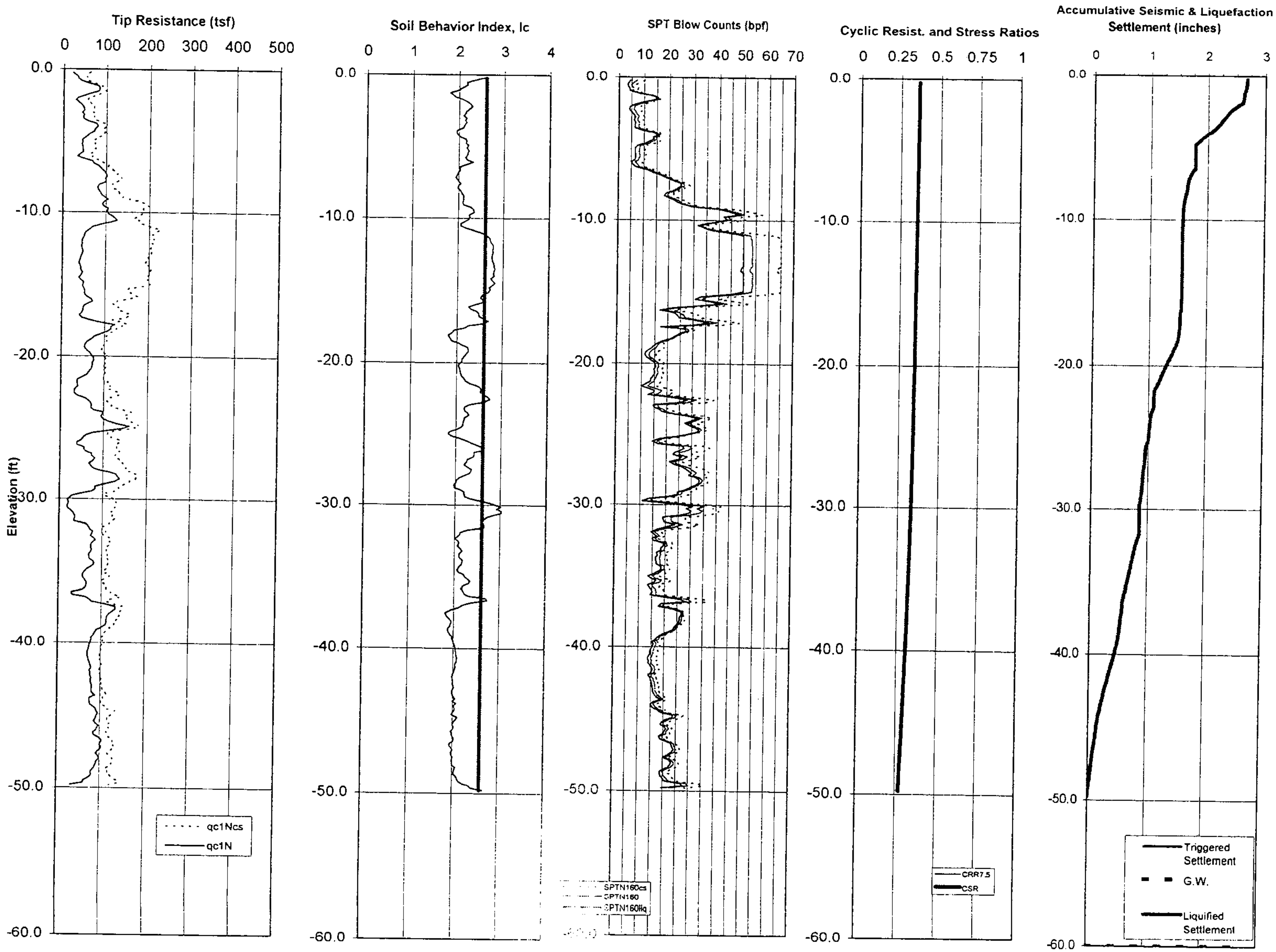
CPT:	CPT1	Elev.:	0
G.W. Depth:	60 ft.	Ic:	2.6 (Standard Value)
Design G.W. Depth:	60 ft.		

Fill Height:	0.0 ft.	125	pcf
Max horizontal acc. @ surface:	0.69 g		
Design earthquake magnitude:	6.9		
Removal:	0 ft		

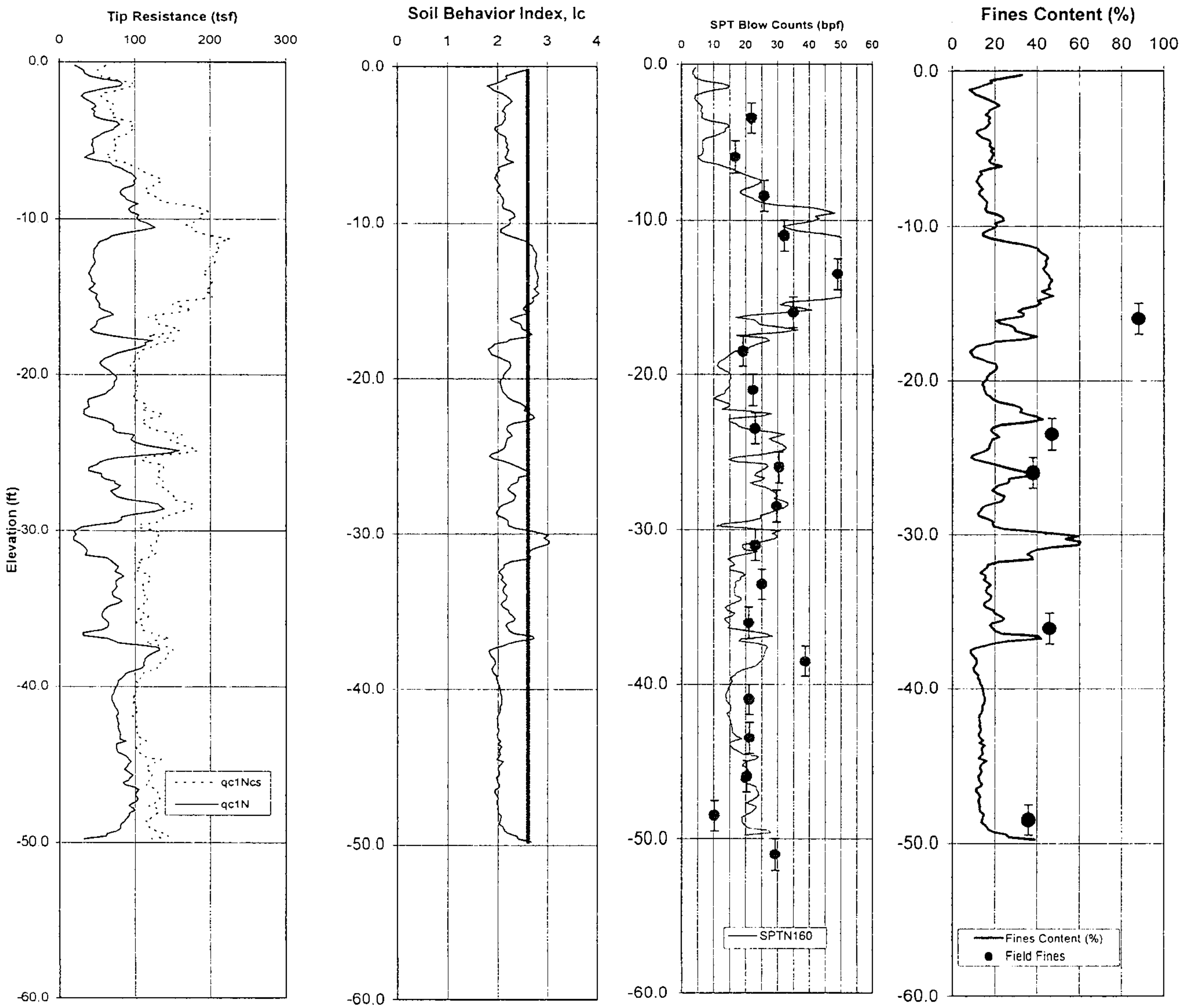


[illegible] ESTLAKE VILLAGE

*Exclude Settlement from layers thinner than 6 inches

Comparison of CPT Data and SPT Data

Boring:	B1	Elev.:	0	C:\JOBS\8953_Everest Terrace\6-29-04\CPT-01.cpd
CPT:	CPT1	Elev.:	0	
G.W. Depth:	60 ft.			
Design G.W. Depth:	60 ft.			



CPT ANALYSIS
CPT 1
DESIGN GROUNDWATER AT 40 FOOT DEPTH

"Dry" Sand Seismic Settlement Using CPT Data

CPT: CPT1
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 0.88

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Fines Content (%)	Avg. Dr (%)	Spt N160cs (bpf)	Cyclic Shear Stress τav (psf)	Gmax (ksf)	γeff / (Geff/Gmax)	γeff	Vol. Strain (%)	"Dry" Settle. (in)
1	0.82	0.49	17.6	0.13	28.7	0.8	0.031	Sand mixtures - silty sand to sandy silt (5)	1.00	2.09	1	23.2	75	8.2	28	2.16E+02	1.24E-04	4.21E-04	2.70E-01	1.85E-02
2	1.80	0.98	41.4	0.28	67.5	0.7	0.080	Sands - clean sand to silty sand (6)	1.00	1.26	1	11.8	87	12.4	72	4.12E+02	1.73E-04	1.20E-03	4.65E-01	5.50E-02
3	3.77	1.97	26.6	0.22	43.3	0.8	0.170	Sand mixtures - silty sand to sandy silt (5)	1.00	1.60	1	18.0	52	9.7	152	5.77E+02	2.60E-04	5.31E-03	2.23E+00	5.27E-01
4	4.43	0.66	46.3	0.43	75.4	0.9	0.249	Sands - clean sand to silty sand (6)	0.99	1.28	1	12.6	66	16.3	222	8.30E+02	2.67E-04	1.42E-02	2.64E+00	2.08E-01
5	6.40	1.97	28.4	0.29	46.1	1.0	0.329	Sand mixtures - silty sand to sandy silt (5)	0.99	1.65	1	18.7	39	11.1	292	8.61E+02	3.39E-04	3.69E-02	3.56E-01	8.40E-02
6	7.55	1.15	57.9	0.65	86.8	1.1	0.424	Sands - clean sand to silty sand (6)	0.99	1.29	1	12.7	63	20.5	375	1.18E+03	3.19E-04	5.99E-03	9.65E-01	1.33E-01
7	7.71	0.16	68.6	1.10	98.5	1.6	0.465	Sand mixtures - silty sand to sandy silt (5)	0.98	1.38	1	14.5	68	28.3	410	1.37E+03	2.98E-04	2.29E-03	2.62E-01	5.15E-03
8	8.20	0.49	66.8	0.87	94.0	1.3	0.485	Sands - clean sand to silty sand (6)	0.98	1.31	1	13.2	66	24.1	428	1.33E+03	3.21E-04	3.18E-03	4.66E-01	2.75E-02
9	9.51	1.31	68.4	1.50	91.0	2.2	0.540	Sand mixtures - silty sand to sandy silt (5)	0.98	1.59	1	17.7	65	32.9	475	1.55E+03	3.09E-04	2.31E-03	2.83E-01	4.46E-02
10	9.68	0.16	73.3	2.81	93.9	3.9	0.584	Silt mixtures - clayey silt to silty clay (4)	0.98	2.11	1	24.3	---	57.6	513	2.79E+03	1.84E-04	3.01E-04	1.62E-02	3.19E-04
11	10.83	1.15	90.7	2.34	112.4	2.6	0.623	Sand mixtures - silty sand to sandy silt (5)	0.98	1.60	1	18.0	73	43.3	547	1.83E+03	2.99E-04	1.49E-03	9.12E-02	1.26E-02
12	10.99	0.16	73.1	2.74	87.9	3.8	0.663	Silt mixtures - clayey silt to silty clay (4)	0.98	2.15	1	24.8	---	53.9	581	2.90E+03	2.00E-04	3.39E-04	1.83E-02	3.60E-04
15	15.91	0.66	53.1	2.28	53.5	4.4	0.944	Silt mixtures - clayey silt to silty clay (4)	0.97	3.02	1	32.9	---	46.5	819	3.30E+03	2.49E-04	4.44E-04	2.39E-02	1.89E-03
16	16.57	0.66	63.8	1.68	63.0	2.7	0.983	Sand mixtures - silty sand to sandy silt (5)	0.97	2.13	1	24.4	48	29.9	852	2.10E+03	4.08E-04	2.21E-03	2.61E-01	2.05E-02
17	17.06	0.49	50.0	1.74	48.5	3.6	1.018	Silt mixtures - clayey silt to silty clay (4)	0.96	2.84	1	31.2	---	35.5	881	3.13E+03	2.82E-04	5.26E-04	4.17E-02	2.47E-03
19	17.39	0.16	48.4	2.00	46.3	4.2	1.047	Silt mixtures - clayey silt to silty clay (4)	0.96	3.21	0.99	34.4	---	41.4	905	3.35E+03	2.70E-04	4.82E-04	2.61E-02	5.13E-04
20	17.72	0.33	82.5	1.70	78.4	2.1	1.062	Sand mixtures - silty sand to sandy silt (5)	0.96	1.69	0.99	19.2	57	27.0	917	2.10E+03	4.39E-04	2.66E-03	3.51E-01	1.38E-02
21	18.70	0.98	111.9	1.14	104.4	1.0	1.103	Sands - clean sand to silty sand (6)	0.96	1.20	0.97	10.6	68	23.2	951	2.01E+03	4.75E-04	3.59E-03	5.77E-01	6.82E-02
22	20.01	1.31	67.6	0.98	61.1	1.5	1.173	Sand mixtures - silty sand to sandy silt (5)	0.96	1.64	0.96	18.5	46	17.2	1008	1.92E+03	5.24E-04	4.69E-03	1.01E+00	1.60E-01
23	20.51	0.49	84.1	0.97	74.3	1.2	1.228	Sands - clean sand to silty sand (6)	0.96	1.38	0.94	14.5	55	18.2	1053	1.99E+03	5.28E-04	4.69E-03	9.44E-01	5.58E-02
24	21.65	1.15	70.6	1.04	61.3	1.6	1.277	Sand mixtures - silty sand to sandy silt (5)	0.95	1.72	0.94	19.2	46	17.6	1093	2.03E+03	5.38E-04	4.74E-03	9.96E-01	1.37E-01
25	22.47	0.82	45.1	1.41	35.9	3.2	1.337	Silt mixtures - clayey silt to silty clay (4)	0.95	3.22	0.94	34.4	---	23.1	679	1.81E+03	2.25E-04	4.39E-04	7.23E-02	7.11E-03
28	24.61	1.80	101.7	2.20	83.0	2.2	1.435	Sand mixtures - silty sand to sandy silt (5)	0.95	1.68	0.9	19.1	58	29.2	1217	2.51E+03	4.89E-04	2.89E-03	3.68E-01	7.97E-02
29	25.26	0.66	171.7	2.38	136.8	1.4	1.510	Sands - clean sand to silty sand (6)	0.94	1.20	0.86	10.6	79	31.3	1275	2.59E+03	4.94E-04	2.88E-03	2.94E-01	2.32E-02
30	25.75	0.49	80.7	1.47	63.6	1.9	1.545	Sand mixtures - silty sand to sandy silt (5)	0.94	1.82	0.88	20.6	47	21.1	1302	2.38E+03	5.48E-04	3.71E-03	6.29E-01	3.71E-02
31	26.25	0.49	62.7	2.38	42.2	3.9	1.574	Silt mixtures - clayey silt to silty clay (4)	0.94	3.26	0.91	34.8	---	36.1	439	1.30E+03	1.13E-04	1.98E-04	1.48E-02	8.73E-04
32	28.38	2.13	104.1	2.54	79.1	2.6	1.653	Sand mixtures - silty sand to sandy silt (5)	0.93	1.91	0.87	21.7	55	33.0	1383	2.83E+03	4.89E-04	2.18E-03	1.98E-01	5.07E-02
33	28.87	0.49	178.6	3.10	132.8	1.8	1.732	Sands - clean sand to silty sand (6)	0.93	1.29	0.81	12.7	78	34.3	1440	2.87E+03	5.02E-04	2.37E-03	1.96E-01	1.16E-02
34	29.69	0.82	106.3	2.21	78.2	2.1	1.772	Sand mixtures - silty sand to sandy silt (5)	0.92	1.73	0.83	19.6	55	27.5	1469	2.75E+03	5.35E-04	2.56E-03	3.29E-01	3.24E-02
38	32.15	0.49	95.5	1.51	67.3	1.6	1.930	Sand mixtures - silty sand to sandy silt (5)	0.91	1.62	0.85	18.2	49	19.8	1575	2.61E+03	6.04E-04	3.12E-03	5.75E-01	3.40E-02
39	32.48	0.33	110.1	1.25	77.1	1.2	1.955	Sands - clean sand to silty sand (6)	0.91	1.35	0.82	14.0	55	18.7	1591	2.56E+03	6.22E-04	3.46E-03	6.80E-01	2.68E-02
40	36.42	3.94	102.5	1.65	69.6	1.7	2.083	Sand mixtures - silty sand to sandy silt (5)	0.89	1.63	0.81	18.3	50	20.7	1669	3.E+03	6.08E-04	2.70E-03	4.68E-01	2.21E-01
42	37.24	0.49	108.5	2.27	71.0	2.2	2.236	Sand mixtures - silty sand to sandy silt (5)	0.88	1.82	0.82	20.5	51	27.2	1756	3.E+03	5.69E-04	2.01E-03	2.74E-01	1.62E-02
43	40.52	2.76	154.5	1.70	98.8	1.1	2.353	Sands - clean sand to silty sand (6)	0.86	1.25	0.75	11.8	64	22.9	1536	3.E+03	5.14E-04	1.95E-03	3.00E-01	1.18E-01

Liquefaction Analysis Using CPT Data

1.36

3.59

CPT: CPT1
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 1.24

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Avg. Fines Content (%)	Avg. Dr (%)	Avg. SPT N1(60)liq (bpf)	Min. CRR (M=7.5)	Avg. CSR	Min. Liq. FS	Avg. Strain (%)	Liq Settle. (in)
43	40.52	0.52	154.5	1.70	98.8	1.1	2.353	Sands - clean sand to silty sand (6)	0.86	1.25	0.75	11.8	64	109.1	0.13	0.312	0.43	1.8	0.12
44	41.01	0.49	111.6	1.29	69.5	1.2	2.470	Sand mixtures - silty sand to sandy silt (5)	0.84	1.42	0.78	15.2	50	15.2	0.13	0.309	0.42	1.9	0.11
45	43.64	2.62	127.5	1.34	77.9	1.1	2.567	Sands - clean sand to silty sand (6)	0.83	1.32	0.76	13.3	55	16.7	0.12	0.309	0.40	1.8	0.55
46	43.80	0.16	125.6	1.61	75.5	1.3	2.654	Sand mixtures - silty sand to sandy silt (5)	0.82	1.42	0.74	15.2	54	17.5	0.14	0.309	0.47	1.7	0.03
47	44.62	0.82	134.6	1.52	80.4	1.2	2.685	Sands - clean sand to silty sand (6)	0.81	1.33	0.76	13.6	56	17.8	0.13	0.309	0.43	1.6	0.16
48	44.95	0.33	156.8	2.61	93.1	1.7	2.720	Sand mixtures - silty sand to sandy silt (5)	0.80	1.43	0.74	15.4	62	25.1	0.22	0.309	0.70	0.0	0
49	48.06	3.12	163.5	2.05	95.1	1.3	2.827	Sands - clean sand to silty sand (6)	0.79	1.30	0.73	12.9	63	22.5	0.16	0.307	0.52	1.1	0.26
50	48.39	0.33	161.5	2.46	92.2	1.6	2.935	Sand mixtures - silty sand to sandy silt (5)	0.77	1.39	0.72	14.7	62	23.7	0.20	0.306	0.64	0.0	0
51	48.72	0.33	153.0	1.99	87.1	1.3	2.955	Sands - clean sand to silty sand (6)	0.77	1.35	0.73	13.9	59	20.7	0.16	0.305	0.53	1.4	0.05
52	49.70	0.98	130.0	2.67	73.5	2.2	2.995	Sand mixtures - silty sand to sandy silt (5)	0.76	1.79	0.74	20.1	52	24.1	0.17	0.305	0.55	0.9	0.08

Evaluation of Lique[illegible]n Resistance of Soils Using CPT Data

CPT: CPT1
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.

Elev.: 0

Ic: 2.6 (Standard Value)

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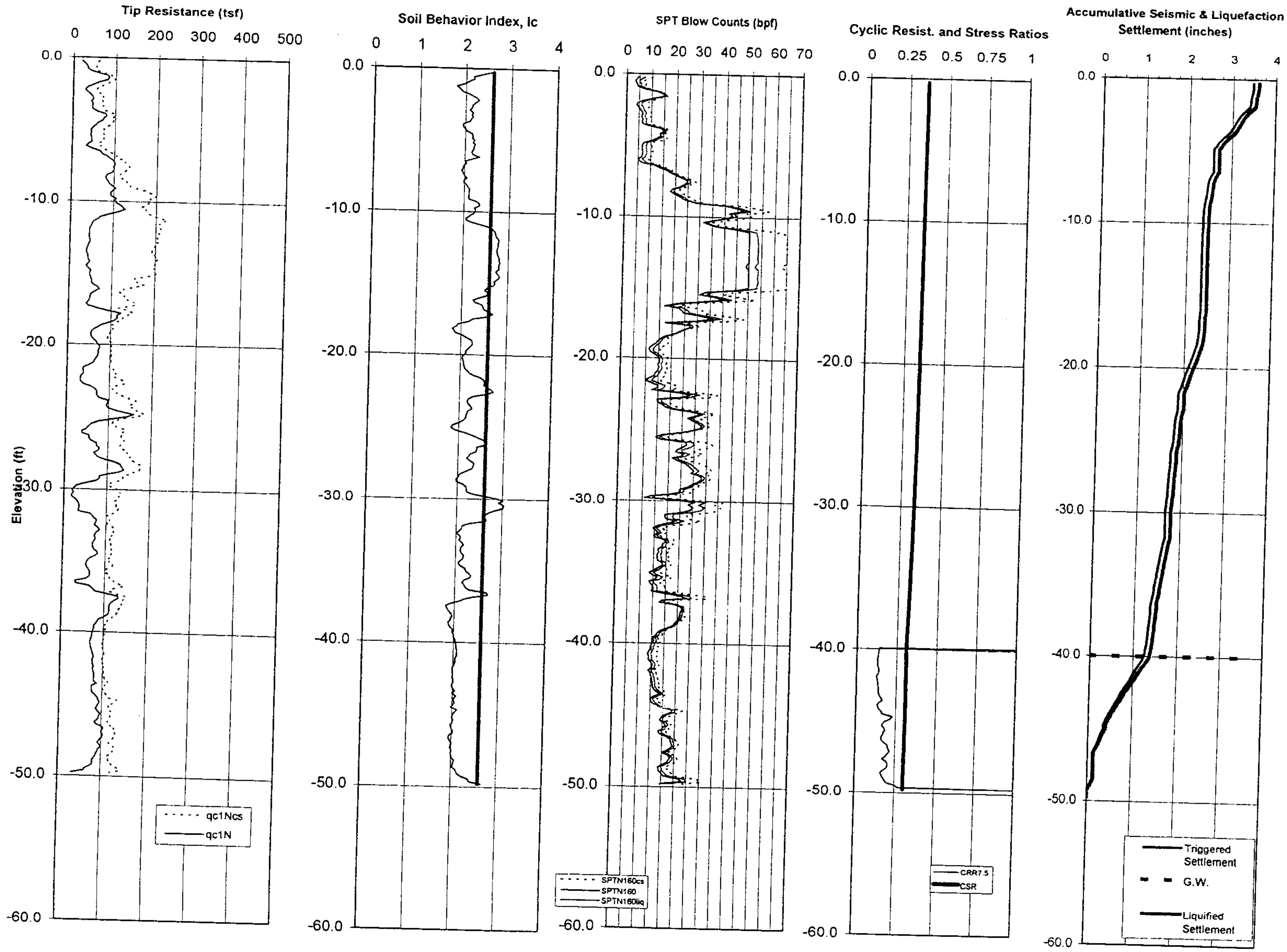
Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.9
Removal: 0 ft



GEOLABS-WESTLAKE VILLAGE

*Exclude Settlement from layers thinner than 6 inches

CPT ANALYSIS
CPT 2

GEOLABS-WESTLAKE VILLAGE

Summary of Analysis of CPT Data

CPT: CPT2
G.W. Depth: 60 ft
Design G.W. Depth: 60 ft
Removal: 0 ft

W.O.: 8953
Elev.: 0
Ic: 2.6

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Fill Height: 0.0 ft 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Est. Wet Den. (pcf)	Soil Behavior Type	O.B. (tsf)	Eff. O.B. (tsf)	Norm. Exp. n	Ic Range	Kc	Fines Content (%)	Phi (deg)	Dr (%)	Su (tsf)	SPT N1(60) (bpf)	SPT N1(60)cs (bpf)
1	2.62	2.30	56.9	0.23	92.7	0.3	125	Sands - clean sand to silty sand	0.092	0.092	0.5	1.41 to 1.98	1.09	7.1	49	78.0	—	14.1	14.5
2	3.28	0.66	14.3	0.01	23.3	0.1	120	Sand mixtures - silty sand to sandy silt	0.184	0.184	0.5	2.06 to 2.14	1.47	16.0	39	24.0	—	2.0	4.9
3	9.68	6.40	71.1	0.65	104.9	0.8	125	Sands - clean sand to silty sand	0.403	0.403	0.5	1.64 to 2.06	1.15	9.2	43	71.0	—	20.4	21.5
4	10.01	0.33	54.1	0.84	67.7	1.6	120	Sand mixtures - silty sand to sandy silt	0.613	0.613	0.5	2.16 to 2.18	1.59	17.9	40	52.0	—	15.2	19.5
5	10.17	0.16	59.2	0.70	73.0	1.2	125	Sands - clean sand to silty sand	0.628	0.628	0.5	2.06	1.39	14.7	40	55.0	—	15.0	18.1
6	10.83	0.66	63.3	0.88	76.7	1.4	120	Sand mixtures - silty sand to sandy silt	0.653	0.653	0.5	2.07 to 2.12	1.45	15.7	40	57.0	—	17.1	20.6
7	12.30	1.48	93.7	1.07	108.0	1.1	125	Sands - clean sand to silty sand	0.719	0.719	0.5	1.75 to 2.07	1.22	11.0	42	70.0	—	22.8	24.7
8	12.96	0.66	56.2	1.00	62.2	1.9	120	Sand mixtures - silty sand to sandy silt	0.785	0.785	0.5	2.07 to 2.44	1.85	20.8	39	47.0	—	15.5	20.4
9	13.12	0.16	40.7	1.61	44.3	4.0	120	Silt mixtures - clayey silt to silty clay	0.809	0.809	0.5	2.58	3.21	34.4	—	—	2.7	28.3	38.7
10	15.75	2.62	37.4	2.43	40.9	6.7	120	Clays - silty clay to clay	0.893	0.893	1	2.69 to 2.82	4.49	43.9	—	—	2.4	44.9	58.8
11	16.40	0.66	43.9	1.88	43.2	4.4	120	Silt mixtures - clayey silt to silty clay	0.991	0.991	0.5 to 1	2.55 to 2.71	3.45	36.2	—	—	2.9	30.1	40.8
12	16.73	0.33	29.4	1.55	27.8	5.5	120	Clays - silty clay to clay	1.021	1.021	1	2.8 to 2.84	4.94	47.0	—	—	1.9	29.1	39.9
13	17.06	0.33	37.7	1.60	35.2	4.4	120	Silt mixtures - clayey silt to silty clay	1.040	1.040	1	2.64 to 2.72	3.84	39.3	—	—	2.4	26.5	36.8
14	17.88	0.82	34.6	1.71	31.2	5.1	120	Clays - silty clay to clay	1.075	1.075	1	2.74 to 2.81	4.46	43.7	—	—	2.2	28.9	39.7
15	18.04	0.16	47.3	1.56	44.1	3.4	120	Silt mixtures - clayey silt to silty clay	1.104	1.104	0.5	2.53	2.91	32.0	—	—	3.1	22.5	31.1
16	18.21	0.16	55.3	1.38	51.3	2.5	120	Sand mixtures - silty sand to sandy silt	1.114	1.114	0.5	2.40	2.30	26.3	37	39.0	—	16.4	22.8
17	18.54	0.33	48.8	1.74	44.9	3.7	120	Silt mixtures - clayey silt to silty clay	1.129	1.129	0.5	2.51 to 2.58	3.02	32.8	—	—	3.2	25.9	35.4
18	18.70	0.16	55.2	1.16	50.5	2.2	120	Sand mixtures - silty sand to sandy silt	1.144	1.144	0.5	2.35	2.13	24.5	36	39.0	—	13.1	18.8
19	20.51	1.80	95.8	0.88	85.5	0.9	125	Sands - clean sand to silty sand	1.205	1.205	0.5	1.82 to 2.08	1.24	11.6	39	60.0	—	16.6	18.4
20	21.82	1.31	52.5	0.49	45.1	1.0	120	Sand mixtures - silty sand to sandy silt	1.301	1.301	0.5	2.09 to 2.27	1.63	18.3	35	33.0	—	6.6	10.4
21	21.98	0.16	66.7	0.49	56.3	0.7	125	Sands - clean sand to silty sand	1.345	1.345	0.5	2.04	1.35	14.0	37	43.0	—	7.7	10.2
22	22.80	0.82	57.0	0.68	47.6	1.2	120	Sand mixtures - silty sand to sandy silt	1.375	1.375	0.5	2.11 to 2.41	1.75	19.8	36	36.0	—	8.0	12.1
23	23.29	0.49	43.7	1.15	34.0	2.7	120	Silt mixtures - clayey silt to silty clay	1.414	1.414	0.5 to 1	2.49 to 2.65	3.08	33.2	—	—	2.8	10.7	17.4
24	27.07	3.77	59.4	0.99	46.8	1.7	120	Sand mixtures - silty sand to sandy silt	1.542	1.542	0.5	2.11 to 2.45	2.02	23.1	35	34.0	—	10.1	15.1
25	27.40	0.33	49.4	1.28	33.4	2.7	120	Silt mixtures - clayey silt to silty clay	1.665	1.665	0.5 to 1	2.48 to 2.65	3.14	33.7	—	—	3.2	10.2	16.7
26	27.56	0.16	51.6	1.20	39.0	2.4	120	Sand mixtures - silty sand to sandy silt	1.680	1.680	0.5	2.47	2.61	29.3	34	27.0	—	10.7	17
27	27.72	0.16	53.3	1.35	40.1	2.6	120	Silt mixtures - clayey silt to silty clay	1.690	1.690	0.5	2.48	2.68	29.9	—	—	3.4	12.3	18.9
28	28.05	0.33	54.7	1.30	41.0	2.4	120	Sand mixtures - silty sand to sandy silt	1.705	1.705	0.5	2.46 to 2.46	2.56	28.8	34	29.0	—	11.7	18
29	28.38	0.33	57.9	1.51	43.1	2.7	120	Silt mixtures - clayey silt to silty clay	1.724	1.724	0.5	2.46 to 2.47	2.61	29.2	—	—	3.7	14.0	20.8
30	28.54	0.16	60.8	1.38	45.1	2.3	120	Sand mixtures - silty sand to sandy silt	1.739	1.739	0.5	2.41	2.36	26.9	34	33.0	—	12.4	18.5
31	29.53	0.98	67.7	2.58	41.2	3.9	120	Silt mixtures - clayey silt to silty clay	1.774	1.774	0.5 to 1	2.48 to 2.7	3.33	35.2	—	—	4.4	25.5	35.1
32	31.17	1.64	86.6	5.43	45.7	6.4	120	Clays - silty clay to clay	1.852	1.852	1	2.66 to 2.75	4.12	41.3	—	—	5.7	45.0	59
33	31.33	0.16	92.0	3.27	65.2	3.6	120	Silt mixtures - clayey silt to silty clay	1.907	1.907	0.5	2.43	2.44	27.7	—	—	6.0	33.2	42.3
34	31.66	0.33	236.2	8.36	166.7	3.6	125	Very stiff sand to clayey sand*	1.922	1.922	0.5	2.13 to 2.2	1.60	18.0	42	87.0	—	50.0	56.5
35	31.82	0.16	184.2	9.97	59.2	8.7	125	Very stiff, fine grained*	1.937	1.937	0.5	2.38	2.24	25.7	—	—	12.2	50.0	60.4
36	35.10	3.28	85.4	6.73	40.9	8.1	120	Clays - silty clay to clay	2.041	2.041	1	2.69 to 2.89	5.02	47.6	—	—	5.6	49.3	64.2
37	35.60	0.49	115.0	4.97	69.0	4.4	120	Silt mixtures - clayey silt to silty clay	2.154	2.154	0.5 to 1	2.39 to 2.6	2.73	30.2	—	—	7.5	42.7	53.9
38	35.76	0.16	121.7	4.24	80.8	3.5	120	Sand mixtures - silty sand to sandy silt	2.174	2.174	0.5	2.36	2.16	24.9	37	57.0	—	38.6	47.3
39	35.93	0.16	105.9	3.98	70.1	3.8	120	Silt mixtures - clayey silt to silty clay	2.183	2.183	0.5	2.43	2.43	27.5	—	—	6.9	37.5	47
40	37.40	1.48	63.7	3.92	27.5	6.5	120	Clays - silty clay to clay	2.233	2.233	1	2.71 to 3.03	5.53	50.8	—	—	4.1	34.6	46.5
41	37.57	0.16	87.2	3.72	37.2	4.4	120	Silt mixtures - clayey silt to silty clay	2.282	2.282	1	2.66	3.71	38.4	—	—	5.7	27.5	38
42	38.06	0.49	83.1	4.76	35.1	5.9	120	Clays - silty clay to clay	2.302	2.302	1	2.74 to 2.81	4.54	44.3	—	—	5.4	36.8	49.2
43	38.22	0.16	121.6	5.75	78.1	4.8	125	Very stiff, fine grained*	2.321	2.321	0.5	2.47	2.63	29.4	—	—	7.9	50.0	62.2
44	38.71	0.49	167.9	5.42	107.4	3.3	120	Sand mixtures - silty sand to sandy silt	2.341	2.341	0.5	2.21 to 2.3	1.82	20.9	39	68.0	—	46.0	53.7
45	38.88	0.16	145.5	6.60	92.7	4.6	125	Very stiff, fine grained*	2.361	2.361	0.5	2.41	2.35	26.8	—	—	9.5	50.0	60.9
46	39.04	0.16	105.0	6.14	43.3	6.0	120	Clays - silty clay to clay	2.371	2.371	1	2.71	4.08	41.1	—	—	6.8	42.1	55.5
47	39.37	0.33	125.6	5.60	67.5	4.6	120	Silt mixtures - clayey silt to silty clay	2.386	2.386	0.5 to 1	2.37 to 2.66	2.96	31.8	—	—	8.2	43.2	54.4
48	39.53	0.16	140.1	6.13	88.5	4.5	125	Very stiff, fine grained*	2.401	2.401	0.5	2.41	2.36	26.9	—	—	9.2	50.0	60.9
49	39.70	0.16	102.7	5.51	41.6	5.5	120	Clays - silty clay to clay	2.411	2.411	1	2.70	3.97	40.2	—	—	6.7	38.0	50.7
50	40.19	0.49	122.3	5.76	59.5	4.8	120	Silt mixtures - clayey silt to silty clay	2.431	2.431	0.5 to 1	2.4 to 2.64	3.17	33.8	—	—	8.0	41.7	53.6
51	40.68	0.49	65.3	3.35	25.5	5.3	120	Clays - silty clay to clay	2.460	2.460	1	2.82 to 2.84	5.07	47.9	—	—	4.2	24.6	34.6
52	41.01	0.33	114.5	4.77	59.8	4.2	120	Silt mixtures - clayey silt to silty clay	2.485	2.485	0.5 to 1	2.42 to 2.63	2.97	32.2	—	—	7.5	37.1	47.7
53	41.50	0.49	159.0	6.55	98.2	4.2	125	Very stiff, fine grained*	2.510	2.510	0.5	2.35 to 2.37	2.16	24.9	—	—	10.4	50.0	60
54	41.83	0.33	171.5	5.78	105.4	3.4	120	Sand mixtures - silty sand to sandy silt	2.535	2.535	0.5	2.27 to 2.28	1.87	21.6	39	68.0	—	47.3	55.4
55	41.99	0.16	163.7	6.24	100.3	3.9	125	Very stiff sand to clayey sand*	2.550	2.550	0.5	2.33	2.05	23.6	38	66.0	—	50.0	59.4
56	42.32	0.33	98.0	4.56	37.2	4.8	120	Silt mixtures - clayey silt to silty clay	2.565	2.565	1	2.68 to 2.69	3.89	39.7	—	—	6.4	30.4	41.4
57	42.49	0.16	165.3	5.64	100.7	3.5	120	Sand mixtures - silty sand to sandy silt	2.580	2.580	0.5	2.29	1.92	22.2	38	66.0	—	46.0	54.3
58	42.98	0.49	107.4	4.57	40.3	4.4	120	Silt mixtures - clayey silt to silty clay	2.600	2.600	1	2.61 to 2.65	3.54	37.0	—	—	7.0	28.9	39.7
59	43.14	0.16	67.0	4.69	24.6	7.3	120	Clays - silty clay to clay	2.619	2.619	1	2.94	6.13	54.7	—	—	4.3	36.9	49.3
60	43.31	0.16	113.5	4.05	68.5	3.7	120	Silt mixtures - clayey silt to silty clay	2.629	2.629	0.5	2.42	2.39	27.2	—	—	7.4	34.7	43.8
61	43.47	0.16	178.9	5.12	107.8	2.9	120	Sand mixtures - silty sand to sandy silt	2.639	2.639	0.5	2.21	1.70	19.5	39	68.0	—	40.5	47.1
62	43.64	0.16	134.9	5.54	81.1	4.2	120	Silt mixtures - clayey silt to silty clay	2.649	2.649	0.5	2.41	2.37	27.0	—	—	8.8	46.2	56.7
63	44.62	0.98	83.4	4.02	30.1	5.0	120	Clays - silty clay to clay	2.683	2.683	1	2.72 to 2.91	4.56	44.4	—	—	5.4	27.5	38
64	44.95	0.33	101.9	4.11	36.4	4.1	120	Silt mixtures - clayey silt to silty clay	2.723	2.723	1	2.64 to 2.66	3.64	37.8	—	—	6.6	25.5	35.5
65	46.26	1.31	79.9	4.39	27.8	5.7	120	Clays - silty clay to clay	2.772	2.772	1	2.75 to 2.92	5.09	48.0	—	—	5.1	30.6	41.7
66	46.42	0.16	110.6	5.26	38.3	4.9	120	Silt mixtures - clayey silt to silty clay	2.816	2.816	1	2.68	3.88	39.6	—	—	7.2	31.7	43
67	46.92	0.49	77.1	4.60	26.2	6.3	120	Clays - silty clay to clay	2.836	2.836	1	2.81 to 2.99	5.52	50.7	—	—	5.0	32.4	43.9
68	47.24	0.33	109.4	4.99	37.2	4.7	120	Silt mixtures - clayey silt to silty clay	2.860	2.860	1	2.65 to 2.7	3.86	39.4	—	—	7.1	29.6	40.5
69	47.74	0.49	81.6	4.65	27.3	6.0	120	Clays - silty clay to clay	2.885	2.885	1	2.79 to 2.94	5.25	49.0	—	—	5.3	31.6	42.9
70	47.90	0.16	127.4	4.67	73.2	3.7	120	Silt mixtures - clayey silt to silty clay	2.905	2.905	0.5	2.41	2.34	26.7	—	—	8.3	37.8	47.1
71	48.88	0.98	191.5	5.65	109.3	3.0	120	Sand mixtures - silty sand to sandy silt	2.939	2.939	0.5	2.16 to 2.32	1.73	19.7	38	69.0	—	42.6	49.5
72	49.05	0.16	117.9	4.81	38.7	4.2	120	Silt mixtures - clayey silt to silty clay	2.974	2.974	1	2.63	3.54	37.0	—	—	7.7	26.8	37.2
73	49.87	0.82	58.8	2.20	18.6	3.9	120	Clays - silty clay to clay	3.003	3.003	1	2.8 to 2.91	5.26	49.2	—	—	3.7	9.7	16.6

Evaluation of Soil Characteristics Using CPT Data

CPT: CPT2
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Elev.: 0

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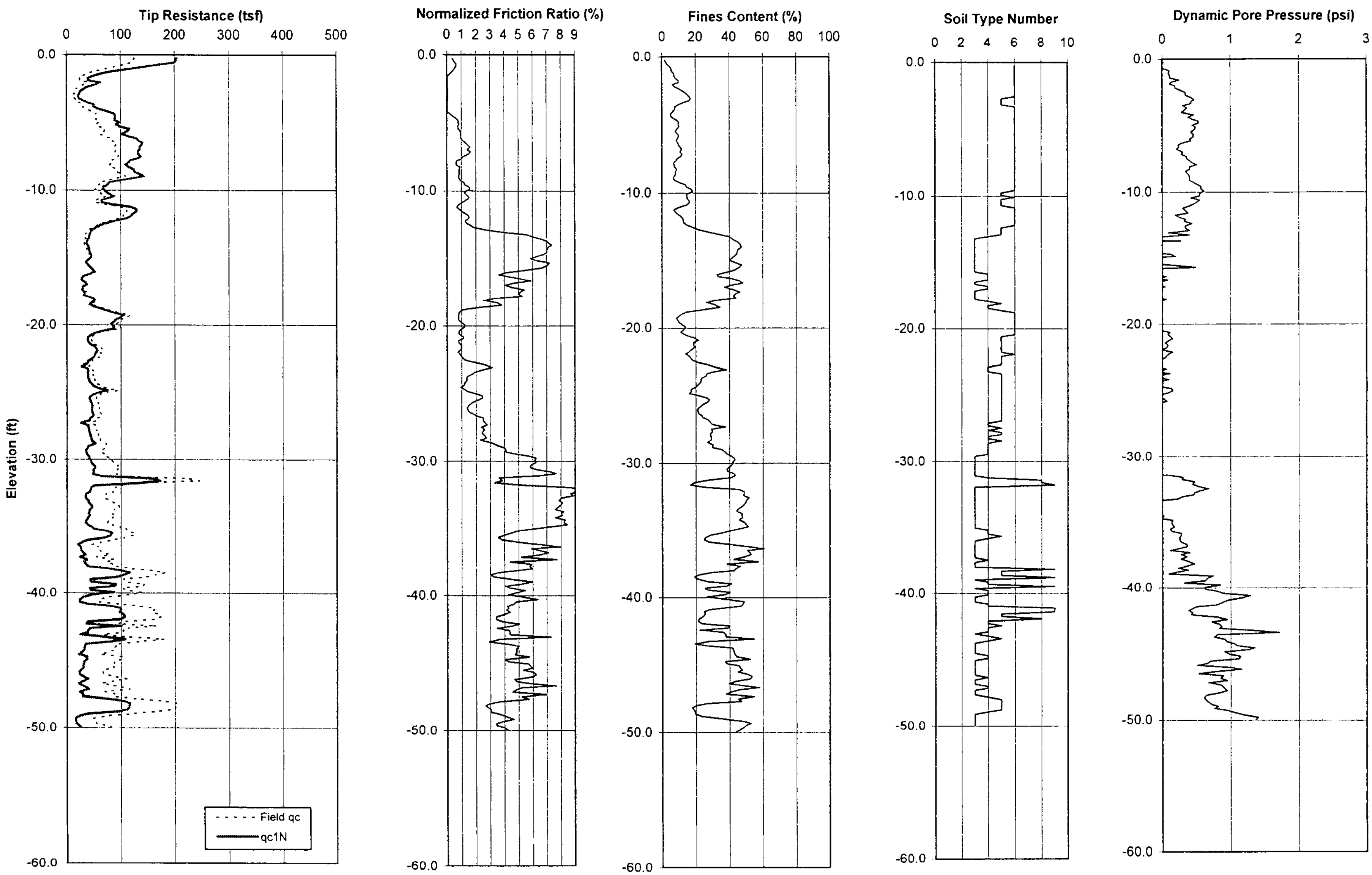

CPT-Based Soil Behavior Type

CPT: **CPT2**
G.W. Depth: 60 ft
Ic: 2.6

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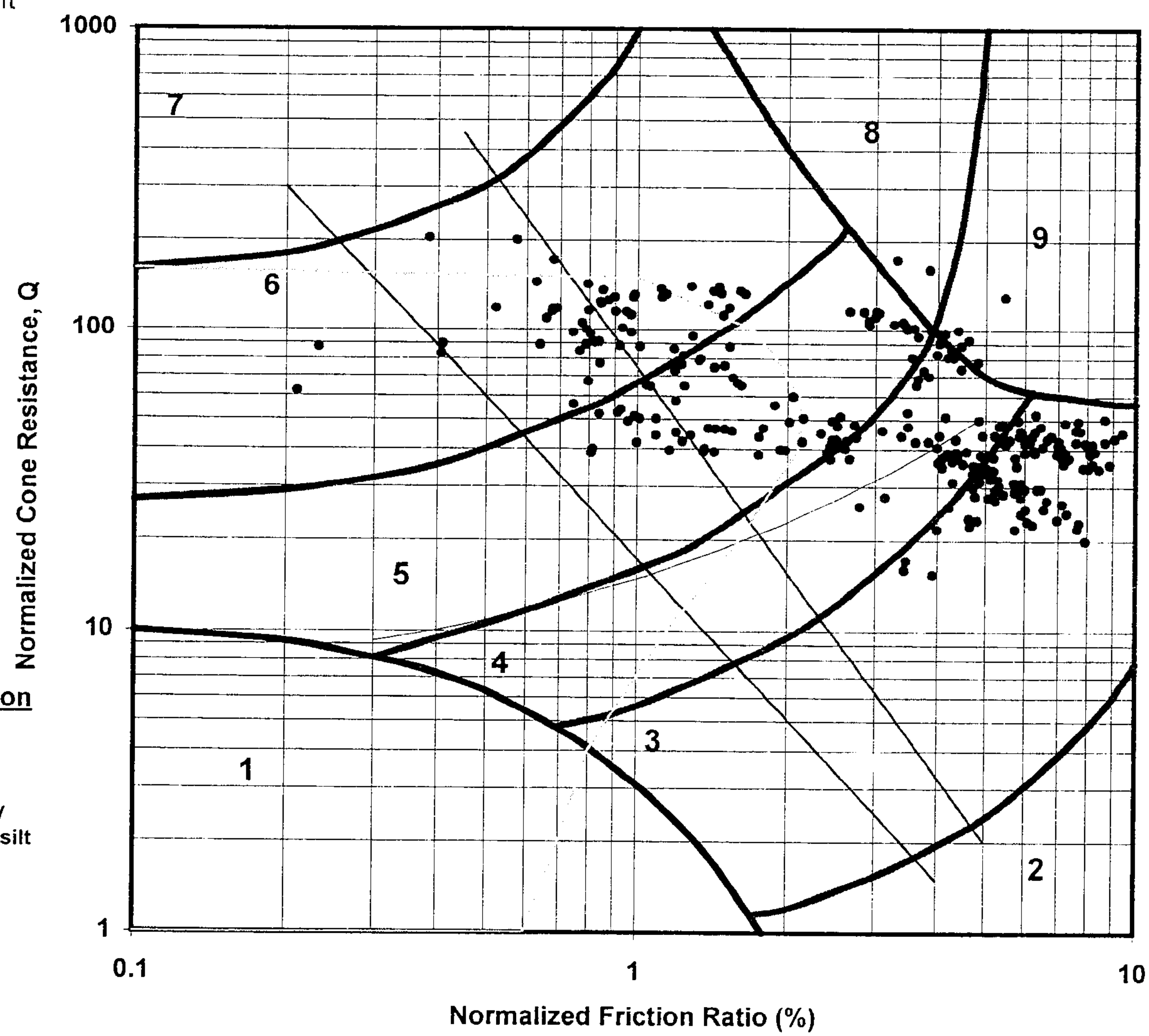

Soil Consistency Number

(Sand / Clay)

1 = very loose / very soft
2 = loose / soft
3 = medium dense / medium stiff
4 = dense / stiff
5 = very dense / very stiff
6 = --- / hard

Soil Behavior Type Classification

1. Sensitive Fine Grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

*Heavily overconsolidated or cemented

GEOLABS-WESTLAKE VILLAGE

Soil Characteristics and Engineering Characteristics
Using CPT Data

CPT: CPT2
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Elev.: 0

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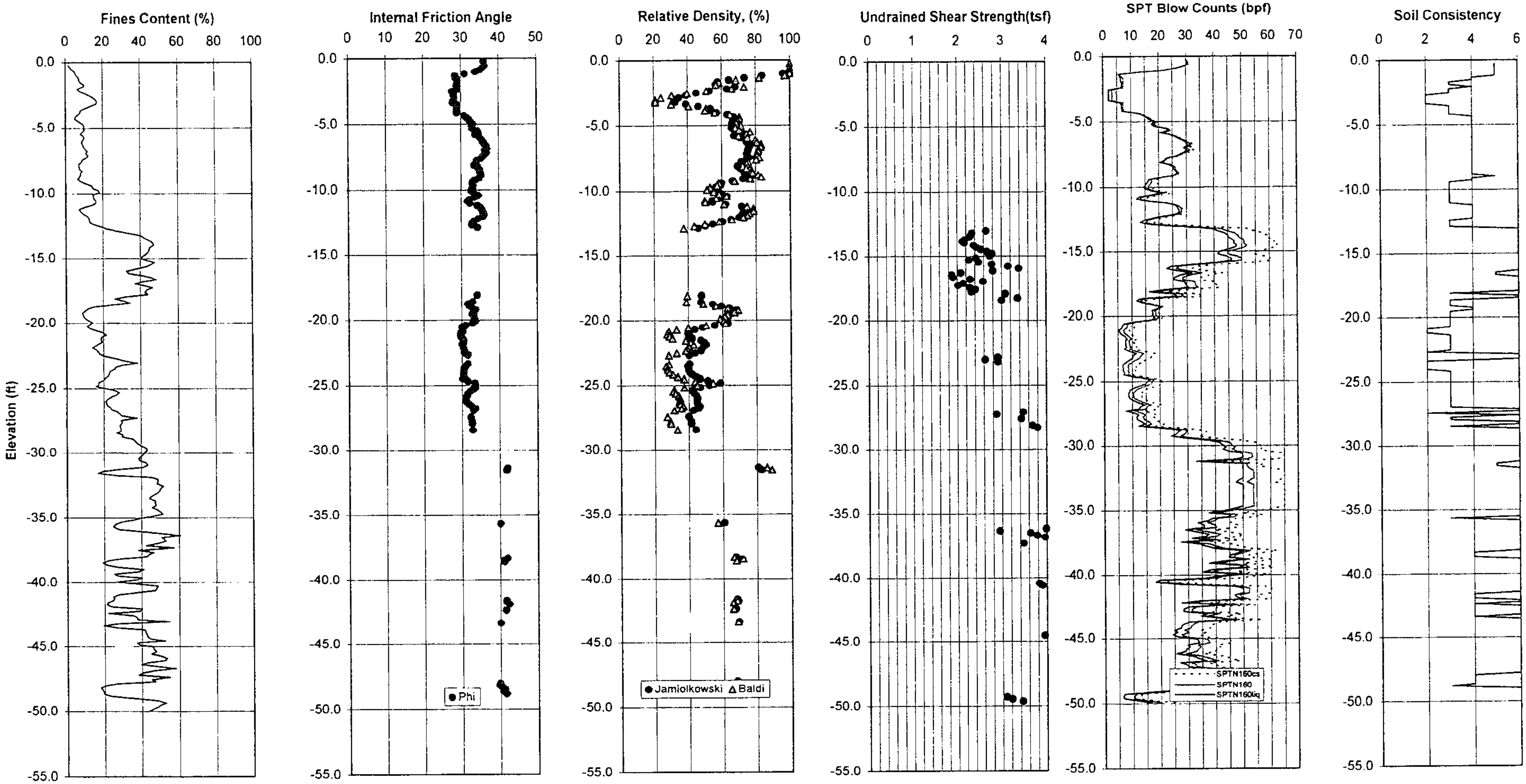

CPT ANALYSIS
CPT 2
NO DESIGN GROUNDWATER

GEOLABS-WESTLAKE VILLAGE

"Dry" Sand Seismic Settlement Using CPT Data

CPT: CPT2
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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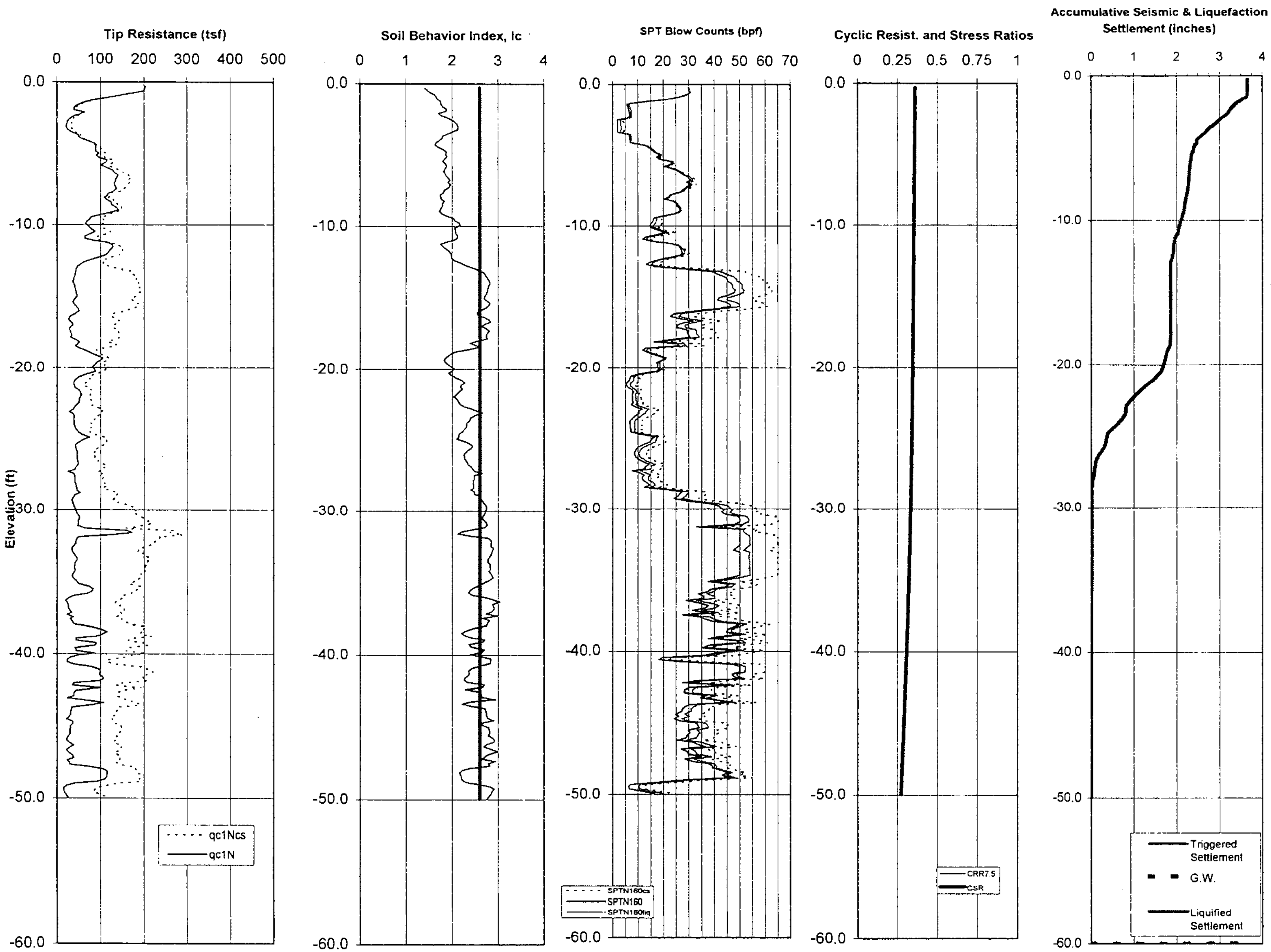
Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 0.88

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Fines Content (%)	Avg. Dr (%)	Spt N160cs (bpf)	Cyclic Shear Stress τav (psf)	Gmax (ksf)	γeff / (Geff/Gmax)	γeff	Vol. Strain (%)	"Dry" Settle. (in)
1	2.62	2.30	56.9	0.23	92.7	0.3	0.092	Sands - clean sand to silty sand (6)	1.00	1.09	1	7.1	78	14.5	82	4.03E+02	1.97E-04	3.69E-03	1.92E+00	5.31E-01
2	3.28	0.66	14.3	0.01	23.3	0.1	0.184	Sand mixtures - silty sand to sandy silt (5)	0.99	1.47	1	16.0	24	4.9	164	4.94E+02	3.32E-04	3.66E-03	3.01E+00	2.37E-01
3	9.68	6.40	71.1	0.65	104.9	0.8	0.403	Sands - clean sand to silty sand (6)	0.99	1.15	1	9.2	71	21.5	357	1.14E+03	3.13E-04	5.59E-03	9.79E-01	7.52E-01
4	10.01	0.33	54.1	0.84	67.7	1.6	0.613	Sand mixtures - silty sand to sandy silt (5)	0.98	1.59	1	17.9	52	19.5	539	1.42E+03	3.78E-04	4.32E-03	7.94E-01	3.13E-02
5	10.17	0.16	59.2	0.70	73.0	1.2	0.628	Sands - clean sand to silty sand (6)	0.98	1.39	1	14.7	55	18.1	551	1.40E+03	3.93E-04	5.20E-03	1.05E+00	2.07E-02
6	10.83	0.66	63.3	0.88	76.7	1.4	0.653	Sand mixtures - silty sand to sandy silt (5)	0.98	1.45	1	15.7	57	20.6	573	1.49E+03	3.85E-04	4.20E-03	7.66E-01	6.03E-02
7	12.30	1.48	93.7	1.07	108.0	1.1	0.719	Sands - clean sand to silty sand (6)	0.98	1.22	1	11.0	70	24.7	629	1.63E+03	3.87E-04	3.75E-03	6.14E-01	1.09E-01
8	12.96	0.66	56.2	1.00	62.2	1.9	0.785	Sand mixtures - silty sand to sandy silt (5)	0.97	1.85	1	20.8	47	20.4	685	1.65E+03	4.15E-04	3.58E-03	6.42E-01	5.06E-02
9	13.12	0.16	40.7	1.61	44.3	4.0	0.809	Silt mixtures - clayey silt to silty clay (4)	0.97	3.21	1	34.4	---	38.7	706	2.87E+03	2.46E-04	4.62E-04	2.79E-02	5.50E-04
11	16.40	0.66	43.9	1.88	43.2	4.4	0.991	Silt mixtures - clayey silt to silty clay (4)	0.97	3.45	1	36.2	---	40.8	429	1.58E+03	1.36E-04	2.51E-04	1.77E-02	1.39E-03
15	18.04	0.16	47.3	1.56	44.1	3.4	1.104	Silt mixtures - clayey silt to silty clay (4)	0.96	2.91	0.98	32.0	---	31.2	953	3.12E+03	3.05E-04	5.71E-04	5.50E-02	1.08E-03
16	18.21	0.16	55.3	1.38	51.3	2.5	1.114	Sand mixtures - silty sand to sandy silt (5)	0.96	2.30	0.98	26.3	39	22.8	961	2.08E+03	4.62E-04	2.91E-03	4.48E-01	8.82E-03
17	18.54	0.33	48.8	1.74	44.9	3.7	1.129	Silt mixtures - clayey silt to silty clay (4)	0.96	3.02	0.97	32.8	---	35.4	973	3.30E+03	2.95E-04	5.34E-04	4.06E-02	1.60E-03
18	18.70	0.16	55.2	1.16	50.5	2.2	1.144	Sand mixtures - silty sand to sandy silt (5)	0.96	2.13	0.97	24.5	39	18.8	985	1.98E+03	4.99E-04	3.83E-03	7.42E-01	1.46E-02
19	20.51	1.80	95.8	0.88	85.5	0.9	1.205	Sands - clean sand to silty sand (6)	0.96	1.24	0.94	11.6	60	18.4	1035	1.97E+03	5.28E-04	4.85E-03	1.01E+00	2.19E-01
20	21.82	1.31	52.5	0.49	45.1	1.0	1.301	Sand mixtures - silty sand to sandy silt (5)	0.95	1.63	0.94	18.3	33	10.4	1113	1.74E+03	6.39E-04	8.70E-03	3.13E+00	4.94E-01
21	21.98	0.16	66.7	0.49	56.3	0.7	1.345	Sands - clean sand to silty sand (6)	0.95	1.35	0.93	14.0	43	10.2	1148	1.74E+03	6.58E-04	9.33E-03	3.23E+00	6.35E-02
22	22.80	0.82	57.0	0.68	47.6	1.2	1.375	Sand mixtures - silty sand to sandy silt (5)	0.95	1.75	0.93	19.8	36	12.1	1172	1.88E+03	6.23E-04	7.75E-03	2.50E+00	2.46E-01
23	23.29	0.49	43.7	1.15	34.0	2.7	1.414	Silt mixtures - clayey silt to silty clay (4)	0.95	3.08	0.93	33.2	---	17.4	802	1.94E+03	2.75E-04	5.66E-04	1.27E-01	7.51E-03
24	27.07	3.77	59.4	0.99	46.8	1.7	1.542	Sand mixtures - silty sand to sandy silt (5)	0.94	2.02	0.91	23.1	34	15.1	1301	2.15E+03	6.09E-04	5.63E-03	1.60E+00	7.23E-01
25	27.40	0.33	49.4	1.28	33.4	2.7	1.665	Silt mixtures - clayey silt to silty clay (4)	0.93	3.14	0.9	33.7	---	16.7	695	1.63E+03	2.13E-04	4.44E-04	8.38E-02	3.30E-03
26	27.56	0.16	51.6	1.20	39.0	2.4	1.680	Sand mixtures - silty sand to sandy silt (5)	0.93	2.61	0.9	29.3	27	17.0	1405	2.37E+03	5.93E-04	3.66E-03	8.12E-01	1.60E-02
27	27.72	0.16	53.3	1.35	40.1	2.6	1.690	Silt mixtures - clayey silt to silty clay (4)	0.93	2.68	0.9	29.9	---	18.9	1412	3.27E+03	4.31E-04	9.10E-04	1.77E-01	3.48E-03
28	28.05	0.33	54.7	1.30	41.0	2.4	1.705	Sand mixtures - silty sand to sandy silt (5)	0.93	2.56	0.9	28.8	29	18.0	1422	2.43E+03	5.86E-04	3.40E-03	7.04E-01	2.77E-02
29	28.38	0.33	57.9	1.51	43.1	2.7	1.724	Silt mixtures - clayey silt to silty clay (4)	0.93	2.61	0.89	29.2	---	20.8	1437	3.41E+03	4.21E-04	8.76E-04	1.47E-01	5.79E-03
30	28.54	0.16	60.8	1.38	45.1	2.3	1.739	Sand mixtures - silty sand to sandy silt (5)	0.93	2.36	0.9	26.9	33	18.5	1447	2.46E+03	5.88E-04	3.34E-03	6.64E-01	1.31E-02
31	29.53	0.98	67.7	2.58	41.2	3.9	1.774	Silt mixtures - clayey silt to silty clay (4)	0.92	3.33	0.89	35.2	---	35.1	486	1.32E+03	1.23E-04	2.30E-04	2.21E-02	2.61E-03
33	31.33	0.16	92.0	3.27	65.2	3.6	1.907	Silt mixtures - clayey silt to silty clay (4)	0.91	2.44	0.88	27.7	---	42.3	1562	4.54E+03	3.44E-04	6.28E-04	3.40E-02	6.70E-04
34	31.66	0.33	236.2	8.36	166.7	3.6	1.922	Very stiff sand to clayey sand* (8)	0.91	1.60	0.82	18.0	87	56.5	1572	3.55E+03	4.43E-04	1.41E-03	7.80E-02	3.07E-03
37	35.60	0.49	115.0	4.97	69.0	4.4	2.154	Silt mixtures - clayey silt to silty clay (4)	0.89	2.73	0.86	30.2	---	53.9	1145	3.54E+03	2.16E-04	3.75E-04	2.03E-02	1.20E-03
38	35.76	0.16	121.7	4.24	80.8	3.5	2.174	Sand mixtures - silty sand to sandy silt (5)	0.89	2.16	0.86	24.9	57	47.3	1726	3.68E+03	4.69E-04	1.27E-03	6.97E-02	1.37E-03
39	35.93	0.16	105.9	3.98	70.1	3.8	2.183	Silt mixtures - clayey silt to silty clay (4)	0.88	2.43	0.79	27.5	---	47.0	1731	5.E+03	3.44E-04	6.14E-04	3.32E-02	6.54E-04
44	38.71	0.49	167.9	5.42	107.4	3.3	2.341	Sand mixtures - silty sand to sandy silt (5)	0.86	1.82	0.78	20.9	68	53.7	1813	4.E+03	4.60E-04	1.15E-03	6.32E-02	3.73E-03
47	39.37	0.33	125.6	5.60	67.5	4.6	2.386	Silt mixtures - clayey silt to silty clay (4)	0.86	2.96	0.84	31.8	—	54.4	918	3.E+03	1.62E-04	2.43E-04	1.31E-02	5.17E-04
50	40.19	0.49	122.3	5.76	59.5	4.8	2.431	Silt mixtures - clayey silt to silty clay (4)	0.85	3.17	0.84	33.8	---	53.6	618	2.E+03	1.07E-04	1.54E-04	8.32E-03	4.92E-04
52	41.01	0.33	114.5	4.77	59.8	4.2	2.485	Silt mixtures - clayey silt to silty clay (4)	0.84	2.97	0.83	32.2	---	47.7	940	3.E+03	1.63E-04	2.18E-04	1.18E-02	4.63E-04
54	41.83	0.33	171.5	5.78	105.4	3.4	2.535	Sand mixtures - silty sand to sandy silt (5)	0.84	1.87	0.78	21.6	68	55.4	1899	4152.000	4.57E-04	1.06E-03	5.82E-02	2.29E-03
55	41.99	0.16	163.7	6.24	100.3	3.9	2.550	Very stiff sand to clayey sand* (8)	0.83	2.05	0.72	23.6	66	59.4	1905	4272.060	4.46E-04	0.0	0.05488	0.00108
57	42.49	0.16	165.3	5.64	100.7	3.5	2.580	Sand mixtures - silty sand to sandy silt (5)	0.83	1.92	0.83	22.2	66	54.3	1917	4170.580	4.60E-04	0.0	0.05799	0.001142
60	43.31	0.16	113.5	4.05	68.5	3.7	2.629	Silt mixtures - clayey silt to silty clay (4)	0.82	2.39	0.82	27.2	---	43.8	1935	5399.270	3.58E-04	0.0	0.02062	0.000406
61	43.47	0.16	178.9	5.12	107.8	2.9	2.639	Sand mixtures - silty sand to sandy silt (5)	0.82	1.70	0.82	19.5	68	47.1	1938	4013.450	4.83E-04	0.0	0.06381	0.001256
62	43.64	0.16	134.9	5.54	81.1	4.2	2.649	Silt mixtures - clayey silt to silty clay (4)	0.82	2.37	0.71	27.0	---	56.7	1942	5906.430	3.29E-04	0.0	0.01826	0.00036
70	47.90	0.16	127.4	4.67	73.2	3.7	2.905	Silt mixtures - clayey silt to silty clay (4)	0.77	2.34	0.81	26.7	---	47.1	2017	5813.860	3.47E-04	0.0	0.01096	0.000216
71	48.88	0.98	191.5	5.65	109.3	3.0	2.939	Sand mixtures - silty sand to sandy silt (5)	0.77	1.73	0.7	19.7	69	49.5	2026.00	4312.760	4.70E-04	0.0	0.05849	0.006908

Evaluation of Liquefac. Resistance of Soils
Using CPT Data

C:\JOBS\8953_Everest Terrace\6-29-04\CPT-02.cpd

CPT:	CPT2			Fill Height:	0.0 ft. 125 pcf
G.W. Depth:	60 ft.	Elev.:	0	Max horizontal acc. @ surface:	0.69 g
Design G.W. Depth:	60 ft.			Design earthquake magnitude:	6.9
		Ic:	2.6 (Standard Value)	Removal:	0 ft



GEOLABS-WESTLAKE VILLAGE

*Exclude Settlement from layers thinner than 6 inches

CPT ANALYSIS
CPT 2
DESIGN GROUNDWATER AT 40 FOOT DEPTH

GEOLABS-WESTLAKE VILLAGE

"Dry" Sand Seismic Settlement Using CPT Data

3.66

CPT: CPT2
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 0.88

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Fines Content (%)	Avg. Dr (%)	Spt N160cs (bpf)	Cyclic Shear Stress τav (psf)	Gmax (ksf)	γeff / (Geff/Gmax)	γeff	Vol. Strain (%)	"Dry" Settle. (in)
1	2.62	2.30	56.9	0.23	92.7	0.3	0.092	Sands - clean sand to silty sand (6)	1.00	1.09	1	7.1	78	14.5	82	4.03E+02	1.97E-04	3.69E-03	1.92E+00	5.31E-01
2	3.28	0.66	14.3	0.01	23.3	0.1	0.184	Sand mixtures - silty sand to sandy silt (5)	0.99	1.47	1	16.0	24	4.9	164	4.94E+02	3.32E-04	3.66E-03	3.01E+00	2.37E-01
3	9.68	6.40	71.1	0.65	104.9	0.8	0.403	Sands - clean sand to silty sand (6)	0.99	1.15	1	9.2	71	21.5	357	1.14E+03	3.13E-04	5.59E-03	9.79E-01	7.52E-01
4	10.01	0.33	54.1	0.84	67.7	1.6	0.613	Sand mixtures - silty sand to sandy silt (5)	0.98	1.59	1	17.9	52	19.5	539	1.42E+03	3.78E-04	4.32E-03	7.94E-01	3.13E-02
5	10.17	0.16	59.2	0.70	73.0	1.2	0.628	Sands - clean sand to silty sand (6)	0.98	1.39	1	14.7	55	18.1	551	1.40E+03	3.93E-04	5.20E-03	1.05E+00	2.07E-02
6	10.83	0.66	63.3	0.88	76.7	1.4	0.653	Sand mixtures - silty sand to sandy silt (5)	0.98	1.45	1	15.7	57	20.6	573	1.49E+03	3.85E-04	4.20E-03	7.66E-01	6.03E-02
7	12.30	1.48	93.7	1.07	108.0	1.1	0.719	Sands - clean sand to silty sand (6)	0.98	1.22	1	11.0	70	24.7	629	1.63E+03	3.87E-04	3.75E-03	6.14E-01	1.09E-01
8	12.96	0.66	56.2	1.00	62.2	1.9	0.785	Sand mixtures - silty sand to sandy silt (5)	0.97	1.85	1	20.8	47	20.4	685	1.65E+03	4.15E-04	3.58E-03	6.42E-01	5.06E-02
9	13.12	0.16	40.7	1.61	44.3	4.0	0.809	Silt mixtures - clayey silt to silty clay (4)	0.97	3.21	1	34.4	---	38.7	706	2.87E+03	2.46E-04	4.62E-04	2.79E-02	5.50E-04
11	16.40	0.66	43.9	1.88	43.2	4.4	0.991	Silt mixtures - clayey silt to silty clay (4)	0.97	3.45	1	36.2	---	40.8	429	1.58E+03	1.36E-04	2.51E-04	1.77E-02	1.39E-03
15	18.04	0.16	47.3	1.56	44.1	3.4	1.104	Silt mixtures - clayey silt to silty clay (4)	0.96	2.91	0.98	32.0	---	31.2	953	3.12E+03	3.05E-04	5.71E-04	5.50E-02	1.08E-03
16	18.21	0.16	55.3	1.38	51.3	2.5	1.114	Sand mixtures - silty sand to sandy silt (5)	0.96	2.30	0.98	26.3	39	22.8	961	2.08E+03	4.62E-04	2.91E-03	4.48E-01	8.82E-03
17	18.54	0.33	48.8	1.74	44.9	3.7	1.129	Silt mixtures - clayey silt to silty clay (4)	0.96	3.02	0.97	32.8	---	35.4	973	3.30E+03	2.95E-04	5.34E-04	4.06E-02	1.60E-03
18	18.70	0.16	55.2	1.16	50.5	2.2	1.144	Sand mixtures - silty sand to sandy silt (5)	0.96	2.13	0.97	24.5	39	18.8	985	1.98E+03	4.99E-04	3.83E-03	7.42E-01	1.46E-02
19	20.51	1.80	95.8	0.88	85.5	0.9	1.205	Sands - clean sand to silty sand (6)	0.96	1.24	0.94	11.6	60	18.4	1035	1.97E+03	5.28E-04	4.85E-03	1.01E+00	2.19E-01
20	21.82	1.31	52.5	0.49	45.1	1.0	1.301	Sand mixtures - silty sand to sandy silt (5)	0.95	1.63	0.94	18.3	33	10.4	1113	1.74E+03	6.39E-04	8.70E-03	3.13E+00	4.94E-01
21	21.98	0.16	66.7	0.49	56.3	0.7	1.345	Sands - clean sand to silty sand (6)	0.95	1.35	0.93	14.0	43	10.2	1148	1.74E+03	6.58E-04	9.33E-03	3.23E+00	6.35E-02
22	22.80	0.82	57.0	0.68	47.6	1.2	1.375	Sand mixtures - silty sand to sandy silt (5)	0.95	1.75	0.93	19.8	36	12.1	1172	1.88E+03	6.23E-04	7.75E-03	2.50E+00	2.46E-01
23	23.29	0.49	43.7	1.15	34.0	2.7	1.414	Silt mixtures - clayey silt to silty clay (4)	0.95	3.08	0.93	33.2	---	17.4	802	1.94E+03	2.75E-04	5.66E-04	1.27E-01	7.51E-03
24	27.07	3.77	59.4	0.99	46.8	1.7	1.542	Sand mixtures - silty sand to sandy silt (5)	0.94	2.02	0.91	23.1	34	15.1	1301	2.15E+03	6.09E-04	5.63E-03	1.60E+00	7.23E-01
25	27.40	0.33	49.4	1.28	33.4	2.7	1.665	Silt mixtures - clayey silt to silty clay (4)	0.93	3.14	0.9	33.7	---	16.7	695	1.63E+03	2.13E-04	4.44E-04	8.38E-02	3.30E-03
26	27.56	0.16	51.6	1.20	39.0	2.4	1.680	Sand mixtures - silty sand to sandy silt (5)	0.93	2.61	0.9	29.3	27	17.0	1405	2.37E+03	5.93E-04	3.66E-03	8.12E-01	1.60E-02
27	27.72	0.16	53.3	1.35	40.1	2.6	1.690	Silt mixtures - clayey silt to silty clay (4)	0.93	2.68	0.9	29.9	---	18.9	1412	3.27E+03	4.31E-04	9.10E-04	1.77E-01	3.48E-03
28	28.05	0.33	54.7	1.30	41.0	2.4	1.705	Sand mixtures - silty sand to sandy silt (5)	0.93	2.56	0.9	28.8	29	18.0	1422	2.43E+03	5.86E-04	3.40E-03	7.04E-01	2.77E-02
29	28.38	0.33	57.9	1.51	43.1	2.7	1.724	Silt mixtures - clayey silt to silty clay (4)	0.93	2.61	0.89	29.2	---	20.8	1437	3.41E+03	4.21E-04	8.76E-04	1.47E-01	5.79E-03
30	28.54	0.16	60.8	1.38	45.1	2.3	1.739	Sand mixtures - silty sand to sandy silt (5)	0.93	2.36	0.9	26.9	33	18.5	1447	2.46E+03	5.88E-04	3.34E-03	6.64E-01	1.31E-02
31	29.53	0.98	67.7	2.58	41.2	3.9	1.774	Silt mixtures - clayey silt to silty clay (4)	0.92	3.33	0.89	35.2	---	35.1	486	1.32E+03	1.23E-04	2.30E-04	2.21E-02	2.61E-03
33	31.33	0.16	92.0	3.27	65.2	3.6	1.907	Silt mixtures - clayey silt to silty clay (4)	0.91	2.44	0.88	27.7	---	42.3	1562	4.54E+03	3.44E-04	6.28E-04	3.40E-02	6.70E-04
34	31.66	0.33	236.2	8.36	166.7	3.6	1.922	Very stiff sand to clayey sand* (8)	0.91	1.60	0.82	18.0	87	56.5	1572	3.55E+03	4.43E-04	1.41E-03	7.80E-02	3.07E-03
37	35.60	0.49	115.0	4.97	69.0	4.4	2.154	Silt mixtures - clayey silt to silty clay (4)	0.89	2.73	0.86	30.2	---	53.9	1145	3.54E+03	2.16E-04	3.75E-04	2.03E-02	1.20E-03
38	35.76	0.16	121.7	4.24	80.8	3.5	2.174	Sand mixtures - silty sand to sandy silt (5)	0.89	2.16	0.86	24.9	57	47.3	1726	3.68E+03	4.69E-04	1.27E-03	6.97E-02	1.37E-03
39	35.93	0.16	105.9	3.98	70.1	3.8	2.183	Silt mixtures - clayey silt to silty clay (4)	0.88	2.43	0.79	27.5	---	47.0	1731	5.E+03	3.44E-04	6.14E-04	3.32E-02	6.54E-04
44	38.71	0.49	167.9	5.42	107.4	3.3	2.341	Sand mixtures - silty sand to sandy silt (5)	0.86	1.82	0.78	20.9	68	53.7	1813	4.E+03	4.60E-04	1.15E-03	6.32E-02	3.73E-03
47	39.37	0.33	125.6	5.60	67.5	4.6	2.386	Silt mixtures - clayey silt to silty clay (4)	0.86	2.96	0.84	31.8	---	54.4	918	3.E+03	1.62E-04	2.43E-04	1.31E-02	5.17E-04
50	40.19	0.30	122.3	5.76	59.5	4.8	2.431	Silt mixtures - clayey silt to silty clay (4)	0.85	3.17	0.84	33.8	---	53.6	618	2.E+03	1.07E-04	1.54E-04	8.30E-03	4.90E-04

Liquefaction Analysis Using CPT Data

CPT: CPT2
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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0
3.66

Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 1.24

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Avg. Fines Content (%)	Avg. Dr (%)	Avg. SPT N1(60)liq (bpf)	Min. CRR (M=7.5)	Avg. CSR	Min. Liq. FS	Avg. Strain (%)	Liq Settle. (in)
50	40.19	0.19	122.3	5.76	59.5	4.8	2.431	Silt mixtures - clayey silt to silty clay (4)	0.85	3.17	0.84	33.8	---	66.7	Infin	0.308	Infin	0.0	0
52	41.01	0.33	114.5	4.77	59.8	4.2	2.485	Silt mixtures - clayey silt to silty clay (4)	0.84	2.97	0.84	32.2	---	39.8	Infin	0.309	Infin	0.0	0
53	41.50	0.49	159.0	6.55	98.2	4.2	2.510	Very stiff, fine grained* (9)	0.84	2.16	0.83	24.9	---	52.1	Infin	0.309	Infin	0.0	0
54	41.83	0.33	171.5	5.78	105.4	3.4	2.535	Sand mixtures - silty sand to sandy silt (5)	0.84	1.87	0.78	21.6	68	49.1	Infin	0.309	Infin	0.0	0
55	41.99	0.16	163.7	6.24	100.3	3.9	2.550	Very stiff sand to clayey sand* (8)	0.83	2.05	0.73	23.6	66	52.0	Infin	0.309	Infin	0.0	0
57	42.49	0.16	165.3	5.64	100.7	3.5	2.580	Sand mixtures - silty sand to sandy silt (5)	0.83	1.92	0.83	22.2	66	47.9	Infin	0.309	Infin	0.0	0
60	43.31	0.16	113.5	4.05	68.5	3.7	2.629	Silt mixtures - clayey silt to silty clay (4)	0.82	2.39	0.83	27.2	---	37.0	Infin	0.309	Infin	0.0	0
61	43.47	0.16	178.9	5.12	107.8	2.9	2.639	Sand mixtures - silty sand to sandy silt (5)	0.82	1.70	0.83	19.5	68	42.2	Infin	0.309	Infin	0.0	0
62	43.64	0.16	134.9	5.54	81.1	4.2	2.649	Silt mixtures - clayey silt to silty clay (4)	0.82	2.37	0.72	27.0	---	48.5	Infin	0.309	Infin	0.0	0
70	47.90	0.16	127.4	4.67	73.2	3.7	2.905	Silt mixtures - clayey silt to silty clay (4)	0.77	2.34	0.82	26.7	---	40.0	Infin	0.306	Infin	0.0	0
71	48.88	0.98	191.5	5.65	109.3	3.0	2.939	Sand mixtures - silty sand to sandy silt (5)	0.77	1.73	0.73	19.7	69	44.3	Infin	0.306	Infin	0.0	0

Evaluation of Liquefa[illegible]n Resistance of Soils Using CPT Data

CPT: CPT2
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.

Elev.: 0

Ic: 2.6 (Standard Value)

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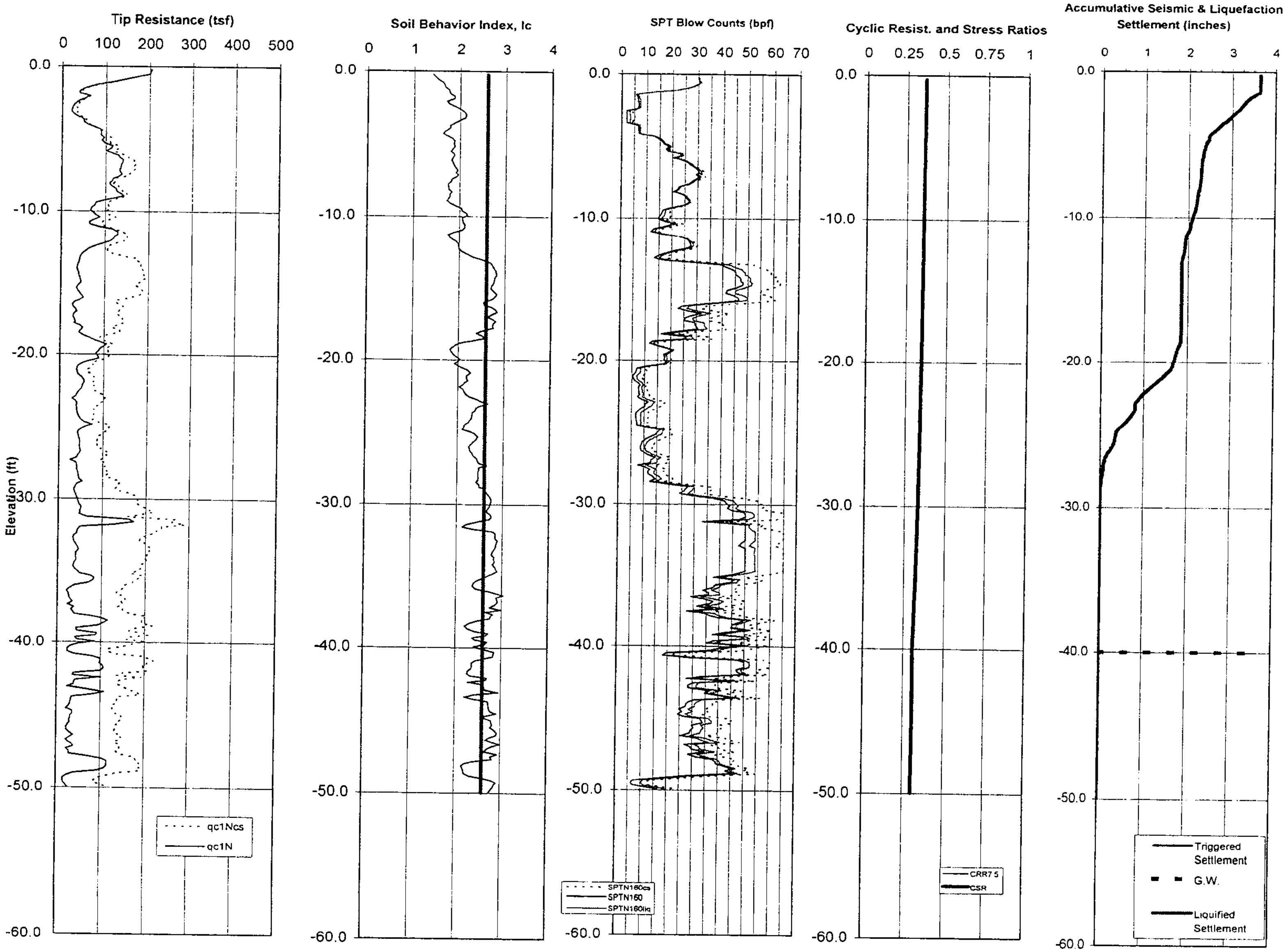
Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.9
Removal: 0 ft



*Exclude Settlement from layers thinner than 6 inches

GEOLABS-WESTLAKE VILLAGE

CPT ANALYSIS
CPT 3

Summary of Analysis of CPT Data

CPT: CPT3
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Est. Wet Den. (pcf)	Soil Behavior Type	O.B. (tsf)	Eff. O.B. (tsf)	Norm. Exp. n	Ic Range	Kc	Fines Content (%)	Phi (deg)	Dr (%)	Su (tsf)	SPT N1(60) (bpf)	SPT N1(60)cs (bpf)
1	0.49	0.16	277.3	1.09	451.5	0.4	125	Gravelly sand to dense sand	0.015	0.015	0.5	1.15	1.00	0.0	60	100.0	---	50.0	50
2	0.98	0.49	167.2	2.00	272.3	1.3	125	Sands - clean sand to silty sand	0.046	0.046	0.5	1.46 to 1.92	1.08	6.3	55	100.0	---	48.7	49.5
3	2.30	1.31	44.3	0.66	72.2	1.3	120	Sand mixtures - silty sand to sandy silt	0.101	0.101	0.5	2.04 to 2.29	1.52	16.7	47	79.0	---	15.6	19.3
4	2.46	0.16	10.1	0.11	16.4	1.1	120	Silt mixtures - clayey silt to silty clay	0.145	0.145	0.5	2.58	3.21	34.4	---	---	0.7	3.2	8.7
5	2.95	0.49	19.3	0.22	31.4	1.1	120	Sand mixtures - silty sand to sandy silt	0.165	0.165	0.5	2.24 to 2.49	2.17	24.7	41	38.0	---	5.4	10.1
6	9.35	6.40	197.7	2.90	302.7	1.4	125	Sands - clean sand to silty sand	0.380	0.380	0.5	1.6 to 2.03	1.06	6.6	48	98.0	---	46.4	46.8
7	10.33	0.98	85.8	2.42	107.8	2.9	120	Sand mixtures - silty sand to sandy silt	0.609	0.609	0.5	2.03 to 2.37	1.74	19.6	42	71.0	---	40.0	46.6
8	10.83	0.49	55.0	2.42	66.6	4.5	120	Silt mixtures - clayey silt to silty clay	0.653	0.653	0.5	2.42 to 2.57	2.73	30.3	---	---	3.6	41.9	53.3
9	13.62	2.79	63.1	4.79	78.5	7.7	125	Very stiff, fine grained*	0.755	0.755	0.5 to 1	2.55 to 2.69	3.52	36.8	---	---	4.2	50.0	64.9
10	13.94	0.33	96.3	2.68	102.0	2.9	120	Sand mixtures - silty sand to sandy silt	0.852	0.852	0.5	2.08 to 2.34	1.76	19.7	41	68.0	---	38.1	44.8
11	16.24	2.30	148.9	1.93	150.7	1.3	125	Sands - clean sand to silty sand	0.934	0.934	0.5	1.67 to 2.01	1.16	9.6	43	83.0	---	31.7	33.1
12	17.55	1.31	90.3	2.54	86.4	2.9	120	Sand mixtures - silty sand to sandy silt	1.045	1.045	0.5	2.15 to 2.37	1.88	21.5	40	61.0	---	32.7	39.5
13	19.85	2.30	184.0	1.67	168.0	0.9	125	Sands - clean sand to silty sand	1.156	1.156	0.5	1.63 to 1.9	1.05	6.6	43	88.0	---	29.5	29.9
14	21.49	1.64	60.7	1.00	52.6	1.7	120	Sand mixtures - silty sand to sandy silt	1.277	1.277	0.5	2.07 to 2.44	1.91	21.8	36	40.0	---	11.6	16.5
15	21.98	0.49	46.7	1.69	35.6	3.7	120	Silt mixtures - clayey silt to silty clay	1.341	1.341	0.5 to 1	2.53 to 2.7	3.48	36.5	---	---	3.0	20.4	29.3
16	22.15	0.16	80.6	1.75	67.6	2.2	120	Sand mixtures - silty sand to sandy silt	1.361	1.361	0.5	2.27	1.85	21.3	38	50.0	---	19.7	25.3
17	22.64	0.49	110.3	1.32	91.8	1.2	125	Sands - clean sand to silty sand	1.381	1.381	0.5	1.96 to 2.04	1.29	12.8	39	63.0	---	20.0	22.5
18	23.29	0.66	113.3	1.88	93.2	1.7	120	Sand mixtures - silty sand to sandy silt	1.416	1.416	0.5	2.06 to 2.13	1.43	15.3	39	64.0	---	23.6	27.4
19	24.61	1.31	156.1	2.18	125.7	1.4	125	Sands - clean sand to silty sand	1.477	1.477	0.5	1.87 to 2	1.23	11.4	41	76.0	---	27.9	30
20	26.57	1.97	98.8	1.84	77.1	1.9	120	Sand mixtures - silty sand to sandy silt	1.577	1.577	0.5	2.09 to 2.29	1.63	18.4	38	55.0	---	20.1	24.7
21	29.36	2.79	207.4	2.62	154.7	1.3	125	Sands - clean sand to silty sand	1.723	1.723	0.5	1.68 to 1.98	1.15	9.2	42	84.0	---	30.8	32.1
22	30.51	1.15	158.0	3.27	113.8	2.1	120	Sand mixtures - silty sand to sandy silt	1.845	1.845	0.5	2.04 to 2.15	1.44	15.6	40	71.0	---	31.1	35.3
23	32.64	2.13	212.1	2.48	148.9	1.2	125	Sands - clean sand to silty sand	1.946	1.946	0.5	1.74 to 2	1.14	8.9	41	82.0	---	29.5	30.7
24	33.96	1.31	127.6	2.19	87.2	1.7	120	Sand mixtures - silty sand to sandy silt	2.052	2.052	0.5	2.06 to 2.16	1.48	16.2	38	60.0	---	22.4	26.4
25	34.12	0.16	126.2	1.74	85.3	1.4	125	Sands - clean sand to silty sand	2.096	2.096	0.5	2.06	1.38	14.6	38	59.0	---	19.4	22.7
26	34.28	0.16	126.1	1.86	85.0	1.5	120	Sand mixtures - silty sand to sandy silt	2.106	2.106	0.5	2.08	1.42	15.2	38	59.0	---	20.0	23.5
27	34.45	0.16	130.2	1.84	87.6	1.4	125	Sands - clean sand to silty sand	2.116	2.116	0.5	2.06	1.38	14.5	38	60.0	---	20.3	23.6
28	34.61	0.16	137.5	2.08	92.3	1.5	120	Sand mixtures - silty sand to sandy silt	2.126	2.126	0.5	2.06	1.39	14.6	38	62.0	---	22.3	25.8
29	36.91	2.30	166.6	2.26	109.9	1.4	125	Sands - clean sand to silty sand	2.203	2.203	0.5	1.9 to 2.02	1.27	12.2	39	70.0	---	25.2	27.7
30	37.89	0.98	142.1	2.62	91.6	1.9	120	Sand mixtures - silty sand to sandy silt	2.304	2.304	0.5	2.06 to 2.17	1.50	16.4	38	62.0	---	24.5	28.8
31	47.57	9.68	180.6	2.27	109.0	1.3	125	Sands - clean sand to silty sand	2.636	2.636	0.5	1.79 to 2.03	1.25	11.7	39	69.0	---	24.5	26.7
32	47.74	0.16	191.4	3.30	109.2	1.8	120	Sand mixtures - silty sand to sandy silt	2.944	2.944	0.5	2.05	1.37	14.3	38	69.0	---	27.3	30.8
33	49.70	1.97	236.8	3.25	133.5	1.4	125	Sands - clean sand to silty sand	3.010	3.010	0.5	1.86 to 2.01	1.21	10.8	39	77.0	---	28.8	30.7

Evaluation of Soil Characteristics
Using CPT Data

CPT: CPT3
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Elev.: 0

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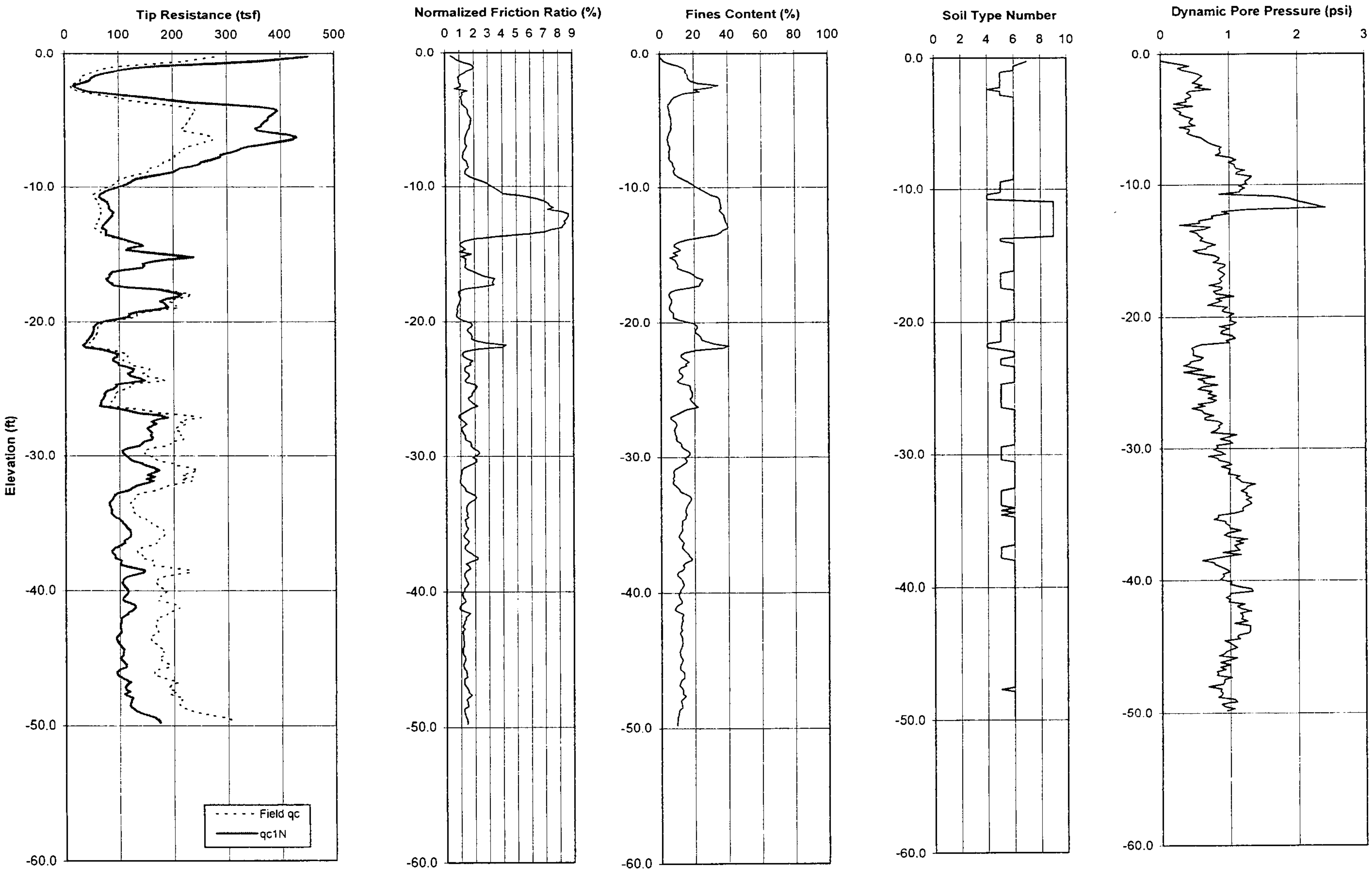

GEOLABS-WESTLAKE VILLAGE

CPT-Based Soil Behavior Type

CPT: CPT3

G.W. Depth: 60 ft

Ic: 2.6

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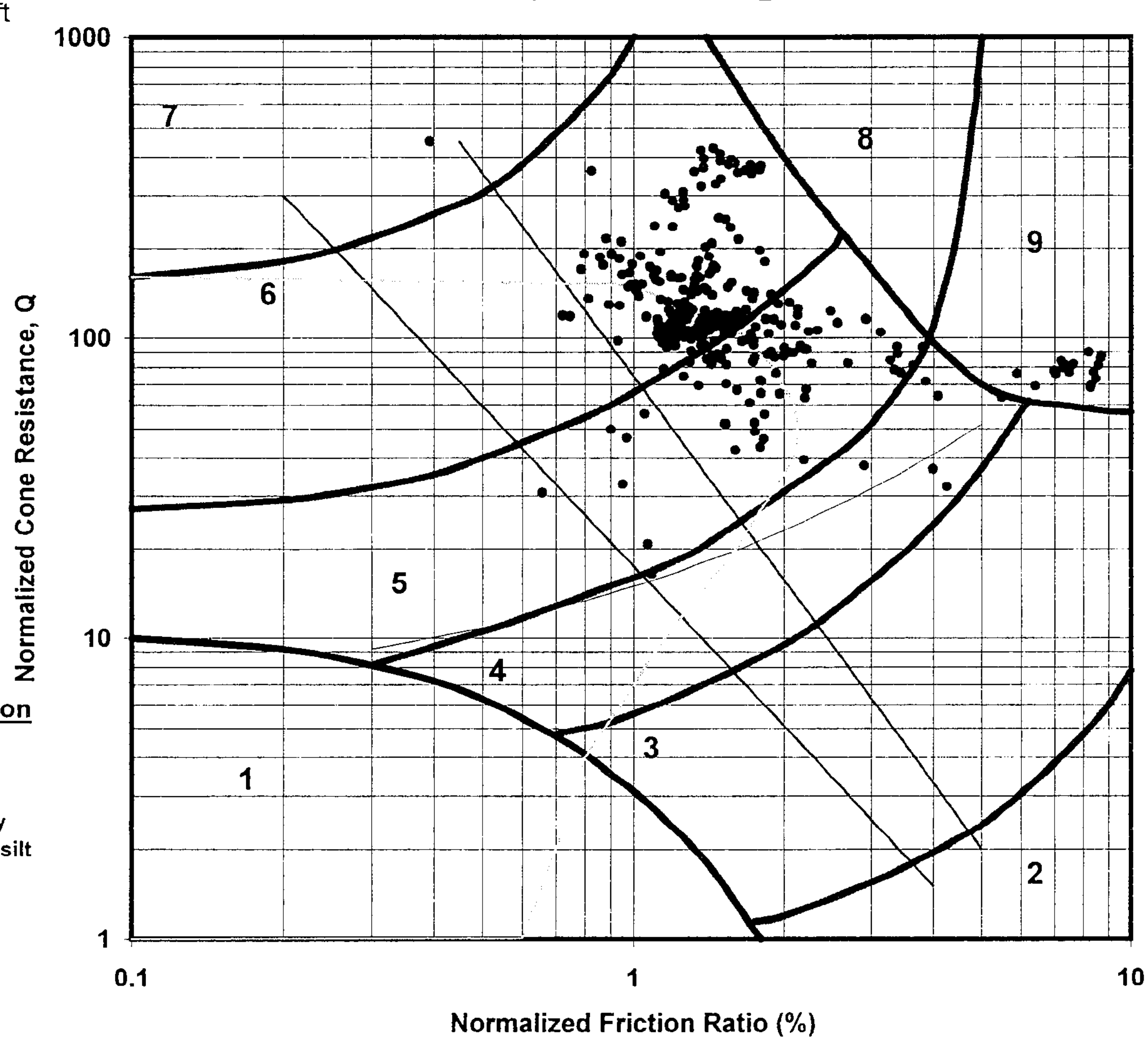

Soil Consistency Number

(Sand / Clay)

1 = very loose / very soft
2 = loose / soft
3 = medium dense / medium stiff
4 = dense / stiff
5 = very dense / very stiff
6 = --- / hard

Soil Behavior Type Classification

1. Sensitive Fine Grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

*Heavily overconsolidated or cemented

GEOLABS-WESTLAKE VILLAGE

Soil Characteristics and Engineering Characteristics Using CPT Data

CPT: CPT3
G.W. Depth: 60 ft.
Elev.: 0
Design G.W. Depth: 60 ft.

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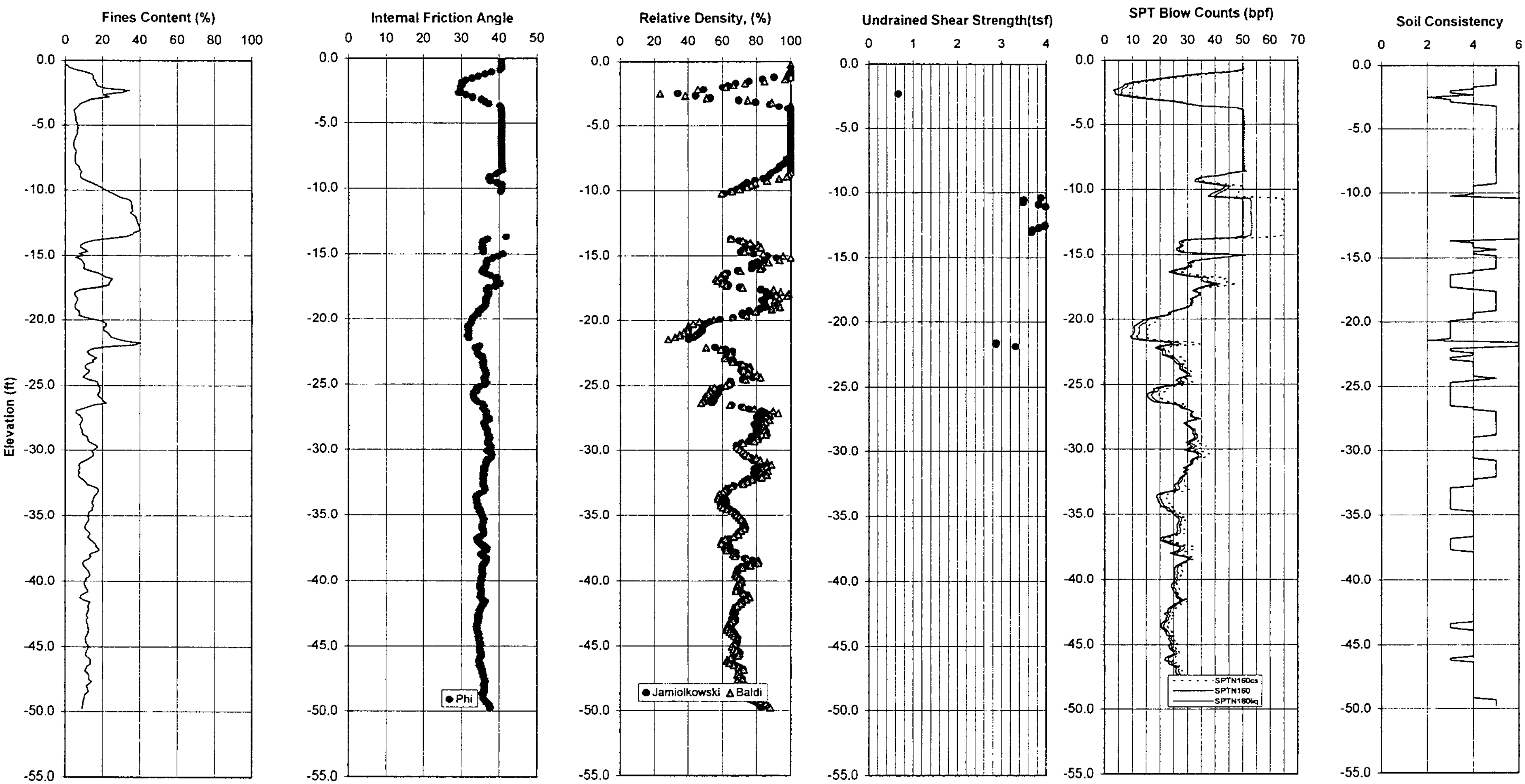

CPT ANALYSIS
CPT 3
NO DESIGN GROUNDWATER

GEOLABS-WESTLAKE VILLAGE

"Dry" Sand Seismic Settlement Using CPT Data

CPT: CPT3
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

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Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 0.88

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Fines Content (%)	Avg. Dr (%)	Spt N160cs (bpf)	Cyclic Shear Stress τav (psf)	Gmax (ksf)	γeff / (Geff/Gmax)	γeff	Vol. Strain (%)	"Dry" Settle. (in)
1	0.49	0.16	277.3	1.09	451.5	0.4	0.015	Gravelly sand to dense sand (7)	1.00	1.00	1	0.0	100	50.0	14	2.66E+02	5.19E-05	7.60E-05	4.07E-03	2.40E-04
2	0.98	0.49	167.2	2.00	272.3	1.3	0.046	Sands - clean sand to silty sand (6)	1.00	1.08	1	6.3	100	49.5	41	4.73E+02	8.68E-05	1.69E-04	9.09E-03	5.37E-04
3	2.30	1.31	44.3	0.66	72.2	1.3	0.101	Sand mixtures - silty sand to sandy silt (5)	1.00	1.52	1	16.7	79	19.3	90	5.14E+02	1.77E-04	1.97E-02	8.01E-01	1.26E-01
4	2.46	0.16	10.1	0.11	16.4	1.1	0.145	Silt mixtures - clayey silt to silty clay (4)	1.00	3.21	1	34.4	---	8.7	130	7.41E+02	1.75E-04	5.31E-04	2.86E-01	5.62E-03
5	2.95	0.49	19.3	0.22	31.4	1.1	0.165	Sand mixtures - silty sand to sandy silt (5)	1.00	2.17	1	24.7	38	10.1	147	5.85E+02	2.53E-04	4.51E-03	1.94E+00	1.14E-01
6	9.35	6.40	197.7	2.90	302.7	1.4	0.380	Sands - clean sand to silty sand (6)	0.99	1.06	1	6.6	98	46.8	336	1.39E+03	2.37E-04	8.15E-03	8.92E-02	6.84E-02
7	10.33	0.98	85.8	2.42	107.8	2.9	0.609	Sand mixtures - silty sand to sandy silt (5)	0.98	1.74	1	19.6	71	46.6	535	1.86E+03	2.88E-04	1.33E-03	7.91E-02	9.34E-03
8	10.83	0.49	55.0	2.42	66.6	4.5	0.653	Silt mixtures - clayey silt to silty clay (4)	0.98	2.73	1	30.3	---	53.3	573	2.87E+03	2.00E-04	3.41E-04	1.84E-02	1.09E-03
10	13.94	0.33	96.3	2.68	102.0	2.9	0.852	Sand mixtures - silty sand to sandy silt (5)	0.97	1.76	1	19.7	68	44.8	742	2.18E+03	3.44E-04	1.61E-03	1.23E-01	4.83E-03
11	16.24	2.30	148.9	1.93	150.7	1.3	0.934	Sands - clean sand to silty sand (6)	0.97	1.16	1	9.6	83	33.1	811	2.04E+03	3.99E-04	2.43E-03	2.40E-01	6.61E-02
12	17.55	1.31	90.3	2.54	86.4	2.9	1.045	Sand mixtures - silty sand to sandy silt (5)	0.96	1.88	0.99	21.5	61	39.5	904	2.35E+03	3.86E-04	1.73E-03	1.31E-01	2.06E-02
13	19.85	2.30	184.0	1.67	168.0	0.9	1.156	Sands - clean sand to silty sand (6)	0.96	1.05	0.95	6.6	88	29.9	995	2.20E+03	4.54E-04	2.94E-03	3.31E-01	9.12E-02
14	21.49	1.64	60.7	1.00	52.6	1.7	1.277	Sand mixtures - silty sand to sandy silt (5)	0.95	1.91	0.94	21.8	40	16.5	1094	2.00E+03	5.48E-04	5.04E-03	1.14E+00	2.24E-01
15	21.98	0.49	46.7	1.69	35.6	3.7	1.341	Silt mixtures - clayey silt to silty clay (4)	0.95	3.48	0.94	36.5	---	29.3	379	9.94E+02	1.27E-04	2.51E-04	4.26E-02	2.51E-03
16	22.15	0.16	80.6	1.75	67.6	2.2	1.361	Sand mixtures - silty sand to sandy silt (5)	0.95	1.85	0.94	21.3	50	25.3	1161	2.36E+03	4.92E-04	3.03E-03	4.12E-01	8.11E-03
17	22.64	0.49	110.3	1.32	91.8	1.2	1.381	Sands - clean sand to silty sand (6)	0.95	1.29	0.91	12.8	63	22.5	1177	2.25E+03	5.22E-04	3.93E-03	6.18E-01	3.65E-02
18	23.29	0.66	113.3	1.88	93.2	1.7	1.416	Sand mixtures - silty sand to sandy silt (5)	0.95	1.43	0.89	15.3	64	27.4	1205	2.43E+03	4.95E-04	3.05E-03	3.74E-01	2.95E-02
19	24.61	1.31	156.1	2.18	125.7	1.4	1.477	Sands - clean sand to silty sand (6)	0.95	1.23	0.87	11.4	76	30.0	1252	2.53E+03	4.94E-04	2.94E-03	3.05E-01	4.81E-02
20	26.57	1.97	98.8	1.84	77.1	1.9	1.577	Sand mixtures - silty sand to sandy silt (5)	0.94	1.63	0.87	18.4	55	24.7	1329	2.50E+03	5.32E-04	3.27E-03	4.80E-01	1.13E-01
21	29.36	2.79	207.4	2.62	154.7	1.3	1.723	Sands - clean sand to silty sand (6)	0.93	1.15	0.81	9.2	84	32.1	1437	2.78E+03	5.16E-04	2.72E-03	2.56E-01	8.55E-02
22	30.51	1.15	158.0	3.27	113.8	2.1	1.845	Sand mixtures - silty sand to sandy silt (5)	0.92	1.44	0.8	15.6	71	35.3	1523	3.02E+03	5.05E-04	2.09E-03	1.63E-01	2.25E-02
23	32.64	2.13	212.1	2.48	148.9	1.2	1.946	Sands - clean sand to silty sand (6)	0.91	1.14	0.77	8.9	82	30.7	1591	2.92E+03	5.44E-04	2.50E-03	2.52E-01	6.46E-02
24	33.96	1.31	127.6	2.19	87.2	1.7	2.052	Sand mixtures - silty sand to sandy silt (5)	0.90	1.48	0.79	16.2	60	26.4	1659	2.92E+03	5.68E-04	2.34E-03	3.07E-01	4.83E-02
25	34.12	0.16	126.2	1.74	85.3	1.4	2.096	Sands - clean sand to silty sand (6)	0.90	1.38	0.79	14.6	59	22.7	1686	2.82E+03	5.98E-04	2.61E-03	4.04E-01	7.96E-03
26	34.28	0.16	126.1	1.86	85.0	1.5	2.106	Sand mixtures - silty sand to sandy silt (5)	0.90	1.42	0.79	15.2	59	23.5	1692	2.86E+03	5.92E-04	2.50E-03	3.73E-01	7.34E-03
27	34.45	0.16	130.2	1.84	87.6	1.4	2.116	Sands - clean sand to silty sand (6)	0.89	1.38	0.79	14.5	60	23.6	1698	2.87E+03	5.92E-04	2.49E-03	3.69E-01	7.27E-03
28	34.61	0.16	137.5	2.08	92.3	1.5	2.126	Sand mixtures - silty sand to sandy silt (5)	0.89	1.39	0.79	14.6	62	25.8	1704	2.95E+03	5.77E-04	2.30E-03	3.06E-01	6.02E-03
29	36.91	2.30	166.6	2.26	109.9	1.4	2.203	Sands - clean sand to silty sand (6)	0.88	1.27	0.76	12.2	70	27.7	1748	3.05E+03	5.72E-04	2.17E-03	2.59E-01	7.14E-02
30	37.89	0.98	142.1	2.62	91.6	1.9	2.304	Sand mixtures - silty sand to sandy silt (5)	0.87	1.50	0.77	16.4	62	28.8	1803	3.19E+03	5.66E-04	1.91E-03	2.20E-01	2.60E-02
31	47.57	9.68	180.6	2.27	109.0	1.3	2.636	Sands - clean sand to silty sand (6)	0.82	1.25	0.72	11.7	69	26.7	1945	3.31E+03	5.88E-04	1.89E-03	2.39E-01	2.78E-01
32	47.74	0.16	191.4	3.30	109.2	1.8	2.944	Sand mixtures - silty sand to sandy silt (5)	0.78	1.37	0.68	14.3	69	30.8	2049	3.69E+03	5.56E-04	1.55E-03	1.52E-01	3.00E-03
33	49.70	1.97	236.8	3.25	133.5	1.4	3.010	Sands - clean sand to silty sand (6)	0.76	1.21	0.67	10.8	77	30.7	2065	4.E+03	5.60E-04	1.58E-03	1.59E-01	3.74E-02

Evaluation of Liquefa... Resistance of Soils
Using CPT Data

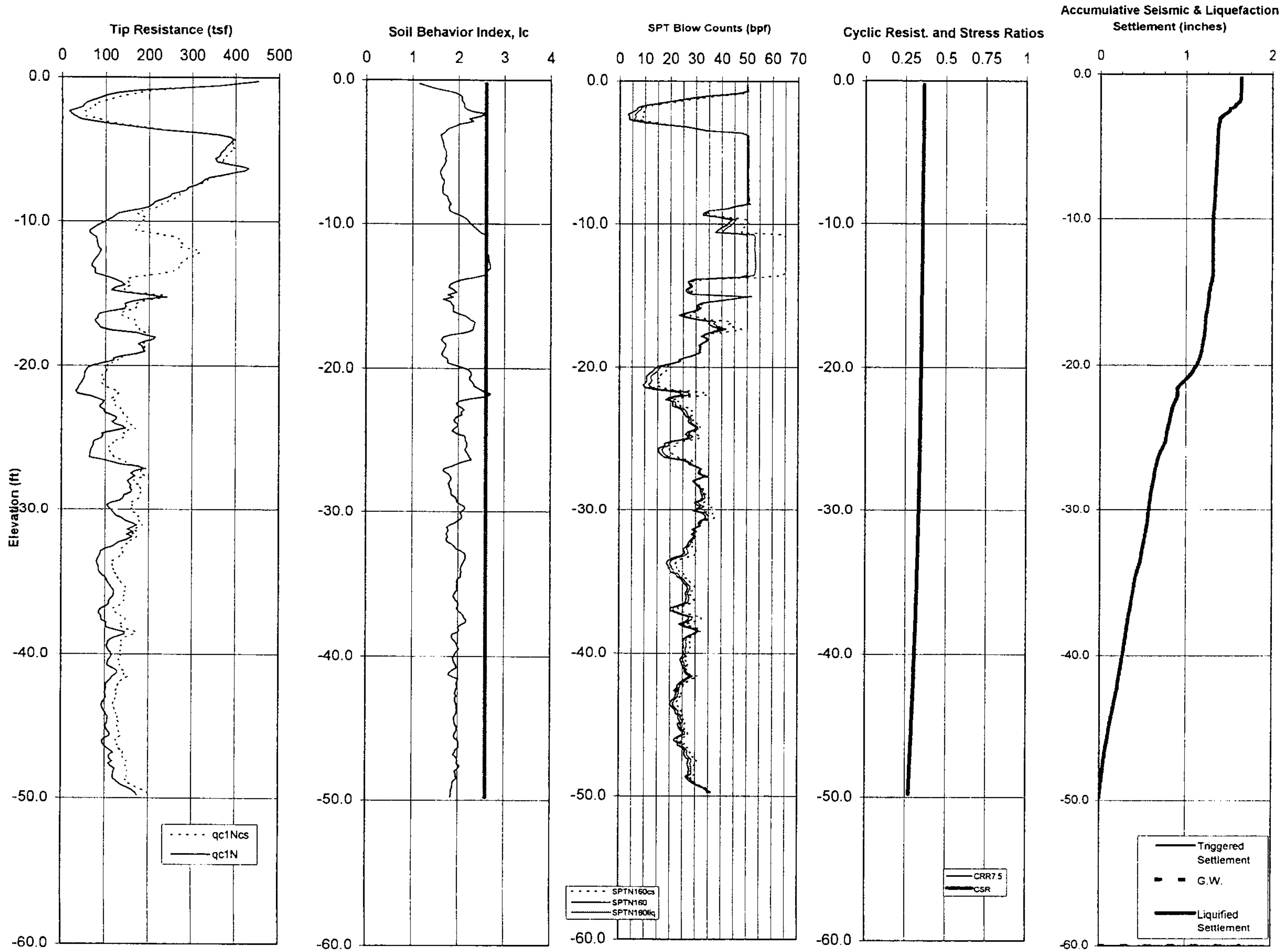
CPT: CPT3
G.W. Depth: 60 ft.
Design G.W. Depth: 60 ft.

Elev.: 0

Ic: 2.6 (Standard Value)

C:\JOBS\8953_Everest Terrace\6-29-04\CPT-03.cpd

Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.9
Removal: 0 ft



*Exclude Settlement from layers thinner than 6 inches

GEOLABS-WESTLAKE VILLAGE

CPT ANALYSIS
CPT 3
DESIGN GROUNDWATER AT 40 FOOT DEPTH

GEOLABS-WESTLAKE VILLAGE

"Dry" Sand Seismic Settlement Using CPT Data

CPT: CPT3 W.O.: 8953
G.W. Depth: 60 ft. Elev.: 0
Design G.W. Depth: 40 ft. Ic: 2.6 C \JOBS\8953_Everest Terrace\6-29-04\CPT-03 cpd
Removal: 0 ft.

Fill Height: **0.0** ft. **125** pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 0.88

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Fines Content (%)	Avg. Dr (%)	Spt N160cs (bpf)	Cyclic Shear Stress τav (psf)	Gmax (ksf)	γeff / (Geff/Gmax)	γeff	Vol. Strain (%)	"Dry" Settle. (in)
1	0.49	0.16	277.3	1.09	451.5	0.4	0.015	Gravelly sand to dense sand (7)	1.00	1.00	1	0.0	100	50.0	14	2.66E+02	5.19E-05	7.60E-05	4.07E-03	2.40E-04
2	0.98	0.49	167.2	2.00	272.3	1.3	0.046	Sands - clean sand to silty sand (6)	1.00	1.08	1	6.3	100	49.5	41	4.73E+02	8.68E-05	1.69E-04	9.09E-03	5.37E-04
3	2.30	1.31	44.3	0.66	72.2	1.3	0.101	Sand mixtures - silty sand to sandy silt (5)	1.00	1.52	1	16.7	79	19.3	90	5.14E+02	1.77E-04	1.97E-02	8.01E-01	1.26E-01
4	2.46	0.16	10.1	0.11	16.4	1.1	0.145	Silt mixtures - clayey silt to silty clay (4)	1.00	3.21	1	34.4	---	8.7	130	7.41E+02	1.75E-04	5.31E-04	2.86E-01	5.62E-03
5	2.95	0.49	19.3	0.22	31.4	1.1	0.165	Sand mixtures - silty sand to sandy silt (5)	1.00	2.17	1	24.7	38	10.1	147	5.85E+02	2.53E-04	4.51E-03	1.94E+00	1.14E-01
6	9.35	6.40	197.7	2.90	302.7	1.4	0.380	Sands - clean sand to silty sand (6)	0.99	1.06	1	6.6	98	46.8	336	1.39E+03	2.37E-04	8.15E-03	8.92E-02	6.84E-02
7	10.33	0.98	85.8	2.42	107.8	2.9	0.609	Sand mixtures - silty sand to sandy silt (5)	0.98	1.74	1	19.6	71	46.6	535	1.86E+03	2.88E-04	1.33E-03	7.91E-02	9.34E-03
8	10.83	0.49	55.0	2.42	66.6	4.5	0.653	Silt mixtures - clayey silt to silty clay (4)	0.98	2.73	1	30.3	—	53.3	573	2.87E+03	2.00E-04	3.41E-04	1.84E-02	1.09E-03
10	13.94	0.33	96.3	2.68	102.0	2.9	0.852	Sand mixtures - silty sand to sandy silt (5)	0.97	1.76	1	19.7	68	44.8	742	2.18E+03	3.44E-04	1.61E-03	1.23E-01	4.83E-03
11	16.24	2.30	148.9	1.93	150.7	1.3	0.934	Sands - clean sand to silty sand (6)	0.97	1.16	1	9.6	83	33.1	811	2.04E+03	3.99E-04	2.43E-03	2.40E-01	6.61E-02
12	17.55	1.31	90.3	2.54	86.4	2.9	1.045	Sand mixtures - silty sand to sandy silt (5)	0.96	1.88	0.99	21.5	61	39.5	904	2.35E+03	3.86E-04	1.73E-03	1.31E-01	2.06E-02
13	19.85	2.30	184.0	1.67	168.0	0.9	1.156	Sands - clean sand to silty sand (6)	0.96	1.05	0.95	6.6	88	29.9	995	2.20E+03	4.54E-04	2.94E-03	3.31E-01	9.12E-02
14	21.49	1.64	60.7	1.00	52.6	1.7	1.277	Sand mixtures - silty sand to sandy silt (5)	0.95	1.91	0.94	21.8	40	16.5	1094	2.00E+03	5.48E-04	5.04E-03	1.14E+00	2.24E-01
15	21.98	0.49	46.7	1.69	35.6	3.7	1.341	Silt mixtures - clayey silt to silty clay (4)	0.95	3.48	0.94	36.5	---	29.3	379	9.94E+02	1.27E-04	2.51E-04	4.26E-02	2.51E-03
16	22.15	0.16	80.6	1.75	67.6	2.2	1.361	Sand mixtures - silty sand to sandy silt (5)	0.95	1.85	0.94	21.3	50	25.3	1161	2.36E+03	4.92E-04	3.03E-03	4.12E-01	8.11E-03
17	22.64	0.49	110.3	1.32	91.8	1.2	1.381	Sands - clean sand to silty sand (6)	0.95	1.29	0.91	12.8	63	22.5	1177	2.25E+03	5.22E-04	3.93E-03	6.18E-01	3.65E-02
18	23.29	0.66	113.3	1.88	93.2	1.7	1.416	Sand mixtures - silty sand to sandy silt (5)	0.95	1.43	0.89	15.3	64	27.4	1205	2.43E+03	4.95E-04	3.05E-03	3.74E-01	2.95E-02
19	24.61	1.31	156.1	2.18	125.7	1.4	1.477	Sands - clean sand to silty sand (6)	0.95	1.23	0.87	11.4	76	30.0	1252	2.53E+03	4.94E-04	2.94E-03	3.05E-01	4.81E-02
20	26.57	1.97	98.8	1.84	77.1	1.9	1.577	Sand mixtures - silty sand to sandy silt (5)	0.94	1.63	0.87	18.4	55	24.7	1329	2.50E+03	5.32E-04	3.27E-03	4.80E-01	1.13E-01
21	29.36	2.79	207.4	2.62	154.7	1.3	1.723	Sands - clean sand to silty sand (6)	0.93	1.15	0.81	9.2	84	32.1	1437	2.78E+03	5.16E-04	2.72E-03	2.56E-01	8.55E-02
22	30.51	1.15	158.0	3.27	113.8	2.1	1.845	Sand mixtures - silty sand to sandy silt (5)	0.92	1.44	0.8	15.6	71	35.3	1523	3.02E+03	5.05E-04	2.09E-03	1.63E-01	2.25E-02
23	32.64	2.13	212.1	2.48	148.9	1.2	1.946	Sands - clean sand to silty sand (6)	0.91	1.14	0.77	8.9	82	30.7	1591	2.92E+03	5.44E-04	2.50E-03	2.52E-01	6.46E-02
24	33.96	1.31	127.6	2.19	87.2	1.7	2.052	Sand mixtures - silty sand to sandy silt (5)	0.90	1.48	0.79	16.2	60	26.4	1659	2.92E+03	5.68E-04	2.34E-03	3.07E-01	4.83E-02
25	34.12	0.16	126.2	1.74	85.3	1.4	2.096	Sands - clean sand to silty sand (6)	0.90	1.38	0.79	14.6	59	22.7	1686	2.82E+03	5.98E-04	2.61E-03	4.04E-01	7.96E-03
26	34.28	0.16	126.1	1.86	85.0	1.5	2.106	Sand mixtures - silty sand to sandy silt (5)	0.90	1.42	0.79	15.2	59	23.5	1692	2.86E+03	5.92E-04	2.50E-03	3.73E-01	7.34E-03
27	34.45	0.16	130.2	1.84	87.6	1.4	2.116	Sands - clean sand to silty sand (6)	0.89	1.38	0.79	14.5	60	23.6	1698	2.87E+03	5.92E-04	2.49E-03	3.69E-01	7.27E-03
28	34.61	0.16	137.5	2.08	92.3	1.5	2.126	Sand mixtures - silty sand to sandy silt (5)	0.89	1.39	0.79	14.6	62	25.8	1704	2.95E+03	5.77E-04	2.30E-03	3.06E-01	6.02E-03
29	36.91	2.30	166.6	2.26	109.9	1.4	2.203	Sands - clean sand to silty sand (6)	0.88	1.27	0.76	12.2	70	27.7	1748	3.05E+03	5.72E-04	2.17E-03	2.59E-01	7.14E-02
30	37.89	0.98	142.1	2.62	91.6	1.9	2.304	Sand mixtures - silty sand to sandy silt (5)	0.87	1.50	0.77	16.4	62	28.8	1803	3.19E+03	5.66E-04	1.91E-03	2.20E-01	2.60E-02
31	47.57	2.11	180.6	2.27	109.0	1.3	2.636	Sands - clean sand to silty sand (6)	0.82	1.25	0.72	11.7	69	26.7	408	7.11E+02	1.26E-04	4.23E-04	4.76E-02	5.53E-02

Liquefaction Analysis Using CPT Data

0.22
1.59

CPT: CPT3
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.
Removal: 0 ft.

W.O.: 8953
Elev.: 0
Ic: 2.6

C:\JOBS\8953_Everest Terrace\6-29-04\CPT-03 cpd

Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.90
Magnitude Scaling Factor: 1.24

Layer	Layer Bott. (ft)	Layer Thick. (ft)	Avg. Tip Resist qc (tsf)	Avg. Side Fric. fs	Avg. Tip Resist qc1N (tsf)	Norm. Frc. Rt. (%)	Eff. O.B. (tsf)	Soil Behavior Type	Avg. rd	Kc	Kσ	Avg. Fines Content (%)	Avg. Dr (%)	Avg. SPT N1(60)liq (bpf)	Min. CRR (M=7.5)	Avg. CSR	Min. Liq. FS	Avg. Strain (%)	Liq Settle. (in)
31	47.57	7.57	180.6	2.27	109.0	1.3	2.636	Sands - clean sand to silty sand (6)	0.82	1.25	0.72	11.7	69	32.0	0.17	0.309	0.56	0.6	0.22
32	47.74	0.16	191.4	3.30	109.2	1.8	2.944	Sand mixtures - silty sand to sandy silt (5)	0.78	1.37	0.7	14.3	69	28.5	0.27	0.306	0.89	0.0	0
33	49.70	1.97	236.8	3.25	133.5	1.4	3.010	Sands - clean sand to silty sand (6)	0.76	1.21	0.69	10.8	77	29.7	0.25	0.305	0.83	0.0	0

Evaluation of Liquefac . Resistance of Soils
Using CPT Data

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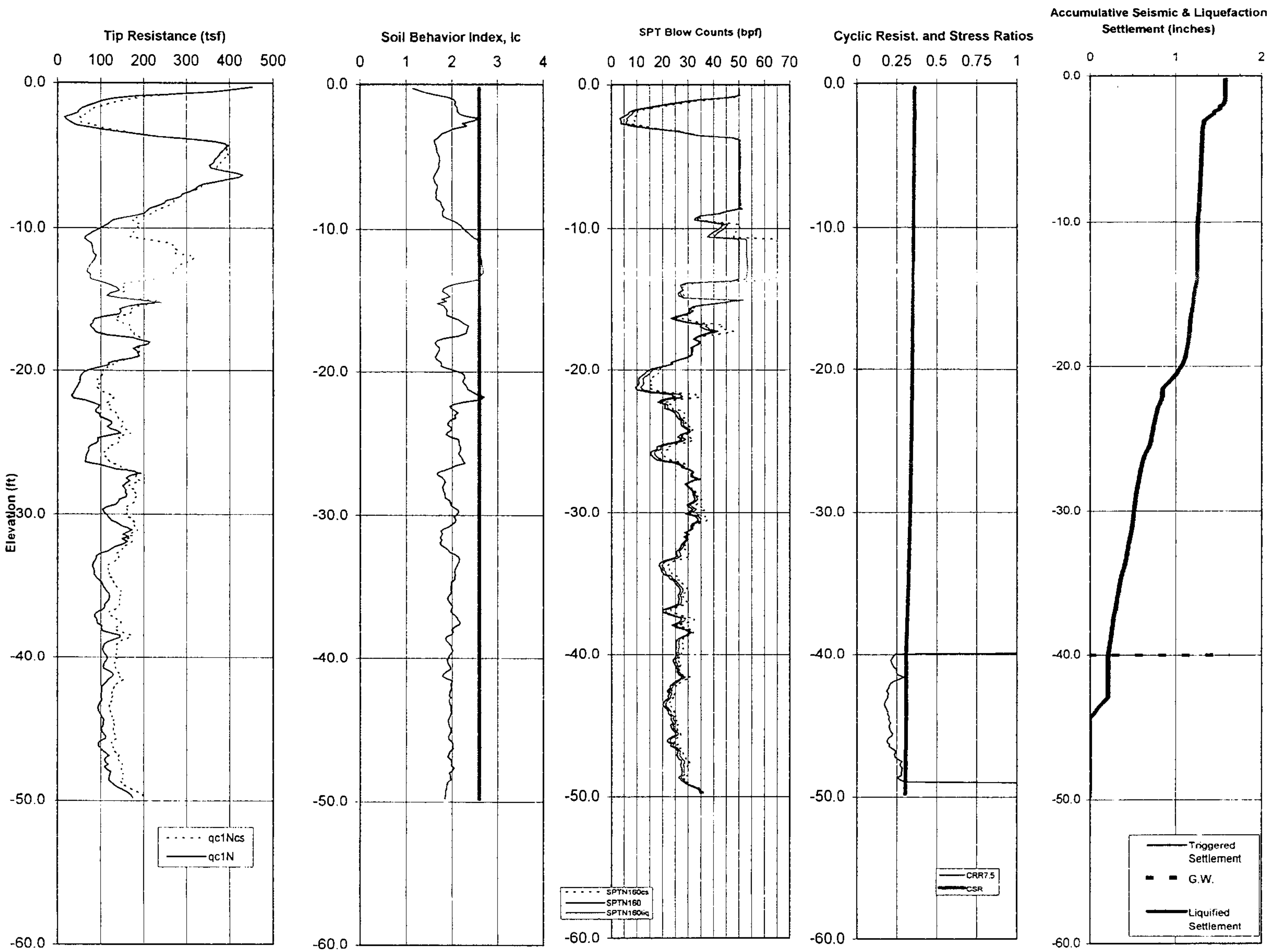
CPT: CPT3
G.W. Depth: 60 ft.
Design G.W. Depth: 40 ft.

Elev.: 0

Ic: 2.6 (Standard Value)

Fill Height: 0.0 ft. 125 pcf
Max horizontal acc. @ surface: 0.69 g
Design earthquake magnitude: 6.9
Removal: 0 ft

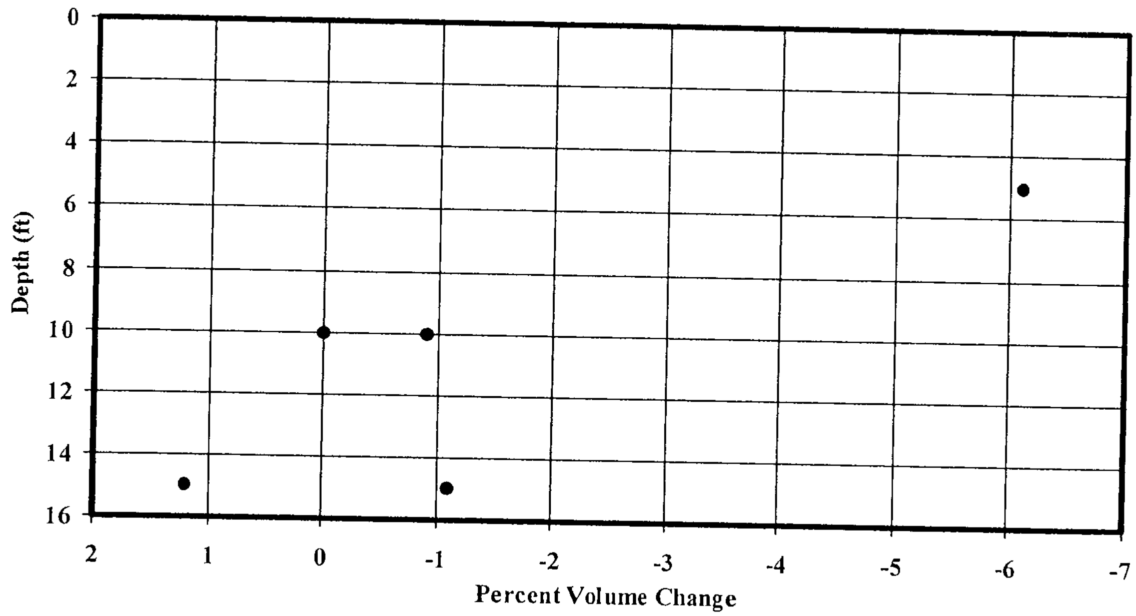


GEOLABS-WESTLAKE VILLAGE

*Exclude Settlement from layers thinner than 6 inches

HYDROCONSOLIDATION/EXPANSION VS. DEPTH

Alluvium

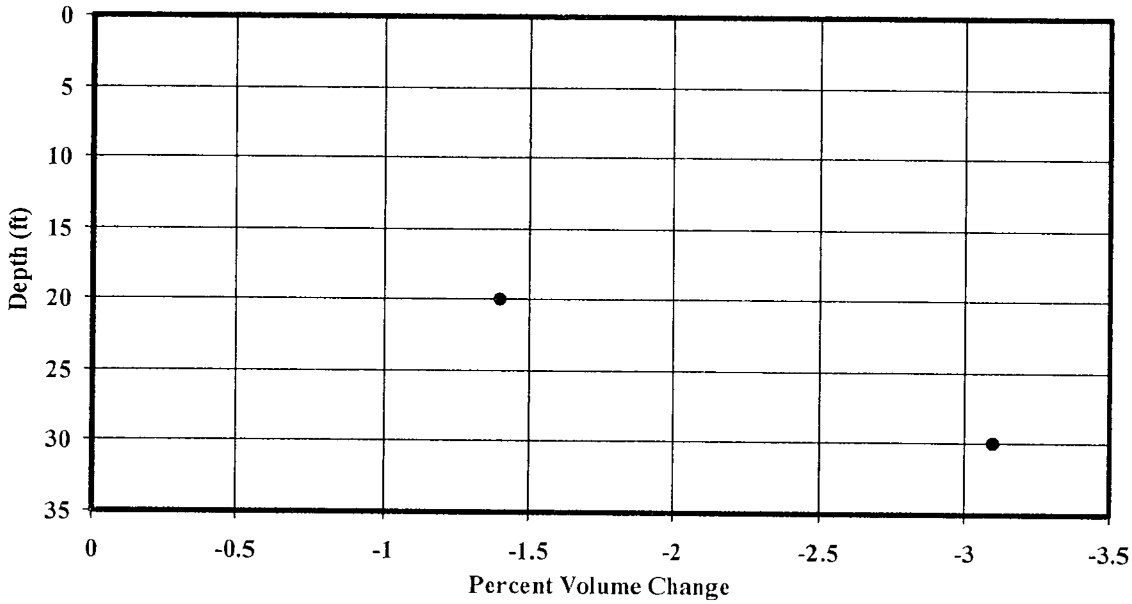


Note: Expansion (+), Collapse (-)

Excavation	Depth (ft)	Field DD (pcf)	M (%)	e	S (%)	Volume Change (%)	Alluvium Material
B3	5	103.7	3.3	0.63	14	-6.1	Silty Sand
B1	10	113.1	8.2	0.48	46.1	0.0	Silty Sand
B4	10	122.5	6.0	0.37	44.1	-0.9	Silty Sand
B1	15	108.5	17.8	0.54	88.1	1.2	Clay
B3	15	107.0	2.7	0.56	13	-1.1	Sand

DD = Field Dry Density, M = Field Moisture, e = initial void ratio, S = initial degree of saturation, Volume Change = percent of hydroconsolidation(-) or expansion (+)

HYDROCONSOLIDATION/EXPANSION VS. DEPTH
Saugus Formation



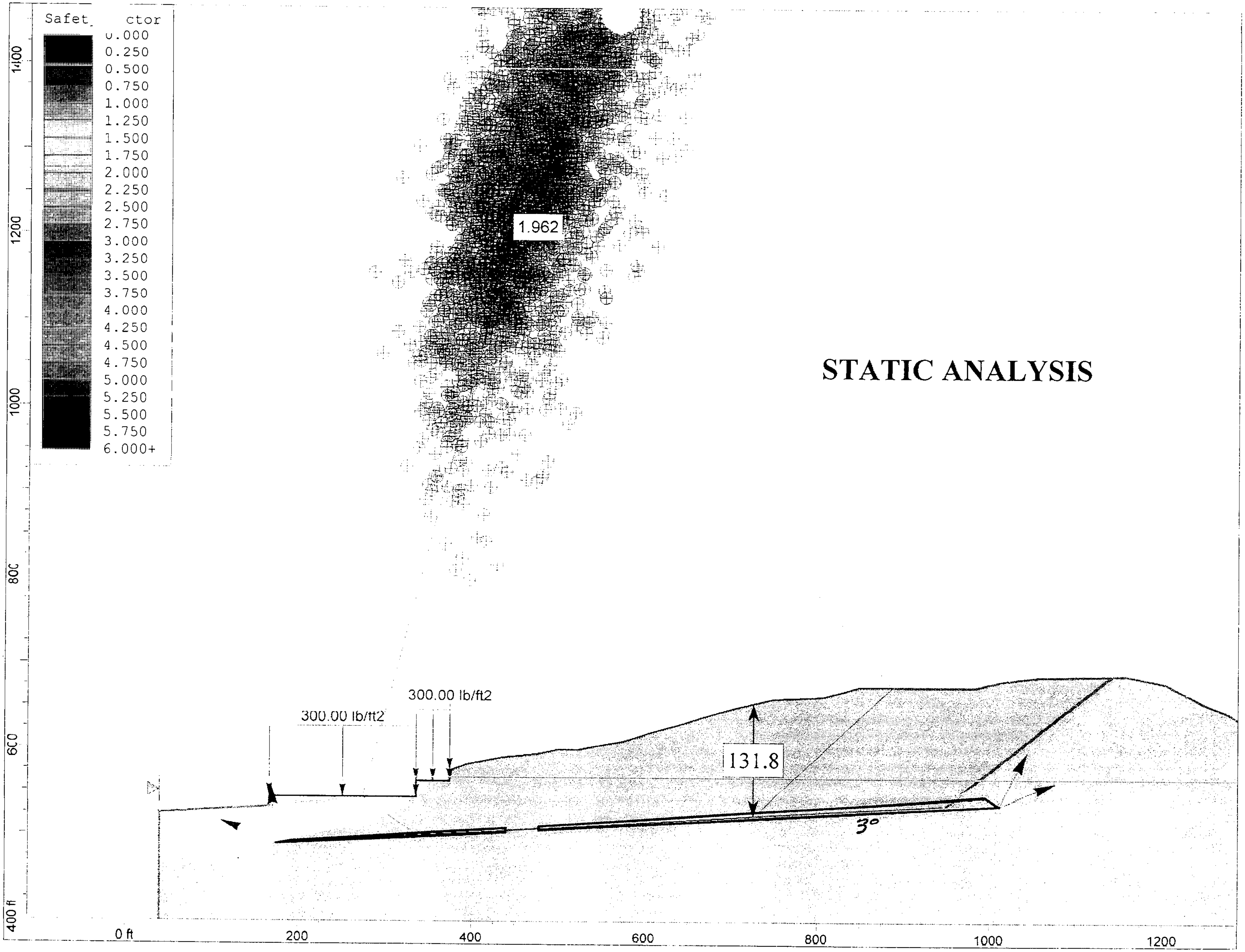
Note: Expansion (+), Collapse (-)

Excavation	Depth (ft)	Field DD (pcf)	M (%)	e	S (%)	Volume Change (%)	Saugus Formation Material
B6	20	114.1	3.7	0.47	21	-1.4	SANDSTONE
B6	30	102.8	6.0	0.62	26	-3.1	SANDSTONE

DD = Field Dry Density, M = Field Moisture, e = initial void ratio, S = initial degree of saturation, Volume Change = percent of hydroconsolidation(-) or expansion (+)

APPENDIX D

SLOPE STABILITY ANALYSIS



Safet_ ctor
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1.500
1.750
2.000
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2.750
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3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.962
STATIC ANALYSIS
300.00 lb/ft2
300.00 lb/ft2
131.8
3°
1400
1200
1000
800
600
400 ft
0 ft
200
400
600
800
1000
1200

Slide Analysis Information

Document Name

File Name: 8953 B-B' 20051130 study.sli

Project Settings

Project Title: Section B-B' Static Analysis
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Maximum
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Non-Circular Block Search
Number of Surfaces: 3000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Enabled
Left Projection Angle (Start Angle): 157
Left Projection Angle (End Angle): 95
Right Projection Angle (Start Angle): 63
Right Projection Angle (End Angle): 22
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

1 Distributed Load present:
Distributed Load Constant Distribution, Orientation: Vertical, Magnitude: 300 lb/ft2

Material Properties

Material: Qal
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 200 psf
Friction Angle: 38 degrees
Water Surface: None

Material: TQs
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs (5' Bed)
Strength Type: Shear Normal function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs above Bed
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: Eng. Fill
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 125 psf
Friction Angle: 32 degrees
Water Surface: None

Global Minimums

Method: spencer
FS: 1.962050
Axis Location: 448.750, 1220.822
Left Slip Surface Endpoint: 274.873, 542.955
Right Slip Surface Endpoint: 886.717, 675.000
Resisting Moment=1.60009e+009 lb-ft
Driving Moment=8.15523e+008 lb-ft
Resisting Horizontal Force=2.0086e+006 lb
Driving Horizontal Force=1.02373e+006 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 1757
Number of Invalid Surfaces: 1243
Error Codes:
Error Code -108 reported for 39 surfaces
Error Code -111 reported for 95 surfaces
Error Code -112 reported for 1109 surfaces

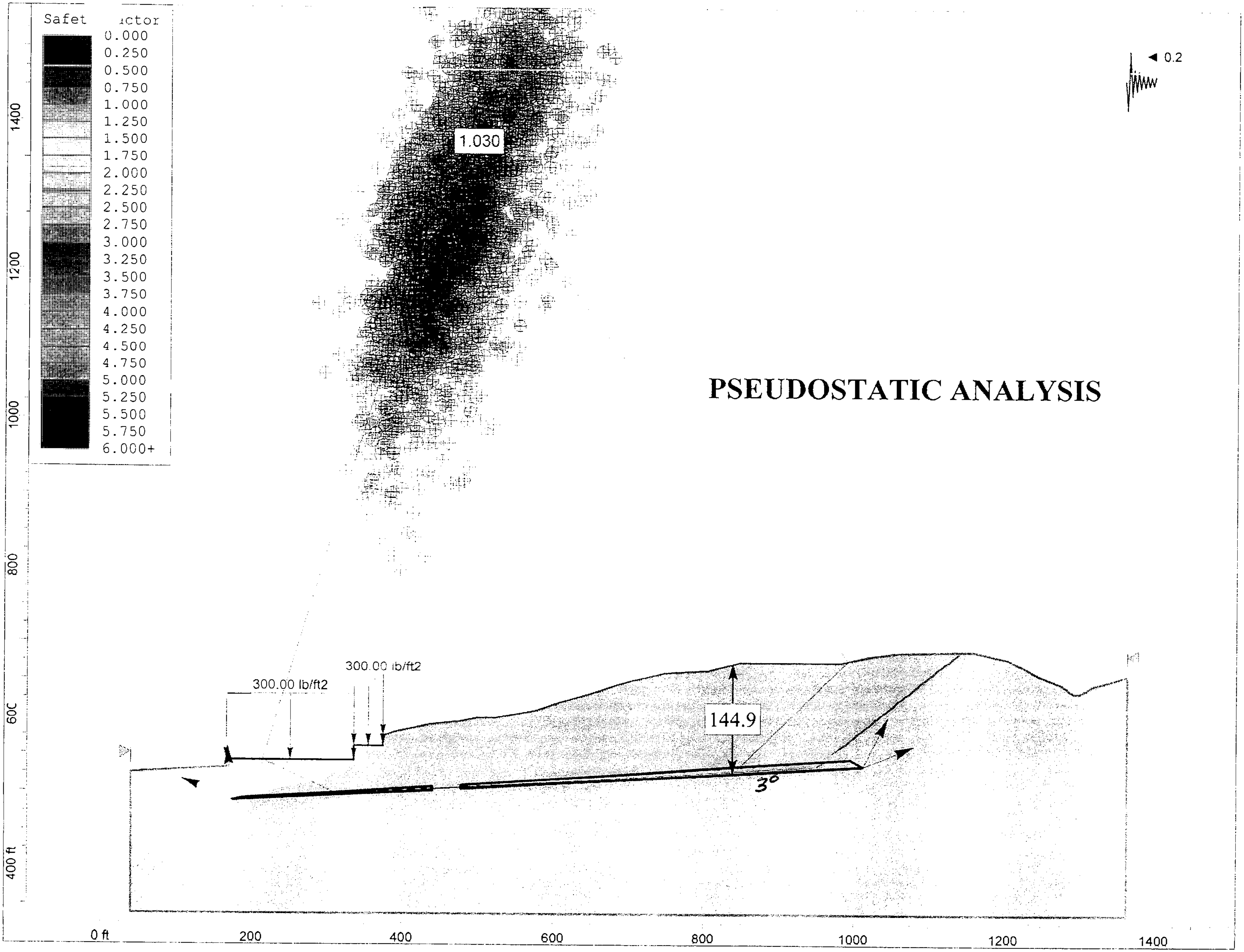
Error Codes

The following errors were encountered during the computation:

-108 = Total driving moment
or total driving force < 0.1. This is to
limit the calculation of extremely high safety
factors if the driving force is very small
(0.1 is an arbitrary number).

-111 = safety factor equation did not converge

-112 = The coefficient M-Alpha =
cos(alpha)(1+tan(alpha)tan(phi)/F)
< 0.2 for the final iteration of the safety factor calculation. This
screens out
some slip surfaces which may not be valid in the context of the
analysis, in
particular, deep seated slip surfaces with many high negative base
angle
slices in the passive zone.



Safet actor
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1.500
1.750
2.000
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3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.030
0.2
PSEUDOSTATIC ANALYSIS
300.00 lb/ft2
300.00 lb/ft2
144.9
3°
1400
1200
1000
800
600
400 ft
0 ft
200
400
600
800
1000
1200
1400

Slide Analysis Information

Document Name

File Name: 8953 B-B' 20051130 study.sli

Project Settings

Project Title: Section B-B' Pseudostatic Analysis
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Maximum
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Non-Circular Block Search
Number of Surfaces: 3000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Enabled
Left Projection Angle (Start Angle): 157
Left Projection Angle (End Angle): 95
Right Projection Angle (Start Angle): 63
Right Projection Angle (End Angle): 22
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.2
1 Distributed Load present:
Distributed Load Constant Distribution, Orientation: Vertical, Magnitude: 300 lb/ft2

Material Properties

Material: Qal
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 200 psf
Friction Angle: 38 degrees
Water Surface: None

Material: TQs
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs (5' Bed)
Strength Type: Shear Normal function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs above Bed
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: Eng. Fill
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3

Cohesion: 125 psf
Friction Angle: 32 degrees
Water Surface: None

Global Minimums

Method: spencer
FS: 1.029770
Axis Location: 468.828, 1386.596
Left Slip Surface Endpoint: 214.712, 542.955
Right Slip Surface Endpoint: 991.271, 677.119
Resisting Moment=2.56358e+009 lb-ft
Driving Moment=2.48946e+009 lb-ft
Resisting Horizontal Force=2.68726e+006 lb
Driving Horizontal Force=2.60957e+006 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 1094
Number of Invalid Surfaces: 1906
Error Codes:
Error Code -108 reported for 7 surfaces
Error Code -111 reported for 107 surfaces
Error Code -112 reported for 1792 surfaces

Error Codes

The following errors were encountered during the computation:

-108 = Total driving moment
or total driving force < 0.1. This is to
limit the calculation of extremely high safety
factors if the driving force is very small
(0.1 is an arbitrary number).

-111 = safety factor equation did not converge

-112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F)
< 0.2 for the final iteration of the safety factor calculation. This screens
out
some slip surfaces which may not be valid in the context of the analysis,
in
particular, deep seated slip surfaces with many high negative base
angle
slices in the passive zone.

Simplified Bray Procedure for Evaluating Seismic Slope Stability

Everett Terrace, Moorpark — W.O. 8953

INPUT

Soil Type for Mean Period:	Rock		Failure yield acceleration, Ky:	0.213	g
Shear wave velocity, Vs:	360	m/s	Depth of Failure Surface, h:	132.0	ft
Maximum Horizontal Accel, MHA:	0.70	g	Modal (Deaggregated) distance to source, r:	2.0	km
Attenuation Method:	From OFR 2000-007		Modal (Deaggregated) Earthquake Magnitude, M:	6.90	

Note: Modal M and r are determined as the greatest contributors to the 475 -year hazard level for the MHA

ANALYSES

Estimation of Strong Motion Duration (central 90% of Arias Intensity):

For r>10km

$$D=2.33\cdot\left[\left[\frac{\left[\frac{e^{(5.204+0.851\cdot(M-6))}}{10^{(1.5\cdot M+16.05)}}\right]^{-\frac{1}{3}}}{15.7\cdot10^{6}}\right]+0.805\cdot S+0.063\cdot(r-10)\right]$$

For r < 10km

$$D=2.33\cdot\left[\frac{\left[\frac{e^{(5.204+0.851\cdot(M-6))}}{10^{(1.5\cdot M+16.05)}}\right]^{-\frac{1}{3}}}{15.7\cdot10^{6}}+0.805\cdot S\right]$$

Set S= 0 for Rock Material S= 0

For r= 2 and S= 0, Duration D(5-95)= 12.8 sec. Standard Deviation: 0.565 sec.

Estimation of mean-square Period, Tm of input rock motion:

if M<=7.25 then ln(Tm)=ln(C1+C2*(M-6)+C3*r)+Sdev
if 7.25<M<8 then ln(Tm)=ln(C1+1.25*C2+C3*r)+Sdev

For soil Type of Rock the Rathje et.al. (1998) coefficients are:

C1:	0.411
C2:	0.0837
C3:	0.00208
Stand. Dev:	0.437

The mean-square Period, Tm is 0.49 sec. (mean)

Estimation of fundamental period of equivalent 1-D slide mass at small strains, Ts:

$$T_s=4\cdot\frac{H}{V_s}$$

For H= 132 ft and Vs= 360 m/s; Ts= 0.45 sec.

Ratio Ts/Tm is*: 0.91

Non-Linear Response Factor, NRF is 0.82

Maximum Horizontal Equivalent Acceleration over the duration of earthquake shaking, MHEA:

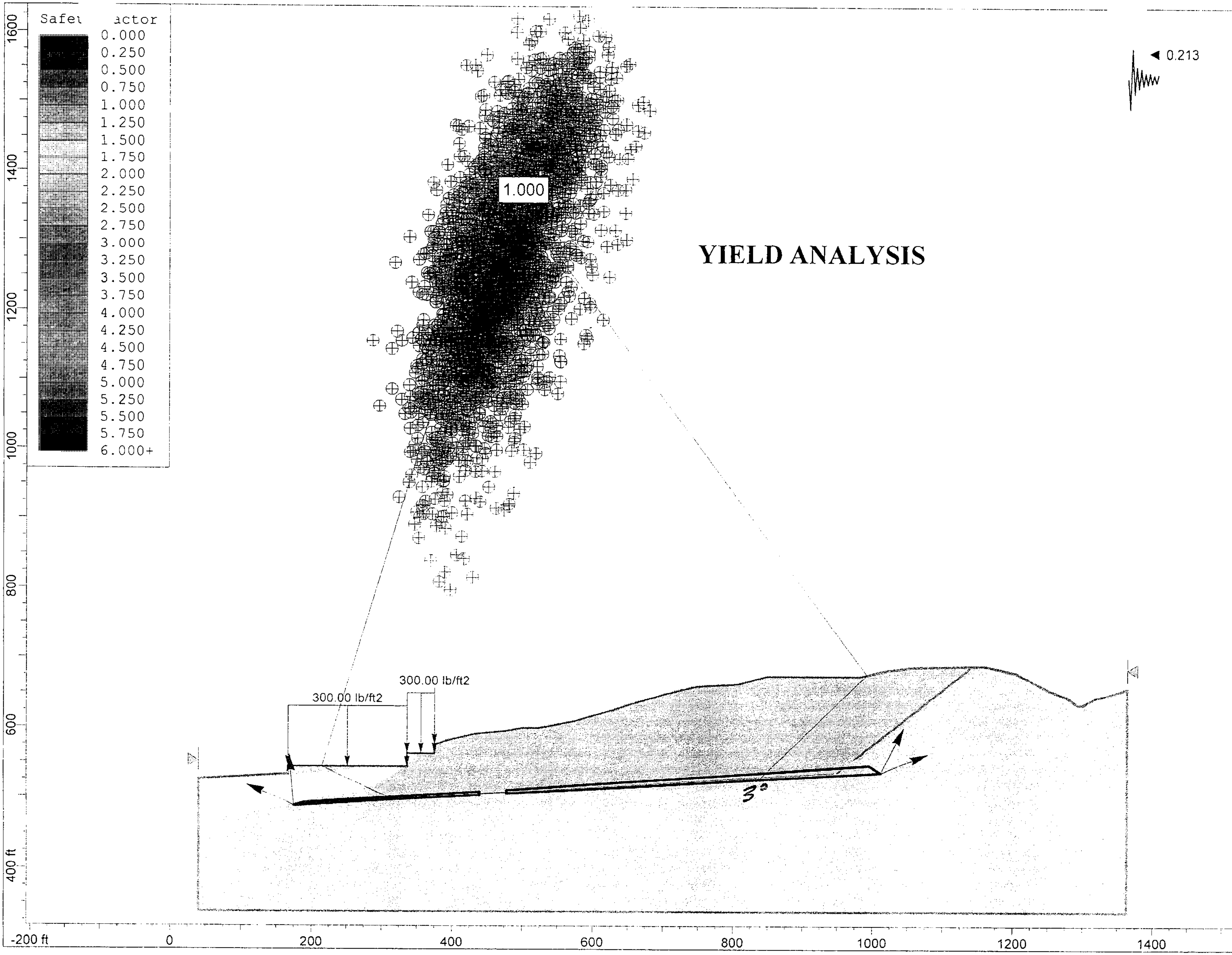
MHEA for the subject slope is 0.33 g

kmax is set to MHEA, therefore, kmax is 0.33 g and ky/kmax is 0.65

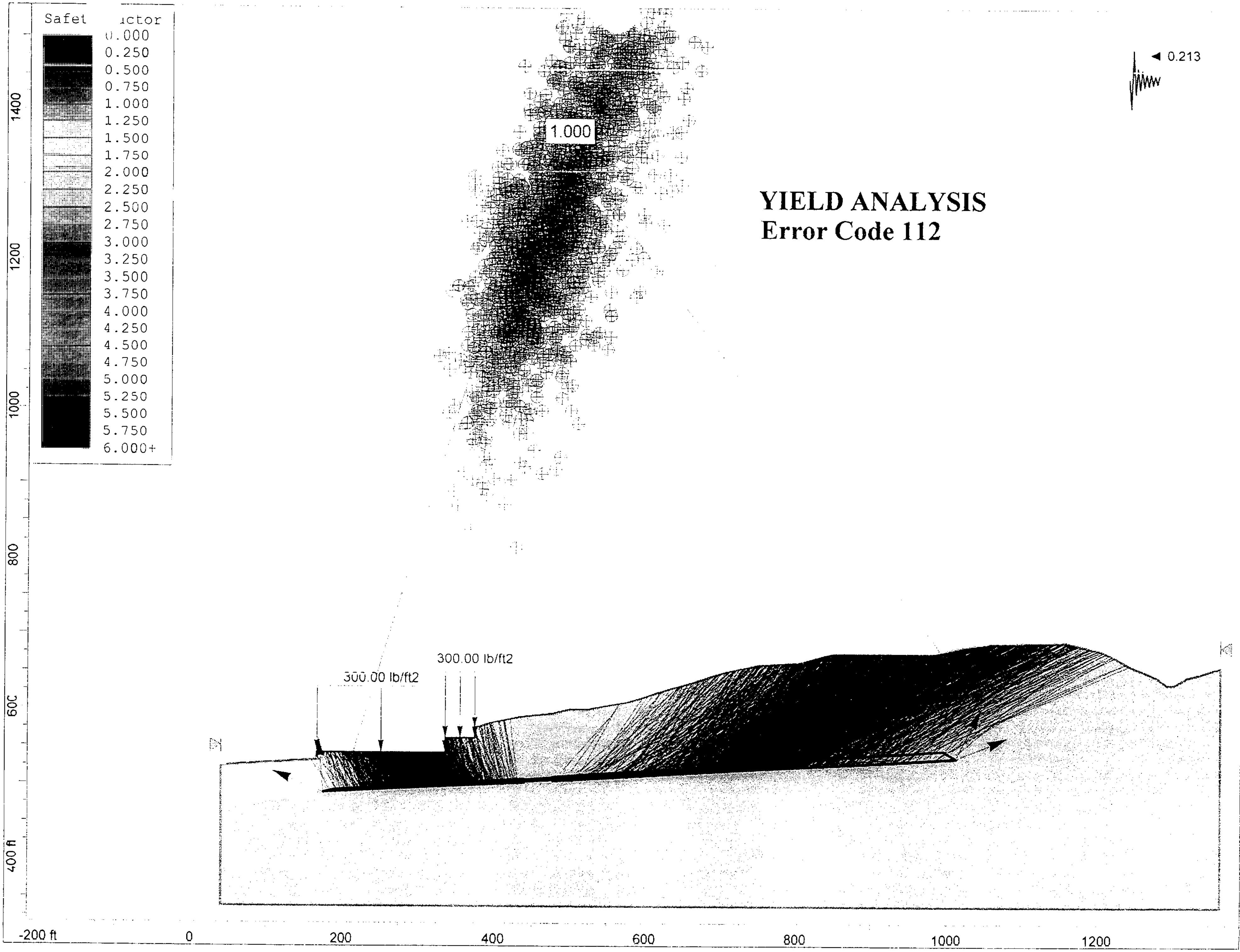
Estimated Displacement, μ:

μ (mean):	1.7 cm (1 in)
μ (M+sig):	4 cm (2 in)
μ (M-sig):	1 cm (0 in)

The results of the analyses indicates the estimated mean displacement is about 2 cm of displacement. The estimated mean plus one standard deviation displacement is about 4 cm.



Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.000
0.213
YIELD ANALYSIS
300.00 lb/ft2
300.00 lb/ft2
3°
1600
1400
1200
1000
800
600
400 ft
-200 ft
0
200
400
600
800
1000
1200
1400



Safet uctor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.000
0.213
YIELD ANALYSIS
Error Code 112
300.00 lb/ft2
300.00 lb/ft2
1400
1200
1000
800
600
400 ft
-200 ft
0
200
400
600
800
1000
1200

Slide Analysis Information

Document Name

File Name: 8953 B-B' 20051130 yield study.sli

Project Settings

Project Title: Section B-B' Pseudostatic Analysis
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Maximum
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Non-Circular Block Search
Number of Surfaces: 3000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Enabled
Left Projection Angle (Start Angle): 157
Left Projection Angle (End Angle): 95
Right Projection Angle (Start Angle): 63
Right Projection Angle (End Angle): 22
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.213
1 Distributed Load present:
Distributed Load Constant Distribution, Orientation: Vertical, Magnitude: 300 lb/ft2

Material Properties

Material: Qal
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 200 psf
Friction Angle: 38 degrees
Water Surface: None

Material: TQs
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs (5' Bed)
Strength Type: Shear Normal function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs above Bed
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: Eng. Fill
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 125 psf

Friction Angle: 32 degrees
Water Surface: None

Global Minimums

Method: spencer
FS: 1.000420
Axis Location: 468.828, 1386.596
Left Slip Surface Endpoint: 214.712, 542.955
Right Slip Surface Endpoint: 991.271, 677.119
Resisting Moment=2.5741e+009 lb-ft
Driving Moment=2.57301e+009 lb-ft
Resisting Horizontal Force=2.70123e+006 lb
Driving Horizontal Force=2.70008e+006 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 1056
Number of Invalid Surfaces: 1944
Error Codes:
Error Code -108 reported for 6 surfaces
Error Code -111 reported for 115 surfaces
Error Code -112 reported for 1823 surfaces

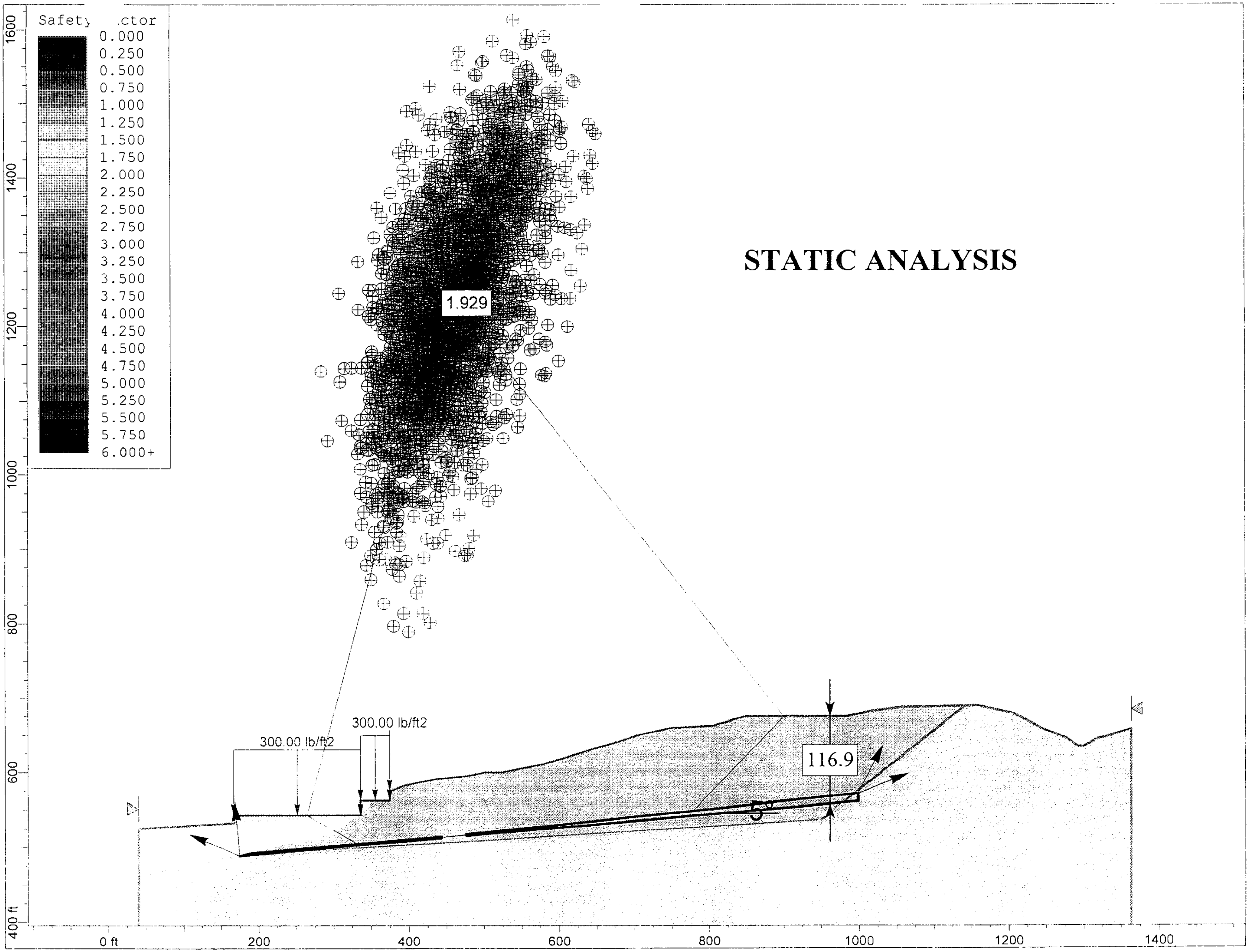
Error Codes

The following errors were encountered during the computation:

-108 = Total driving moment
or total driving force < 0.1. This is to
limit the calculation of extremely high safety
factors if the driving force is very small
(0.1 is an arbitrary number).

-111 = safety factor equation did not converge

-112 = The coefficient M-Alpha =
cos(alpha)(1+tan(alpha)tan(phi)/F)
< 0.2 for the final iteration of the safety factor calculation. This
screens out
some slip surfaces which may not be valid in the context of the
analysis, in
particular, deep seated slip surfaces with many high negative base
angle
slices in the passive zone.



Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
STATIC ANALYSIS
1.929
300.00 lb/ft2
300.00 lb/ft2
116.9
5°
1600
1400
1200
1000
800
600
400 ft
0 ft
200
400
600
800
1000
1200
1400

Slide Analysis Information

Document Name

File Name: 8953 B-B' 20051130 5deg study.sli

Project Settings

Project Title: Section B-B' Static Analysis
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Maximum
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Non-Circular Block Search
Number of Surfaces: 3000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Enabled
Left Projection Angle (Start Angle): 157
Left Projection Angle (End Angle): 95
Right Projection Angle (Start Angle): 63
Right Projection Angle (End Angle): 22
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

1 Distributed Load present:
Distributed Load Constant Distribution, Orientation: Vertical, Magnitude: 300 lb/ft2

Material Properties

Material: Qal
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 200 psf
Friction Angle: 38 degrees
Water Surface: None

Material: TQs
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs (5' Bed)
Strength Type: Shear Normal function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs above Bed
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: Eng. Fill
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 125 psf
Friction Angle: 32 degrees
Water Surface: None

Global Minimums

Method: spencer
FS: 1.929270
Axis Location: 450.233, 1244.504
Left Slip Surface Endpoint: 264.514, 542.955
Right Slip Surface Endpoint: 900.040, 675.000
Resisting Moment=1.55239e+009 lb-ft
Driving Moment=8.0465e+008 lb-ft
Resisting Horizontal Force=1.925e+006 lb
Driving Horizontal Force=997785 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 1851
Number of Invalid Surfaces: 1149
Error Codes:
Error Code -108 reported for 57 surfaces
Error Code -111 reported for 39 surfaces
Error Code -112 reported for 1053 surfaces

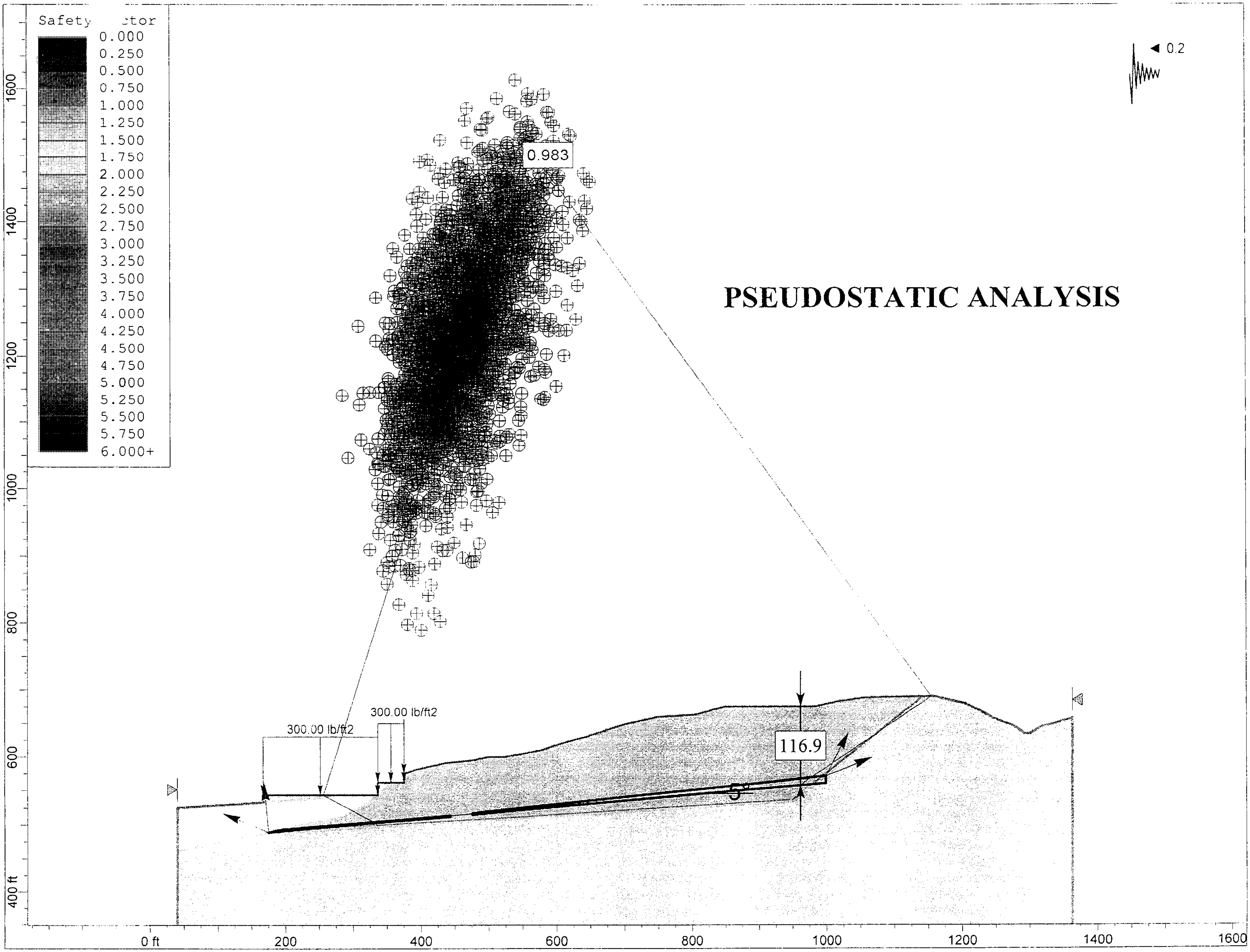
Error Codes

The following errors were encountered during the computation:

-108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).

-111 = safety factor equation did not converge

-112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.



Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.2
0.983
PSEUDOSTATIC ANALYSIS
300.00 lb/ft2
300.00 lb/ft2
116.9
1600
1400
1200
1000
800
600
400 ft
0 ft
200
400
600
800
1000
1200
1400
1600

Slide Analysis Information

Document Name

File Name: 8953 B-B' 20051130 5deg Pseudostaticstudy.sli

Project Settings

Project Title: Section B-B' Pseudostatic Analysis
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Maximum
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Non-Circular Block Search
Number of Surfaces: 3000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Enabled
Left Projection Angle (Start Angle): 157
Left Projection Angle (End Angle): 95
Right Projection Angle (Start Angle): 63
Right Projection Angle (End Angle): 22
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.2
1 Distributed Load present:
Distributed Load Constant Distribution, Orientation: Vertical, Magnitude: 300 lb/ft2

Material Properties

Material: Qal
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 200 psf
Friction Angle: 38 degrees
Water Surface: None

Material: TQs
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs (5' Bed)
Strength Type: Shear Normal function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: TQs above Bed
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

Material: Eng. Fill
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 125 psf
Friction Angle: 32 degrees
Water Surface: None

Global Minimums

Method: spencer
FS: 0.983370
Axis Location: 557.957, 1514.225
Left Slip Surface Endpoint: 256.025, 542.955
Right Slip Surface Endpoint: 1153.814, 689.917
Resisting Moment=3.08194e+009 lb-ft
Driving Moment=3.13406e+009 lb-ft
Resisting Horizontal Force=2.9217e+006 lb
Driving Horizontal Force=2.97111e+006 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 1265
Number of Invalid Surfaces: 1735
Error Codes:
Error Code -108 reported for 17 surfaces
Error Code -111 reported for 47 surfaces
Error Code -112 reported for 1671 surfaces

Error Codes

The following errors were encountered during the computation:

-108 = Total driving moment
or total driving force < 0.1. This is to
limit the calculation of extremely high safety
factors if the driving force is very small
(0.1 is an arbitrary number).

-111 = safety factor equation did not converge

-112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F)
< 0.2 for the final iteration of the safety factor calculation. This screens out
some slip surfaces which may not be valid in the context of the analysis, in
particular, deep seated slip surfaces with many high negative base angle
slices in the passive zone.

Simplified Bray Procedure for Evaluating Seismic Slope Stability

Tract 5405, Section I-I'	W.O. 8623-WL

INPUT

Soil Type for Mean Period:	Rock		Failure yield acceleration, Ky:	0.190	g
Shear wave velocity, Vs:	360	m/s	Depth of Failure Surface, h:	117.0	ft
Maximum Horizontal Accel, MHA:	0.70	g	Modal (Deaggregated) distance to source, r:	2.0	km
Attenuation Method:	From OFR 2000-007		Modal (Deaggregated) Earthquake Magnitude, M:	6.90	

Note: Modal M and r are determined as the greatest contributors to the 475 -year hazard level for the MHA.

ANALYSES

Estimation of Strong Motion Duration (central 90% of Arias Intensity):

For r>10km

$$D=2.33\cdot\left[\left[\frac{\left[\frac{e^{(5.204+0.851\cdot(M-6))}}{10^{(1.5\cdot M+16.05)}}\right]^{\frac{-1}{3}}}{15.7\cdot10^{6}}\right]+0.805\cdot S+0.063\cdot(r-10)\right]$$

For r < 10km

$$D=2.33\cdot\left[\frac{\left[\frac{e^{(5.204+0.851\cdot(M-6))}}{10^{(1.5\cdot M+16.05)}}\right]^{\frac{-1}{3}}}{15.7\cdot10^{6}}+0.805\cdot S\right]$$

Set S= 0 for Rock Material S= 0

For r= 2 and S= 0, Duration D(5-95)= 12.8 sec. Standard Deviation: 0.565 sec.

Estimation of mean-square Period, Tm of input rock motion:

if M<=7.25 then ln(Tm)=ln(C1+C2*(M-6)+C3*r)+Sdev
if 7.25<M<8 then ln(Tm)=ln(C1+1.25*C2+C3*r)+Sdev

For soil Type of Rock the Rathje et.al. (1998) coefficients are:

C1:	0.411
C2:	0.0837
C3:	0.00208
Stand. Dev:	0.437

The mean-square Period, Tm is 0.49 sec. (mean)

Estimation of fundamental period of equivalent 1-D slide mass at small strains, Ts:

$$T_s=4\cdot\frac{H}{V_s}$$

For H= 117 ft and Vs= 360 m/s; Ts= 0.4 sec.

Ratio Ts/Tm is*: 0.81

Non-Linear Response Factor, NRF is 0.82

Maximum Horizontal Equivalent Acceleration over the duration of earthquake shaking, MHEA:

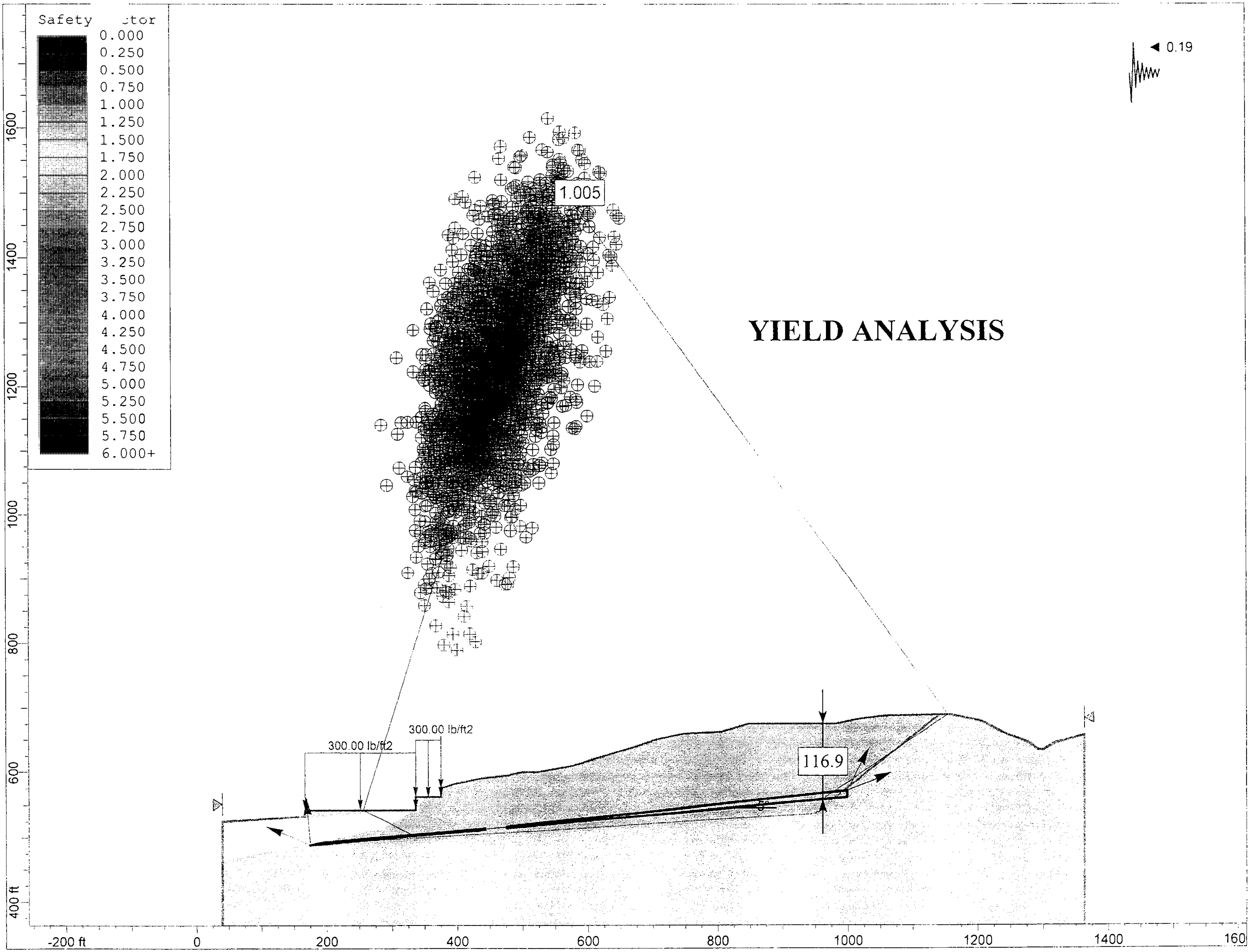
MHEA for the subject slope is 0.36 g

kmax is set to MHEA, therefore, kmax is 0.36 g and ky/kmax is 0.53

Estimated Displacement, μ:

μ (mean):	**5.0 cm (2 in)**
μ (M+sig):	11 cm (4 in)
μ (M-sig):	2 cm (1 in)

The results of the analyses indicates the estimated mean displacement is about 5 cm of displacement. The estimated mean plus one standard deviation displacement is about 11 cm.



Safety ctor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.19
1.005
YIELD ANALYSIS
300.00 lb/ft2
300.00 lb/ft2
116.9
1600
1400
1200
1000
800
600
400 ft
-200 ft
0
200
400
600
800
1000
1200
1400
160

Slide Analysis Information

Document Name

File Name: 8953 B-B' 20051130 5deg Yield study.sli

Project Settings

Project Title: Section B-B' Pseudostatic Yield Analysis
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Maximum
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Non-Circular Block Search
Number of Surfaces: 3000
Pseudo-Random Surfaces: Enabled
Convex Surfaces Only: Enabled
Left Projection Angle (Start Angle): 157
Left Projection Angle (End Angle): 95
Right Projection Angle (Start Angle): 63
Right Projection Angle (End Angle): 22
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.19
1 Distributed Load present:
Distributed Load Constant Distribution, Orientation: Vertical, Magnitude: 300 lb/ft2

Material Properties

<u>Material: Qal</u>
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 200 psf
Friction Angle: 38 degrees
Water Surface: None

<u>Material: TQs</u>
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

<u>Material: TQs (5' Bed)</u>
Strength Type: Shear Normal function
Unit Weight: 130 lb/ft3
Water Surface: None

<u>Material: TQs above Bed</u>
Strength Type: Anisotropic function
Unit Weight: 130 lb/ft3
Water Surface: None

<u>Material: Eng. Fill</u>
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 125 psf
Friction Angle: 32 degrees
Water Surface: None

Global Minimums

Method: spencer
FS: 1.004880
Axis Location: 557.957, 1514.225
Left Slip Surface Endpoint: 256.025, 542.955
Right Slip Surface Endpoint: 1153.814, 689.917
Resisting Moment=3.06634e+009 lb-ft
Driving Moment=3.05145e+009 lb-ft
Resisting Horizontal Force=2.90743e+006 lb
Driving Horizontal Force=2.89331e+006 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 1286
Number of Invalid Surfaces: 1714
Error Codes:
Error Code -108 reported for 19 surfaces
Error Code -111 reported for 45 surfaces
Error Code -112 reported for 1650 surfaces

Error Codes

The following errors were encountered during the computation:

-108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).

-111 = safety factor equation did not converge

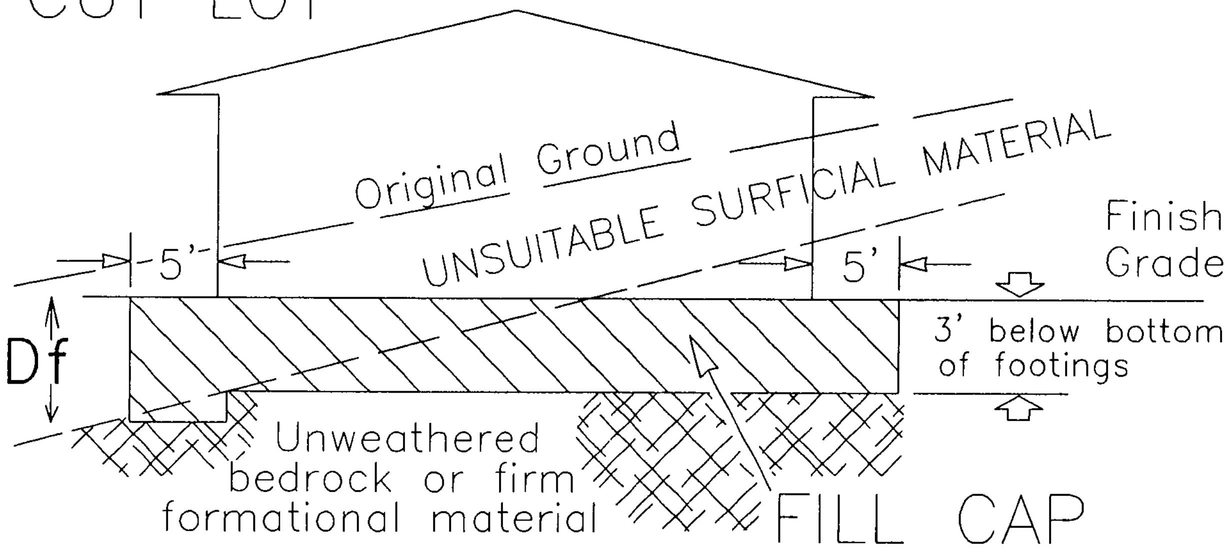
-112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.

APPENDIX E

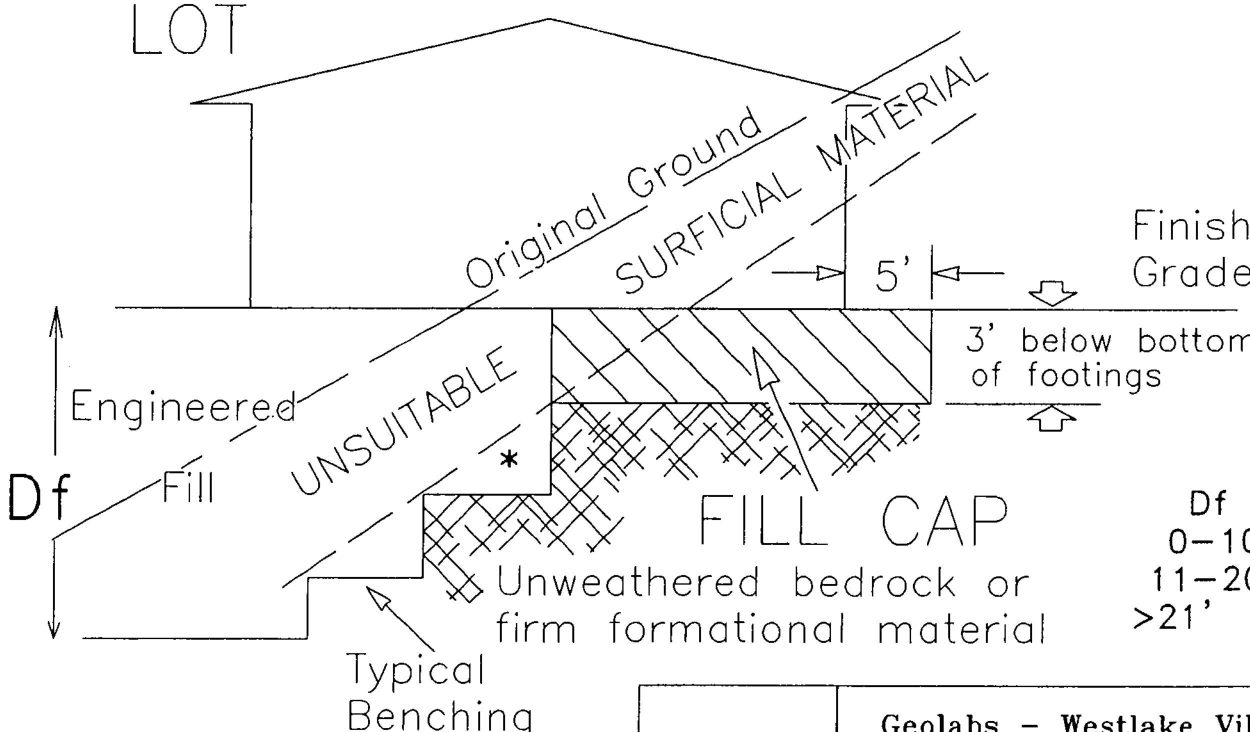
TYPICAL DETAILS

FILL CAP DETAIL

CUT LOT



TRANSITION LOT



Df	Dc
0–10'	5'
11–20'	7'
>21'	10'

* For backslopes steeper than 3:1 the uppermost bench is to be 15' wide.

Geolabs – Westlake Village
GEOLOGY AND SOIL ENGINEERING

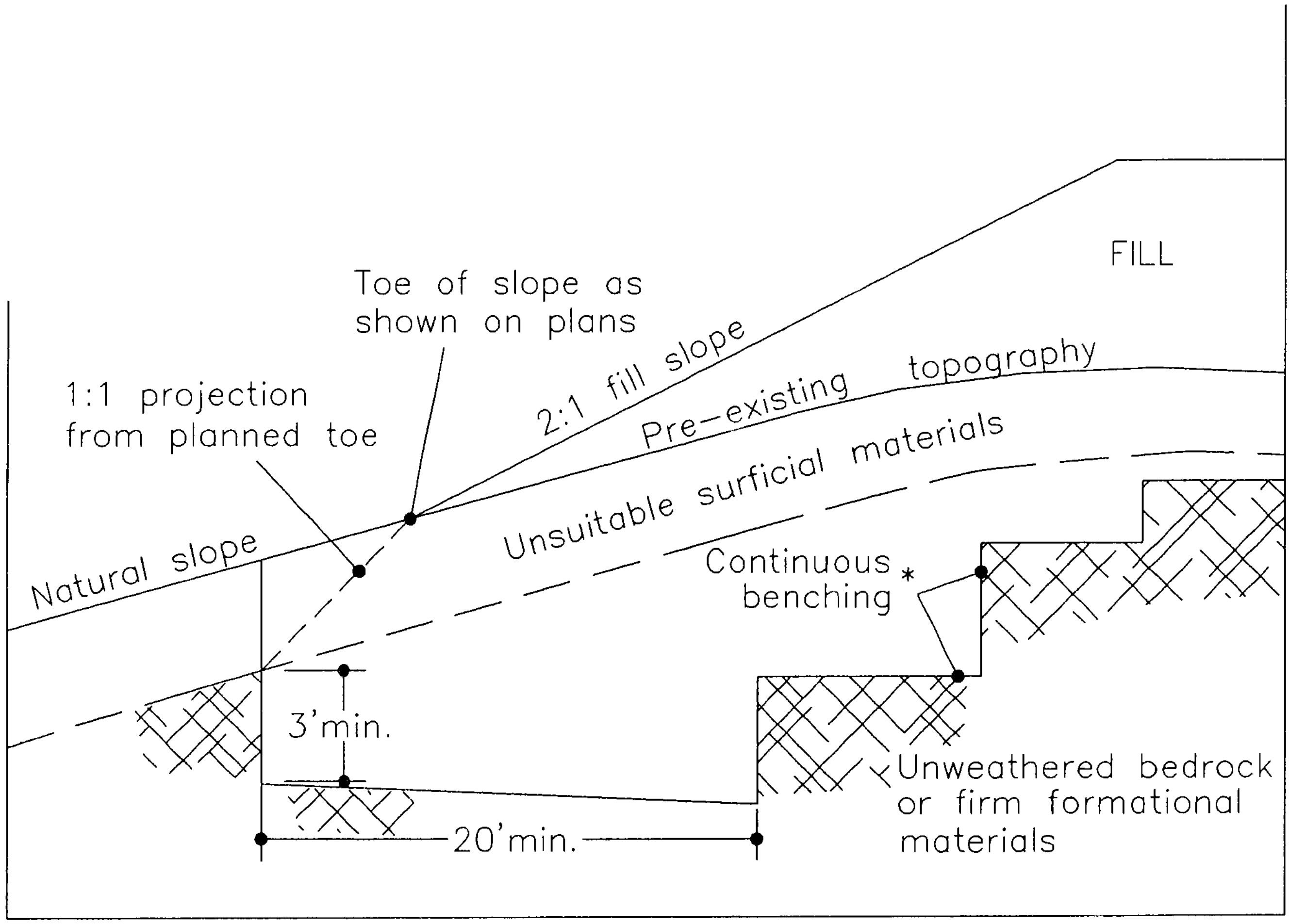
DATE 12.2.05 BY SBS
SCALE NTS W.O. 8953

TRANSLOT

PLATE

E-1

TYPICAL FILL OVER NATURAL SLOPE



* Where natural slope gradient is 5:1 or less, benching is not necessary except as required to remove unsuitable or compressible surficial material.

FILLNAT

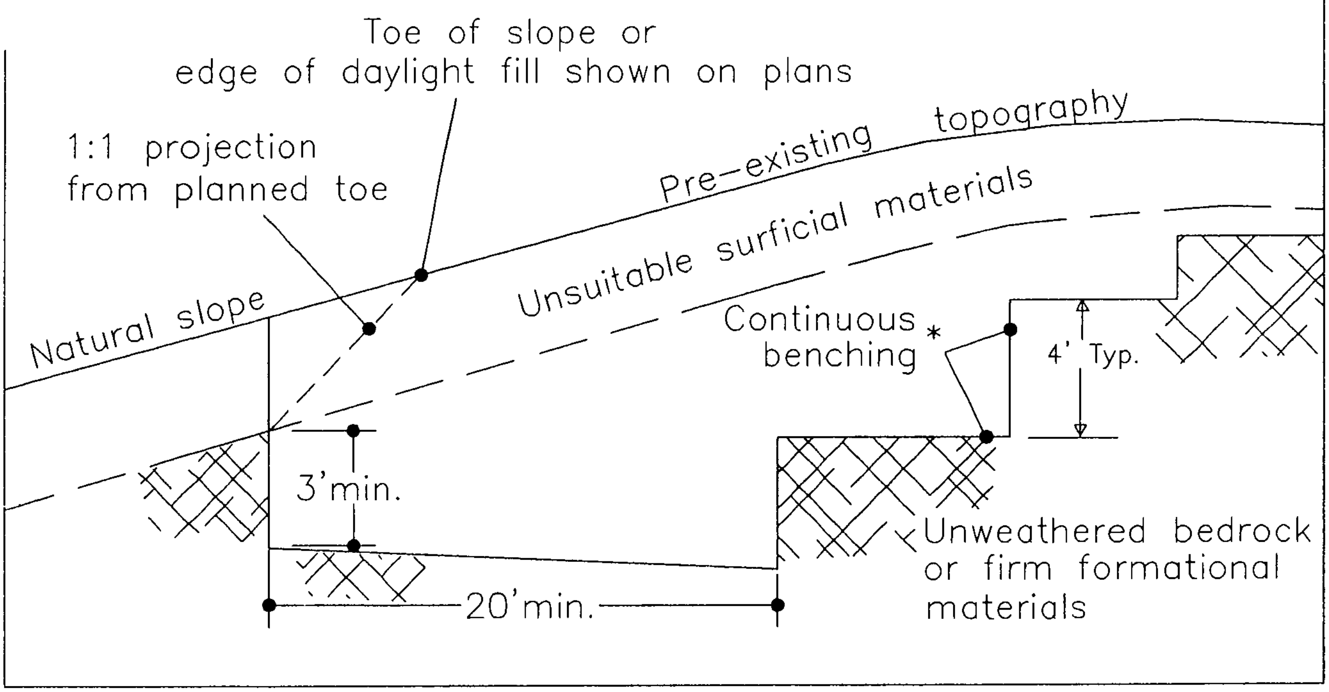
Geolabs - Westlake Village
GEOLOGY AND SOIL ENGINEERING

DATE 12.2.05 BY SBS
SCALE NTS W.O. 8953

PLATE

E-2

TYPICAL KEYING AND BENCHING



* Where natural slope gradient is 5:1 or less, benching is not necessary except as required to remove unsuitable or compressible surficial material.

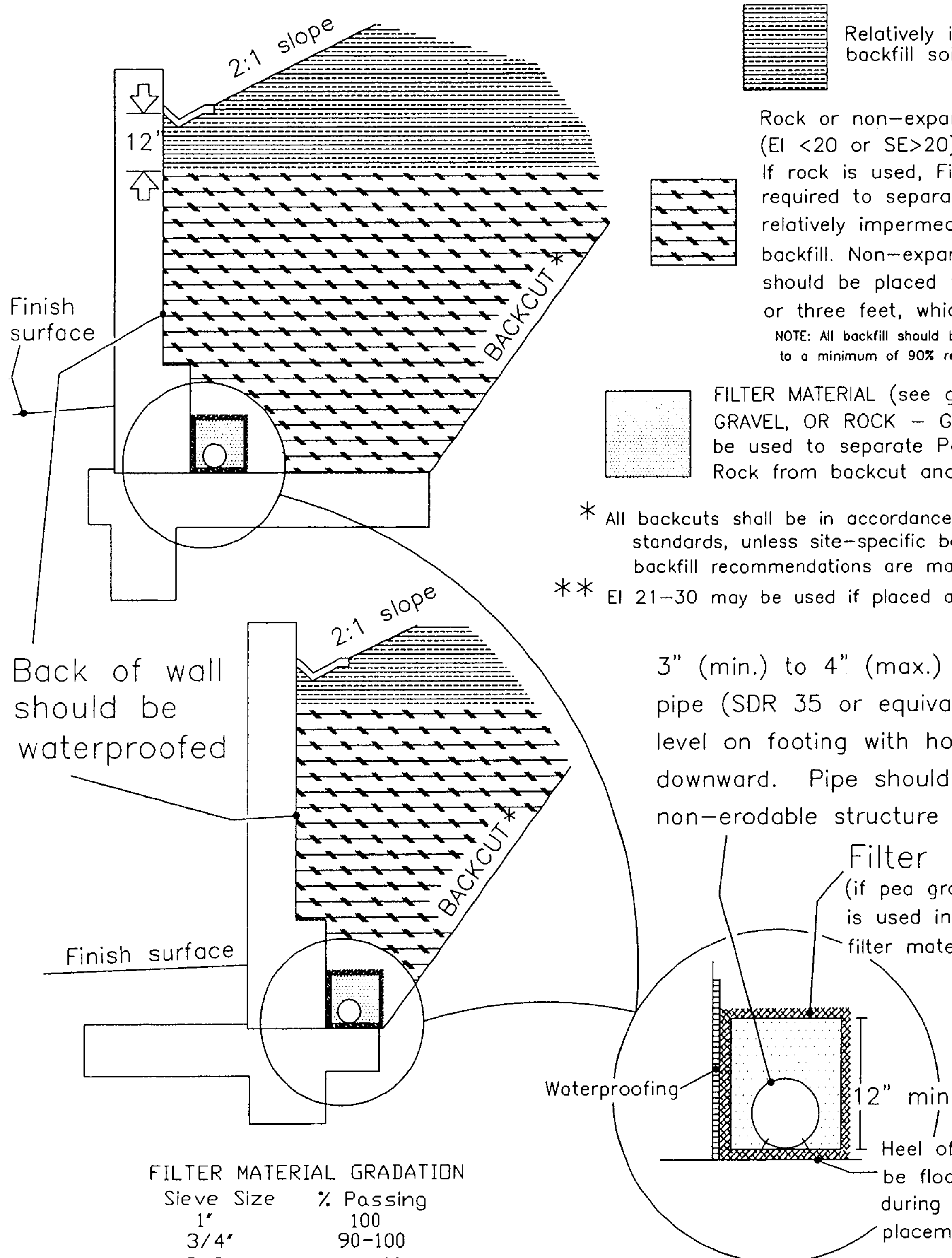
	Geolabs – Westlake Village GEOLOGY AND SOIL ENGINEERING
	DATE 12.2.05 BY SBS SCALE NTS W.O. 8953

FILLNAT

PLATE

E-3

TYPICAL RETAINING WALL



Relatively impermeable backfill soil

Rock or non–expansive soil backfill (EI <20 or SE>20).** If rock is used, Filter Cloth is required to separate rock from relatively impermeable soil and backfill. Non–expansive backfill should be placed to the backcut or three feet, whichever is less.

NOTE: All backfill should be compacted to a minimum of 90% relative density.

FILTER MATERIAL (see gradation), PEA GRAVEL, OR ROCK – Geotextile should be used to separate Pea Gravel or Rock from backcut and backfill.

* All backcuts shall be in accordance with OSHA standards, unless site–specific backcut and/or backfill recommendations are made by this office.

** EI 21–30 may be used if placed at 2% over optimum

3" (min.) to 4" (max.) perforated pipe (SDR 35 or equivalent) laid level on footing with holes set facing downward. Pipe should outlet to a non–erodable structure or device.

FILTER MATERIAL GRADATION

Sieve Size	% Passing
1"	100
3/4"	90–100
3/8"	40–100
#4	24–50
#8	15–35
#30	5–15
#50	0–7
#200	0–2

Retwal1A

Geolabs – Westlake Village
GEOLOGY AND SOIL ENGINEERING

DATE 12.2.05 BY SBS
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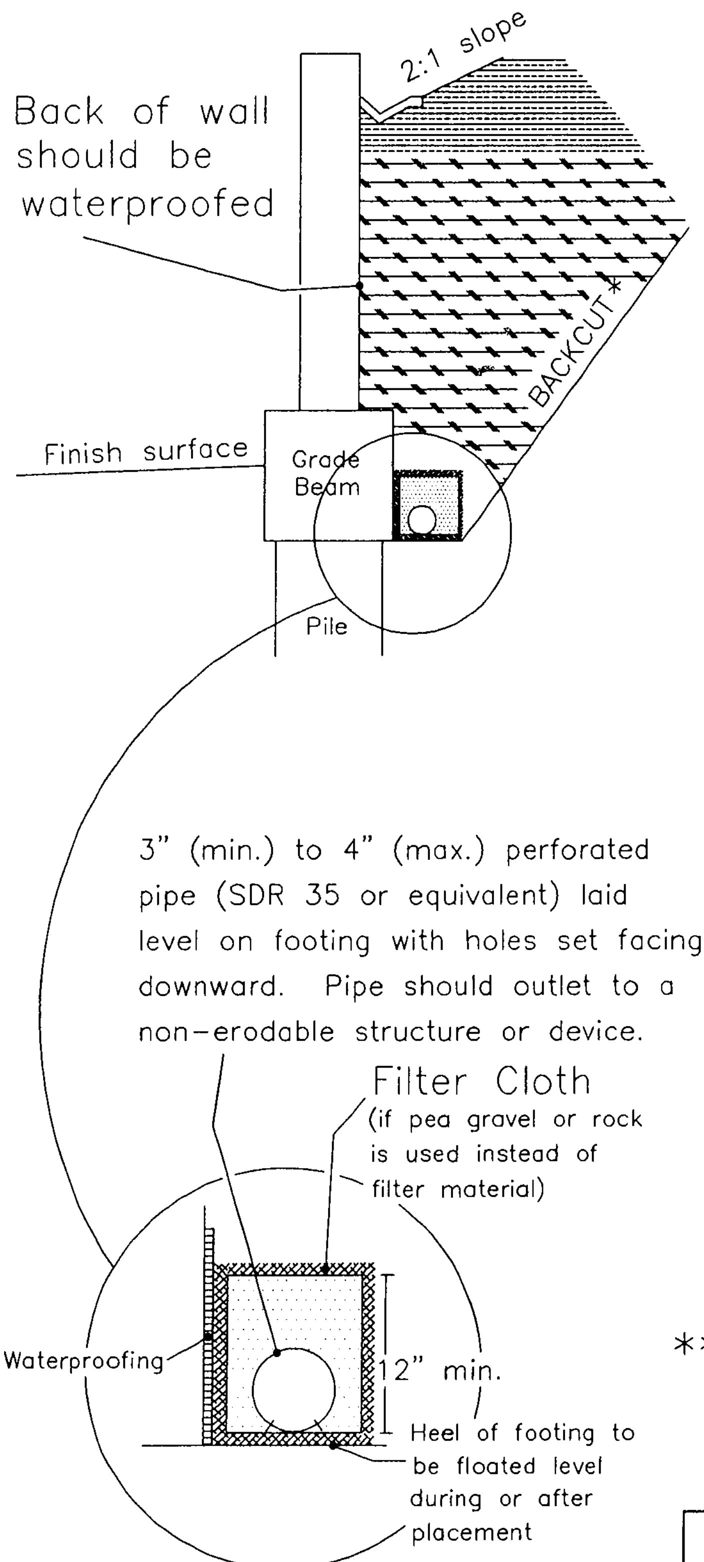
PLATE RW

TYPICAL PILE AND GRADE BEAM RETAINING WALL



Relatively impermeable backfill soil

Rock or non-expansive soil backfill (EI <20 or SE>20).**
If rock is used, Filter Cloth is required to separate rock from relatively impermeable soil and backfill. Non-expansive backfill should be placed to the backcut or three feet, whichever is less.
NOTE: All backfill should be compacted to a minimum of 90% relative density.

FILTER MATERIAL (see gradation), PEA GRAVEL, OR ROCK – Geotextile should be used to separate Pea Gravel or Rock from backcut and backfill.

FILTER MATERIAL GRADATION

Sieve Size	% Passing
1'	100
3/4'	90-100
3/8'	40-100
#4	24-50
#8	15-35
#30	5-15
#50	0-7
#200	0-2

* All backcuts shall be in accordance with OSHA standards, unless site-specific backcut and/or backfill recommendations are made by this office.

** EI 21-30 may be used if placed at 2% over optimum

Retwal3A

Geolabs – Westlake Village
GEOLOGY AND SOIL ENGINEERING

DATE 12.2.05 BY SBS
SCALE NTS W.O. 8953

PLATE RW3

E-5

GEOLABS-WESTLAKE VILLAGE
Foundation and Soils Engineering, Geology
31119 Via Colinas, Suite 502 • Westlake Village, CA 91362
Voice: (818) 889-2562 (805) 495-2197
Fax: (818) 889-2995 (805) 379-2603
a dba of
R & R Services
Corporation

December 30, 2015
W.O. 8953

John C. Chiu, FLPM
c/o John W. Newton & Associates, Inc.
159 Moonsong Court
P.O. Box 471
Moorpark, California 93020

Attention: Mr. John Newton

SUBJECT: Response to City of Moorpark Incompleteness Letter, Tentative Tract 5739, Everett Street and Walnut Canyon Road, Everett Street Terraces Apartment Complex, City of Moorpark, California

Reference: Geolabs-Westlake Village, July 22, 2015; Update Geotechnical Investigation For Proposed Everett Street Terraces Apartment Complex, Everett Street and Walnut Canyon Road, City of Moorpark, California

Mr. Newton

In accordance with your request, we have prepared this letter-report to provide our response to the subject Incompleteness Letter by the City of Moorpark for Tentative Tract Map 5739. This letter, dated November 24, 2015, is attached for your convenience.

INCOMPLETENESS LETTER 11-24-2015

Comment - Page 3:

The requested update geotechnical report, referenced above, was previously submitted to the City of Moorpark. Discussion of Lot 14 can be found on Page 2 of the Proposed Project section. For your convenience, we have attached our updated geotechnical report (dated July 22, 2015).

CLOSURE

This geotechnical report has been prepared in accordance with generally accepted engineering practices at this time and location. No other warranties, either express or implied, are made as to the professional advice provided under terms of our agreement and included in this report.

We appreciate this opportunity to be of service. Please do not hesitate to contact the undersigned if you have any questions regarding this report.

Respectfully submitted,
GEOLABS-WESTLAKE VILLAGE

Joanna Nygren
Staff Geologist

Ronald Z. Shmerling
C.E.G. 1047
R.C.E. 35444

REGISTERED PROFESSIONAL ENGINEER
RONALD Z. SHMERLING
No. 35444
CIVIL
STATE OF CALIFORNIA

CAS: JN: jw

REGISTERED GEOLOGIST
RONALD Z. SHMERLING
NO. 1047
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

Enclosures: City of Moorpark Review Letter
Geolabs-Westlake Village Report Dated July 22, 2015..........Appendix A

XC: (4) Addressee

CITY OF MOORPARK

COMMUNITY DEVELOPMENT DEPARTMENT | 799 Moorpark Avenue, Moorpark, California 93021
Main City Phone Number (805) 517-6200 | Fax (805) 532-2540 | www.moorparkca.gov

November 24, 2015

Dr. John C. Chiu FLP-N
1001 Newbury Road
Thousand Oaks, CA 91320

RE: INCOMPLETENESS LETTER FOR RESIDENTIAL PLANNED DEVELOPMENT NO. 2005-02, TENTATIVE TRACT MAP 5739, GENERAL PLAN AMENDMENT NO. 2005-02, ZONE CHANGE NO. 2005-02, REQUESTING APPROVAL FOR CONSTRUCTION OF A SIXTY UNIT BUILDING ON 2.4 ACRES LOCATED AT THE NORTHEAST CORNER OF EVERETT STREET AND WALNUT CANYON ROAD, ON THE APPLICATION OF JOHN C. CHIU

Dear Dr. Chiu:

The City of Moorpark has reviewed your application resubmitted on October 27, 2015, for Residential Planned Development No. 2005-02, Tentative Tract Map 5739, General Plan Amendment No. 2005-02, and Zone Change No. 2005-02, requesting approval for construction of a sixty unit building on 2.4 acres located at the northeast corner of Everett Street and Walnut Canyon Road, and finds it remains incomplete at this time. Until such time as the application can be determined to be complete, the City's processing is being suspended.

On August 3, 2010, a list of outstanding completeness items was emailed to you, describing those items required to be submitted in order to determine the application complete for processing. Many of these items do not appear to have been addressed. That list is reiterated as follows:

Planning/Zoning Issues:

1. *Although the City Engineer finds that the drainage feasibility study and plans depict an acceptable concept for the drainage system from a technical perspective, the Community Development Department has determined that the detention basin design is not acceptable from a planning perspective and must be redesigned. This design creates an area for loitering and litter accumulation. A mechanical system which does not create a deep basin should be considered as an alternative. (Conceptual Grading and Drainage Plan)*

2. *An improvement plan for the realignment of Everett Street is needed. Remove Wicks Road realignment from plans and maintain Everett Street opened to Walnut Canyon Road. (Conceptual Grading and Drainage Plan, Site Plan)*

3. *HVAC and water heater, locations must be shown on the plan. (Site Plan, Floor Plan, Landscape Plan)*
4. *Detailed, fully dimensioned floor plans are needed for each unit type. (Site Plan, Floor Plan)*
5. *Confirm new ADA accessibility requirements with Moorpark Building Official and show on plan as applicable. (Site Plan)*
6. *Fully dimension the pool and spa areas, including changing rooms. (Conceptual Grading and Drainage Plan, Site Plan)*
7. *Show conceptual lighting locations and types. (Site Plan)*
8. *Remove monument sign and bus shelter from plans. (Site Plan, Landscape Plan)*
9. *Show how standard trash bins will fit and explain how trash hauler will remove and replace bins. (Site Plan)*
10. *Provide details and elevations of gazebo and all other accessory structures including trellises and fountains. (Elevations, Landscape Plan)*
11. *Show building height at several points, including highest overall height of the building, from lowest to highest point. (Elevations)*
12. *Show all roof vents. Use flat vents where possible. (Elevations)*
13. *Provide fencing details, colors, and materials. (Landscape Plan)*
14. *Fully dimension the off-street loading area for residents moving in and out and truck deliveries. (Site Plan)*

City Engineer Issues:

Provide a letter updating the Hydrology and Drainage Study, including Lot 14. Previous comments were provided as follows:

> *The drainage feasibility study and plans depict an acceptable concept for the drainage system on the site plan (See Planning/Zoning comment No. 1). When the project has received approval you will be required at final design to submit a drainage report based on Ventura County design standards showing the site does not produce post development storm water runoff quantities (Q50) that exceed the pre development conditions (Q10) and onsite storm water clarification and the capacity of downstream systems.*

Provide a letter updating the Traffic Study, including Lot 14 and removal of the Wicks Road connection. Previous comments were provided as follows:

1. *The traffic report states, "An extension of Everett Street is planned to be constructed from its current westerly terminus at Moorpark Avenue to Wicks Road. The Everett Street extension is to run parallel along the east side of Moorpark Avenue (Walnut Canyon Road) between its current westerly terminus to Wicks Road. Direct vehicular access to and from Moorpark*

Avenue and Everett Street or Wicks Road will no longer be provided." The entrance/exit is treated as if this is the case in the traffic report. However, this is not how the site is designed. The conceptual plan shows cars entering and exiting the project site in the middle of the merge between Everett Street and Walnut Canyon Road. This is cause for concern on this project.

2. *The traffic report must analyze traffic movements in and out of the site. It must show that traffic movements in and out of the site do not adversely impact traffic movements on Walnut Canyon Road. Include an analysis of potential for causing vehicles to back up into Walnut Canyon Road.*
3. *Include striping and signing plan for the site entrance and exit on Everett Street/Walnut Canyon Road (SR 23).*
4. *Include line of sight exhibits for vertical and horizontal lines of sight on Everett Street/Walnut Canyon Road (SR 23).*
5. *The final design must show that an on-site circulation corridor can accommodate movements necessary for access by a CA fire truck and the site can be entered and exited by a CA fire truck.*
6. *Show all proposed dedications on Everett Street and Walnut Canyon Road (SR 23).*
7. *Show sections extending across Everett Street at the entrance and at mid-block.*

Subdivision Map

1. *Show how lot merges will be accomplished.*
2. *Include legal descriptions.*
3. *Please verify all affected title reports have been submitted, including lot 14.*

Provide a letter updating the Geotechnical Study, including Lot 14. Previous comments were provided as follows:

Review of the geotechnical report did not reveal anything prohibitive to the conceptual design. The geotechnical review addressed the following items which could affect the project design. The following recommendations were included in the review:

1. *Faulting and Seismicity – The closest active fault is 750-feet north of the site and there is no danger that the ground will rupture. The report recommends that minimum structural design be in compliance with the UBC.*
2. *Hydro-consolidation Potential – There is potential for hydro-consolidation in the upper 5-7 feet. Over-excavation is therefore recommended.*
3. *Liquefaction Potential – Potential for liquefaction induced settlement due to a design level earthquake could be on the order of 3-½ inches in the southern portion of the site. Recommendations are made to the foundation system*

because of the potential settlement. Lateral spreading and surface manifestations are not anticipated.

4. *Slope Stability – Over-excavation and removal of the top 5-7 feet of soil and engineered fill is recommended.*

Based on recommendations by your soils engineer in the preliminary geotechnical report, it does not appear to prevent this project from being built as depicted in the conceptual plan as long as the recommendations are followed. As such a more thorough review by the City's geotechnical consultant during this preliminary entitlement process was not warranted based on recommendations asserted in the preliminary report. During final design, review of the geotechnical study will be required by the City's geotechnical consultant and it is possible the recommendations may change.

In your response, please submit a cover letter noting in detail how and where on the plans and supporting documents these comments have been addressed. The following additional corrections and additional information are required to be addressed at this time as well:

1. The project as redesigned (without the Wicks Road realignment) shows driveway access on Walnut Canyon Road. All access must be from Everett Street. The project traffic engineer should evaluate the appropriate distance of the driveways from the intersection of Everett Street and Walnut Canyon Road, since this intersection is to remain open. The existing and proposed street improvements and full driveway plans must be shown on the site plan, including the full right-of-way of all adjacent streets.

2. The redesigned project shows 59 units (previously 60). Please identify and explain all changes to the plan since its last submittal.

3. A brief review of the updated traffic study shows that it is deficient in that it is based on 60 units, not 59, and it is based on the previously proposed alignment of Wicks Road and the closing of Everett Street. The new access needs to be addressed (also see No. 1 above).

Please note that on December 2, 2015, the City Council will be considering your request to extend the timeframe for the validity of the General Plan Amendment Pre-Screening for this project from December 4, 2015 to March 31, 2016. While staff is supportive of this request, the decision will be ultimately be made by the City Council and may affect the timing by which additional information is needed to complete this application.

Sincerely,

Joseph Fiss
Planning Manager

C: Steven Kueny, City Manager
David A. Bobardt, Community Development Director
John Newton, John Newton & Associates, Inc.
Case File: RPD 2005-01
Chron

APPENDIX A

GEOLABS-WESTLAKE VILLAGE REPORT
DATED JULY 22, 2015

December 30, 2015
W.O. 8953

GEOLABS-WESTLAKE VILLAGE
Foundation and Soils Engineering, Geology
31119 Via Colinas, Suite 502 • Westlake Village, CA 91362
Voice: (818) 889-2562 (805) 495-2197
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a dba of
R & R Services
Corporation

July 22, 2015
W.O. 8953

John C. Chiu, FLPM
c/o John W. Newton & Associates, Inc.
159 Moonsong Court
P.O. Box 471
Moorpark, California 93020

Attention: Mr. John Newton

Subject: Update Geotechnical Investigation for Proposed
Everett Street Terraces Apartment Complex,
Everett Street and Walnut Canyon Road
City of Moorpark, California

References: Geolabs-Westlake Village, December 2, 2005; Preliminary Geotechnical Investigation, Proposed Everett Street Terraces Apartment Complex, Northeast Corner of Everett Street and Walnut Canyon
City of Moorpark, California

Mr. Chiu

The project site was investigated by Geolabs-Westlake Village in 2005 for entitlement process. The age of the referenced investigation report, which was previously found acceptable, requires an update of information, analyses, or findings that may be outdated due to changes in the site condition, analyses methodology, standard of practice, or building code changes. In accordance with your agent's request, we present herein updated geotechnical criteria to address future construction designs. We are presenting this report to update design criteria using methodologies in the 2013 California Building Code. The updated criteria include seismic ground motion values, conventional foundation and slab on grade (Green Code) design criteria, slope deformation, liquefaction, and retaining wall design criteria. A site geologic map showing the current development plan with previously defined geologic conditions is also included (see Plate1.2).

In order to perform the update, we have visited the project site and observed the surface conditions and reviewed the referenced report, current codes and local practices. The interested reader may consult the referenced Geolabs-Westlake Village report dated December 2, 2005 for a more thorough characterization of the onsite soil conditions. All recommendations and criteria presented in the referenced report remains applicable unless superseded herein.

SITE CONDITIONS

Based on our recent reconnaissance, the site remains in essentially the same condition as reported in our 2005 report. An exception is that previously observed older residential structures now no longer exist. It appears that concrete retaining walls have been constructed in areas where the previous structures may have retained the hillside.

PROPOSED PROJECT

The project addressed in the referenced report consisted of a terraced complex of 44 apartments with two levels of partially subterranean to subterranean parking. The project was to be accessed from Everett Street. Retaining walls up to 24 feet in height were proposed. The terraced pads were planned for approximate elevations 533 feet, 544 feet, 555 feet, and 564 feet. The highest proposed fill slope was to be approximately 6 feet, fronting Everett Street. No permanent cut slopes were proposed.

The current project is illustrated on the Site Plan prepared by Holmes Enterprises, Inc. (HEI), dated 26 May 2015. The general concept of the project remains the same. The three level project has extended westward approximately 100 feet onto a property that was not a part of the previous project. The project now incorporates a 15 foot rear setback, 5 foot side setbacks, and 10 foot wide utility easement. The terrace elevations differ somewhat from the previous design. Based on elevations noted on the HEI plans, the tallest wall appears to be 15.5 feet in the northwest corner of the project, and adjacent to the northern portion of the utility easement. The new design grade changes are considered to be insignificant, so no additional exploration or changes to our cross sections are deemed warranted at this time.

DISCUSSION AND RECOMMENDATIONS

Based on our review of the site conditions and relevant available documents, many of the previous recommendations and findings remain applicable. In our opinion the liquefaction, slope stability analyses remain applicable. The current California Building Code requires tall retaining walls be designed for seismic lateral earth pressures. We have supplemented our previous retaining wall recommendations to address this requirement. We offer the following updates to our previous recommendations.

SEISMIC GROUND MOTION VALUES - GENERAL PROCEDURE

For this report we provide seismic ground motion values in accordance with the 2013 CBC (California Building Code). This code addresses seismic design based on response spectra considering an earthquake with a 2% probability of exceedance in 50 years (2475 year return period). Seismic ground motion values were determined in accordance with the procedure within CBC §1613.3 using the U.S. Seismic Design Maps website provided by the USGS.

Output from the analysis is summarized herein.

Latitude: 34.288º Longitude: -118.882º	Factor/Coefficient	Value
Site Profile Type	Site Class	D
Short-Period MCE at 0.2s	S_s	2.760
1.0s Period MCE	S_1	0.966
Site Coefficient	F_a	1.0
Site Coefficient	F_v	1.5
Adjusted MCE Spectral Response Parameters	S_{ms} S_{m1}	2.760 1.448
Design Spectral Acceleration Parameters	S_{DS} S_{D1}	1.840 0.966
Peak Ground Acceleration	PGA_M	1.047

FOUNDATION SYSTEMS

For planning purposes, this section provides preliminary foundation recommendations for conventional foundations. Once specific building types and foundation loads and locations are known, project specific foundation recommendations can be prepared.

Conventional Foundations

Continuous or pad footings may be used to support the proposed structures. In order to achieve the capacities specified below, they should be founded a minimum of 12 inches into engineered fill, with the concrete placed against in-place, undisturbed material. Foundation design criteria are based, in part, upon the expansive properties of the materials anticipated to be present near the finished pad grade. The building pad will contain expansive soils (EI>20).

The parameters provided in the following table are our minimum design values for the pertinent expansion range. Some of these values are empirical in nature. The foundation and slab designer should evaluate and design the foundations for the effects of expansive soils. The final foundation and slab-on-grade configuration should contain details that are not less than the values provided. Laboratory testing to verify the expansive properties of the near-pad-grade materials should be performed at the completion of rough grading.

Pre-saturation guidelines are presented in the following table. Pre-saturation of the foundation soils should be initiated well before concrete is scheduled to be placed. Care should be taken to see that the water has properly penetrated the soil. Last minute flooding is not a good practice. Excess water remaining in the target pre-saturation zone at the time of concrete placement will penetrate further into the soil, possibly causing additional expansion and uplift of the curing concrete.

Expansion Index Range	**0 - 20**
Pre-moisten	12"
Footings(1)	
Allowable Bearing Capacity	1800 PSF(2)
Lateral Resistance	400 PSF/Ft(2) (3)
Maximum Lateral Resistance	2500 PSF(2) (3)
Coefficient of Friction	0.40
Minimum Embedment Into Foundation Material	12 inches
Minimum Embedment Below Adjacent Grade	24 inches)
Minimum Reinforcement	2 #4 bars; 1 near top, 1 near bot.
Slabs-On-Grade	
Thickness	Full 4"
Minimum Reinforcement	#4 bars @ 16" o.c.e.w.

Expansion Index Range	**21 - 90**
Pre-saturation	18" (EI 21-50) 21" (EI 51-90)
Footings(1)	
Allowable Bearing Capacity	1500 PSF(2)
Lateral Resistance	250 PSF/Ft(2) (3)
Maximum Lateral Resistance	1800 PSF (2) (3)
Coefficient of Friction	0.3
Minimum Embedment Into Foundation Material	12 inches
Minimum Embedment Below Adjacent Grade	24 inches)
Minimum Reinforcement	2 #4 bars; 1 near top, 1 near bot.
Slabs-On-Grade	
Thickness	Full 4"
Minimum Reinforcement	#4 bars @ 16" o.c.e.w.

(1) Bearing portions of all footings should be at least five feet (measured horizontally) from the face of adjacent, descending slopes. All footings should bear at least three feet below an imaginary plane projected upward at 1.5:1 from the toe of locally over-steepened slopes. Pad footings should be at least 24 inches square. Continuous footings should be at least 12 inches wide for one-story and 15

inches wide for two-story.

(2) May be increased by 1/3 for short duration loading such as by wind or seismic forces.

(3) Decrease by 1/3 when combined with friction.

(4) Applies to exterior footings. Depth must meet the CBC requirements for the specific level of stories supported.

(5) Dowel slab to exterior footing using #3 bars @ 32" on center, bent 3' into slab for EI=51-90..

SLAB-ON-GRADE SUBGRADE

Approximately four inches of sand for EI=21-90, or two inches of sand for EI 0-20, should be placed across the slab subgrade, with a vapor retarder placed on top of the sand in all areas where moisture penetration of the slab is undesirable. The vapor retarder should consist of at least 10 mil thick, polyolefin plastic that complies with specifications in the present version of ASTM E1745. Concrete for the floor slab should be placed directly upon the vapor retarder.

The vapor retarder should be placed in general conformance with ASTM E1643 – 10. The permeance (propensity to transmit water) and strength (i.e. Class A, B or C) of the vapor retarder, as well as the water/cement ratio, mix design and strength of the concrete, will influence a variety of things, including slab finishing, construction schedules, moisture released from the slab, and floor coverings. Project design and construction professionals should consider these factors when developing specifications for, and/or selecting materials for, the vapor retarder, concrete, and floor covering.

RETAINING WALLS

Seismic Increment of Earth Pressure

As required by CBC §1803.5.12 geotechnical reports for structures assigned to Seismic Design Category D, E or F must include information regarding lateral pressures on foundation walls and retaining walls due to earthquake motions. Recent writings such as Lew et al. (2010) and Al Atik et al. (2010) attempt to address the appropriate means to implement this code requirement. These works conclude in part that seismic earth pressures can be neglected when the peak ground acceleration is equal to or less than 0.4g. For this site, the peak ground acceleration PGA_M is considered to be 1.05g .

For retaining walls, the following design criteria are provided considering the general provisional recommendations proposed by Lew et al. (2010) and findings presented in Al Atik (2010) for walls founded on non-saturated, level ground conditions. Lew et al. recommended the seismic earth pressure increment need only be included in design when wall height (H) exceeds 12 feet; however, 2013 CBC Section 1803.5.12 indicates that seismic lateral earth pressures be addressed for retaining walls supporting more than six feet of backfill, using design earthquake ground motions. When H meets this criterion, cantilever walls free to move and rotate can be designed for a seismic earth pressure increment considering an equivalent fluid pressure of **33** pcf (triangular pressure distribution). Walls restricted from moving or rotating, such as basement walls, can be designed for a seismic earth pressure increment considering an equivalent fluid pressure of **46** pcf (triangular pressure distribution). The resultant of this seismic earth pressure increment is considered to act at one-third H above the base of the wall. The seismic earth pressure increment should be applied to the active earth pressure for both the free-to-rotate and restrained cases. Often, for the case of walls restricted from moving or rotating, this combination of active earth pressure and seismic earth pressure increment will not exceed the at-rest earth pressure for the static case when considering factored loads used for the basic load combinations prescribed in the California Building Code.

CLOSURE

This geotechnical report has been prepared in accordance with generally accepted engineering practices at this time and location. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

Thank you for this opportunity to be of service. Please do not hesitate to call if you have any questions regarding this report.

Respectfully submitted,
GEOLABS-WESTLAKE VILLAGE

REGISTERED PROFESSIONAL ENGINEER
LAWRENCE K. STARK
No. 2772
GEOTECHNICAL
STATE OF CALIFORNIA

Lawrence K. Stark
G.E. 2772

Ronald Z. Shmerling
C.E.G. 1047
R.C.E. 35444

REGISTERED GEOLOGIST
RONALD Z. SHMERLING
NO. 1047
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

No. 35444
CIVIL
STATE OF CALIFORNIA

ENCLOSURE LIST:

Reference List....................................Plate R
Geologic Map....................................Plate 1.2 (in pocket)

XC: (3) Addressee
LKS:jr

REFERENCES

Geolabs-Westlake Village (2005), "Preliminary Geotechnical Investigation of 2-Acre Parcel, Northeast Corner of Everett Street and Walnut Canyon Road, Moorpark, California.

Al Atik, L, and Sitar, N. (2010). "Seismic Earth Pressures on Cantilever Retaining Structures." J. Geotech. Geoenviron. Eng., 136(10), 1324-1333.

Bray, J.B., Travasarou, T., Zupan, J. (2010). "Seismic Displacement Design of Earth Retaining Structures." Proc., 2010 Earth Retention Conference 3, ASCE, Bellevue, WA., 638-655.

Kramer, S.L. (1996). Geotechnical Earthquake Engineering. Prentice Hall, Upper Saddle River, New Jersey.

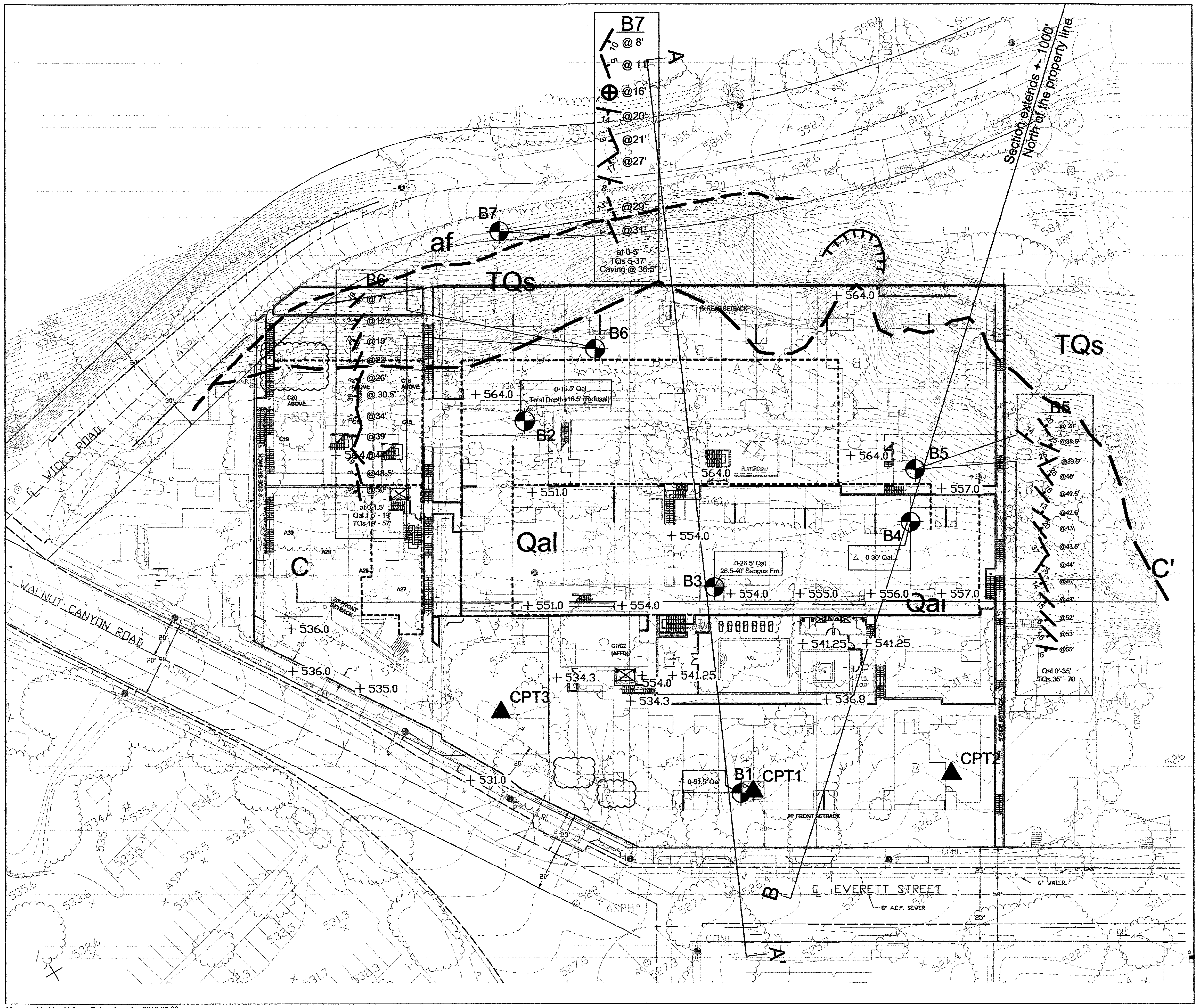
Lew, M., Sitar, N., Al Atik, L. (2010). "Seismic Earth Pressures: Fact or Fiction." Proc., 2010 Earth Retention Conference 3, ASCE, Bellevue, WA., 656-673.

Lew, M., Sitar, N., Al Atik, L., Pourzanjani, M., Hudson, M.B. (2010). "Seismic Earth Pressures on Deep Building Basements." Proc., SEAOC 2010 Convention, SEAOC, Indian Wells, CA.

GEOLOGIC MAP

EVERETT STREET TERRACES

City of Moorpark, CA



Map provided by: Holmes Enterprises, inc 2015.05.26

EXPLANATION

af	Artificial Fill
Qal	Alluvial deposits
TQs	Saugus Formation
— — —	Geologic contact
	Scarp area
B1	Boring location
CPT1	CPT location
45	Strike and dip of bedding
45	Strike and dip of fault/shear
45	Strike and dip of fracture
A — A'	Cross Section

40 20 10 0 40 80 120

1"=40'

P:\8953 Everett Terraces\8953 geo Update 2015.dwg (Layout: Geo Map 2015.07.22)

Geolabs – Westlake Village
GEOLOGY AND SOIL ENGINEERING

DATE 7/22/15 BY JAR
SCALE 1"=40' W.O. 8953

PLATE 1.2

GEOLABS-WESTLAKE VILLAGE
Foundation and Soils Engineering, Geology
31119 Via Colinas, Suite 502 • Westlake Village, CA 91362
Voice: (818) 889-2562 (805) 495-2197
Fax: (818) 889-2995 (805) 379-2603
a dba of
R & R Services
Corporation

January 22, 2021
W.O. 8953
Revised February 9, 2021

John C. Chiu, FLPM
C/o John W. Newton & Associates, Inc.
159 Moonsong Court
P.O. Box 471
Moorpark, California 93020

Attention: Mr. John Newton

SUBJECT: *Revised* Response to Third-Party Peer Review and Update Geotechnical Report, Everett Street Terraces Apartment Complex, Everett Street and Walnut Canyon Road, City of Moorpark, California

Mr. Chiu,

In accordance with your agent's request, Geolabs – Westlake Village (GWV) presents herein a *revised* response to the third-party peer review prepared by Haley & Aldrich, Inc. dated 20 November 2020. A copy of the review letter is provided in Appendix A. In addition, GWV presents updated geotechnical design criteria using methodologies addressing the 2019 California Building Code. The updated criteria include seismic ground motion values, conventional foundation design criteria, and retaining wall design criteria. Finally, an evaluation of feasibility of onsite stormwater infiltration is provided. Our finding is that onsite infiltration of stormwater is **not** feasible. A site geologic map showing the current development plan with previously defined geologic conditions is also included (see Plate 1).

This report was revised in response to email comments from the City Planning Department (Farley-Judkins, 2021) to make the response to review portion of the report more distinct from the update portion of the report. Revisions to the text are italicized to facilitate their identification. Note that review comments in the original version of this report were reiterated in bold italics, so though reiterated review comments remain in italics, they are not *revised.*

In order to perform the update, we have visited the project site and observed the surface conditions and reviewed the referenced reports, current codes and local practices. The interested reader may consult the referenced GWV report dated 2 December 2005 for a more thorough characterization of the onsite soil conditions. All recommendations and criteria presented in the referenced reports remains applicable unless superseded herein.

RESPONSE TO REVIEW

A copy of the review letter is provided in Appendix A. Review comments are reiterated in bold italics and responses are provided in the following sections.

Comment #1: ***Update Letter, Cover Page: Reference is made to the 2013 California Building Code (CBC). The 2019 CBC was adopted on 1 January 2020. Unless a waiver has been provided, methodologies from the 2019 CBC should be used.***

RESPONSE:

The reviewed report addressed the CBC edition that was current at the time. This report again updates our work to address the current CBC edition (2019). Findings and design criteria in this report supersede that in previous writings. *They are presented below under the "UPDATE REPORT" header. The updated criteria include seismic ground motion values, conventional foundation design criteria, and retaining wall design criteria.* Previous findings and design criteria in previous reports, not specifically addressed in the current writing, remain applicable.

Comment #2: ***Update Letter, Seismic Ground Motion Values: Recommendations for ground motion values were presented based on the 2013 CBC. Those values are derived based on America Society of Civil Engineers (ASCE) 7-10,[1] which has been updated with ASCE 7-16.[2] The methodologies should be updated to reflect current methodologies.***

RESPONSE:

This update report includes updated Seismic Ground Motion Values in accordance with ASCE 7-16 methodologies. Please see *pages 6 and 7* of this report.

Comment #3: ***Update Letter, Foundation Systems: The Update Letter states, "Once specific building types and foundation loads and locations are known, project specific foundation recommendations can be prepared." The Update Letter only includes preliminary recommendations and proposes that a final design-level study should be performed. Haley & Aldrich agrees that physical samples of the exposed soils should be collected after completing the rough grading to confirm the selection of the geotechnical recommendations.***

RESPONSE:

Acknowledged.

Comment #4: ***Update Letter, Conventional Foundations: It is recommended that the concrete be placed against in-place, undisturbed material. This appears to contradict earlier-referenced recommendations for overexcavation to address hydro-consolidation and slope stability, which implies that the foundations would be supported on engineered fill rather than in-place, undisturbed material.***

RESPONSE:

The authors of the reviewed geotechnical report understood that grading was going to occur, replacing native materials with engineered fill in areas to support conventional foundations. To clarify our design criteria, at the time the foundations are to be constructed, the engineered fill will be the in-place material within which the foundations will be embedded. The criteria that concrete is to be placed against in-place, undisturbed material is to be interpreted that foundations are to be constructed in excavations with compacted, undisturbed side walls consisting of engineered fill, as opposed to being constructed using forms and then backfilled against.

Comment #5: ***Update Letter, Conventional Foundations: Recommendations are provided that the expansive properties of the near-pad grade materials should be evaluated after completing the rough grading. The section then provides two separate sets of recommendations for various expansive conditions, including differing allowable bearing capacities, lateral resistance, and coefficients of friction. Haley & Aldrich agrees that physical samples of the exposed soils should be collected after completing the rough grading to confirm the selection of the geotechnical recommendations.***

RESPONSE:

Acknowledged.

Comment #6: ***Update Letter, Slab-on-Grade: The proposed 2- to 4-inch thickness of sand below the slab does not appear to be sufficiently thick enough to mitigate the potential for swell if the soils have a high expansive index. A detailed discussion should be provided.***

RESPONSE:

The design criterion for the sand below the slab-on-grade has been our standard for nearly four decades, and has been used successfully for hundreds of structures in Moorpark. This is based on Table 18-1-D-2 (formerly Table 29-A-2) that has been incorporated into ordinances for several jurisdictions within Ventura County. In the Moorpark Municipal Code it is

Table 1809.7 (http://qcode.us/codes/moorpark/misc/pdf-code/title15.pdf). The table has been attached to this response letter for your convenience.

Comment #7: ***Preliminary Report, Liquefaction-Induced Settlement Potential: Liquefaction analysis resulted in an estimated 3½ inches of settlement from a seismic event. This is a significant amount of potential settlement for a residential structure. New methodologies for evaluating earthquake parameters have also been developed that may modify the presented findings by Geolabs-Westlake Village. These are discussed under comments for Seismic Ground Motion Values above.***

RESPONSE:

Acknowledged.

Comment #8: ***Preliminary Report, Settlement: Total settlement of up to 4 ½ inches and differential settlement of up to 2 ¼ inches, including static and seismic conditions, were reported. These values exceed conventional limits of 1 inch of total settlement and ½ inch of differential settlement for most structures. Mitigation measures should be provided to reduce the settlement or the structural engineer should confirm that the proposed structures are capable of tolerating such excessive movement.***

RESPONSE:

For this writing we have updated both the ground motion parameters and the seismic settlement analyses considering new methodologies. Based on these updated evaluations, post-grading seismic settlement is estimated to be in the range of one to two inches. Differential seismic settlement can be assumed to be half the total settlement, ½ to 1 inch. For design purposes, this differential seismic settlement can be assumed to act over the horizontal distance 30 feet. This equates to distortion ratios of less than 0.003L (where "L" is the horizontal distance). This is within the 0.010L distortion ratio upper limit presented in the most recent guide, Table 12.13-3 of ASCE 7-16, for use to prevent structure collapse when designing shallow foundations for multi-story structures in risk Category II without concrete or masonry wall systems.

For the static condition, for planning purposes, structural foundation designs should consider total static settlement from foundation loads to be on the order of one inch, with differential settlement on the order of ½ inch over a horizontal distance of 30 feet. The combined anticipated static & seismic differential settlement equates to a distortion ratio of 0.004L which remains well below the upper limits of Table 12.13-3. We concur with the reviewer; the structural engineer should confirm the proposed structures are capable of tolerating this movement.

<u>Comment #9:</u> *Preliminary Report, Retaining Wall Recommendations: An allowable passive resistance of 600 pounds per square foot per foot with a factor of safety of 1.5 was provided. Based on the laboratory testing provided, this value exceeds the engineering properties of the soils. Additional justification should be provided for the recommended passive pressure.*

<u>RESPONSE:</u>

To clarify, retaining walls with conventional shallow foundations would use the 400 psf/ft lateral resistance provided on page 28 of the reviewed report. The 600 psf/ft lateral resistance applies to pile supported retaining walls. As noted in Caltrans Trenching and Shoring Manual (pages 6-9, 6-10), "passive resistance in front of an isolated pile is a three dimensional problem" and "the passive resistance in front of a pile calculated by classical earth pressure theories shall be multiplied by the adjusted pile width." The manual continues, "(F) or granular soils, if the pile spacing is 3 times the effective width (d) or less the arching capability factor may be taken as 3." This produces an adjusted pile width equal to the effective pile width (pile diameter) multiplied by the arching capability factor. The passive resistance provided for the piles in the reviewed report takes into account the adjusted pile width.

For example, using a soil with an internal friction, phi, of 27 degrees, the passive resistance estimated using the log spiral solution would be about 290 psf/ft considering a factor of safety of 1.5. Multiplying this value by the arching capability factor of 3 results in a passive resistance of over 850 psf/ft, well in excess of the recommended 600 psf/ft.

This concludes the response to review portion of this report.

<u>UPDATE REPORT</u>

The remaining portions of this report provide updates to our previous work that may be outdated due to changes in site condition, analysis methodology, standard of practice, or building code changes.

<u>SITE CONDITIONS</u>

Based on our recent reconnaissance, the site remains in essentially the same condition as reported in our 2015 report *(GWV, 22 July 2015)*.

PROPOSED PROJECT

The current project is illustrated on the Site Plan prepared by Holmes Enterprises, Inc. (HEI), dated 1 June 2020 (see Plate 1). It consists of a terraced complex of 60 condominium units with two levels of partially subterranean to subterranean parking. The project is to be accessed from Everett Street. Retaining walls up to 17 feet in height are proposed. The tallest wall is located in the northeast corner of the project. The terraced pads are planned for approximate elevations 535 feet, 541 feet, 554 feet, and 564 feet. The highest proposed fill slope is approximately 8 feet, fronting Everett Street. No permanent cut slopes are proposed. A 15-foot rear setback and 5-foot side setbacks are incorporated, as well as a north-south oriented, 10-foot-wide utility easement in the western portion of the site.

The general concept of the project remains the same as that described in our previous Update Report (GWV, 22 July 2015). The new design grade changes are considered to be insignificant, so no additional exploration or changes to our cross sections are deemed warranted at this time.

DISCUSSION AND RECOMMENDATIONS

Based on our review of the site conditions and relevant available documents, many of the previous recommendations and findings remain applicable. In our opinion the liquefaction and slope stability analyses remain applicable. Local policy has changed regarding the calculation of seismic earth pressures on retaining walls. We have revised our previous retaining wall recommendations to address this policy change. We offer the following updates to our previous recommendations *for seismic ground motion values, foundation systems, and retaining walls.*

Seismic Ground Motion Values – General Procedure

This report includes preliminary seismic ground motion values in accordance with the methodology of ASCE Standard 7-16. Seismic ground motion values were determined using the U.S. Seismic Design Maps website (https://seismicmaps.org) provided by OSHPD and SEA. These seismic design maps present data for a maximum considered earthquake ground motion, defined by an earthquake with a 2 percent probability of exceedance within a 50-year return period (recurrence interval of 2475 years). Output from these analyses are provided in Appendix B and summarized herein.

Latitude: 34.2880º Longitude: -118.8821º	**Factor/Coefficient**	**Value**
Site Profile Type	Site Class	D – Stiff Soil
Short-Period MCE at 0.2s	S_s	1.9
1.0s Period MCE	S_1	0.701
Site Coefficient	F_a	1.0
Site Coefficient	F_v	null
Adjusted MCE Spectral Response Parameters	S_{ms} S_{m1}	1.9 null
Design Spectral Acceleration Parameters	S_{DS} S_{D1}	1.266 null
Long-Period Transition Period	T_L	8.0 sec
Peak Ground Acceleration	PGA_M	0.911

Structures on soil profiles designated as Site Class D with S_1 values greater than or equal to 0.2, need not use site-specific ground motion values provided the value of the seismic response coefficient C_S is determined in accordance with the procedures in ASCE 7-16 §12.8.1.1 (per exception 2 of §11.4.8). The following parameters are considered appropriate for use in determining C_S per exception 2.

F_a	1.0	Site amplification factor at 0.2	
F_v	1.7	Site amplification factor at 1.0	
S_{MS}	1.9	Site-modified spectral acceleration value	(11.4-1)
S_{M1}	1.192	Site-modified spectral acceleration value	(11.4-2)
S_{DS}	1.266	Numeric seismic design value at 0.2 second SA	(11.4-3)
S_{D1}	0.795	Numeric seismic design value at 1.0 second SA	(11.4-4)

If the designer uses the simplified lateral force analysis procedure, §12.14.8 allows F_a to be taken as 1.0 for rock sites, or 1.4 for soil sites, for development of S_{DS}. Also, the value of S_S can be capped at 1.5 for development of parameters in accordance with §11.4.4. Sites are permitted to be considered rock if the soil thickness is no greater than 10 feet below the footing.

Foundation Systems

For planning purposes, this section provides preliminary foundation recommendations for conventional foundations. Once specific building types and foundation loads and locations are known, project specific foundation recommendations can be prepared.

Conventional Foundations

Continuous or pad footings may be used to support the proposed structures. In order to achieve the capacities specified below, they should be founded a minimum of 12 inches into engineered fill, with the concrete placed against in-place, undisturbed material. Foundation design criteria are based, in part, upon the expansive properties of the materials anticipated to be present near the finished pad grade. The building pad will contain expansive soils (EI>20).

The parameters provided in the following table are our minimum design values for the pertinent expansion range. Some of these values are empirical in nature. The foundation and slab designer should evaluate and design the foundations for the effects of expansive soils. The final foundation and slab-on-grade configuration should contain details that are not less than the values provided. Laboratory testing to verify the expansive properties of the near-pad-grade materials should be performed at the completion of rough grading.

Pre-saturation guidelines are presented in the following table. Pre-saturation of the foundation soils should be initiated well before concrete is scheduled to be placed. Care should be taken to see that the water has properly penetrated the soil. Last minute flooding is not a good practice. Excess water remaining in the target pre-saturation zone at the time of concrete placement will penetrate further into the soil, possibly causing additional expansion and uplift of the curing concrete.

FOUNDATION DESIGN PARAMETER	**DESIGN CRITERIA**			**UNITS**	**NOTES**
	EI = 0-20	**EI=21-50**	**EI=51-90**		
Pre-Saturation depth	12	18	21	in	
Allowable Bearing Capacity (net) (FS>3)	1800	1500	1500	psf	1,2
Allowable Lateral Resistance (FS=1.5)	400	250	250	psf/ft	2,3
Maximum Allowable Lateral Resistance	2500	1800	1800	psf	2,3
Coefficient of Friction (FS=1.0)	0.40	0.30	0.30		
Minimum Embedment Below Adjacent Grade	24	24	24	in	4
Minimum Embedment Into Supporting Material	12	12	12	in	
Minimum Reinforcement	2 - #4, 1 near top and 1 near bottom	2 - #4, 1 near top and 1 near bottom	2 - #4, 1 near top and 1 near bottom		
SLAB-ON-GRADE DESIGN PARAMETER					

FOUNDATION DESIGN PARAMETER	DESIGN CRITERIA			UNITS	NOTES
	EI = 0-20	EI=21-50	EI=51-90		
Minimum Concrete Thickness	4	4	4	in	
Minimum Reinforcement (On-Center-Each-Way)	#4 @ 16"	#4 @ 16"	#4 @ 16"		5

NOTES
1) Bearing portions of all footings should be at least five feet (measured horizontally) from the face of adjacent descending slopes. All footings should bear at least three feet below an imaginary plane projected upward at 1.5:1 from the toe of locally over-steepened slopes. Pad footings should be at least 24 inches square. Continuous footings should be at least 12-inches wide for on-story and 15-inches wide for two-story construction.
2) May be increased by 1/3 for short duration loading such as by wind or seismic forces.
3) Decrease by 1/3 when combined with friction.
4) Applies to exterior footings.
5) For EI>50, dowel slab to exterior footing using #3 bars @ 24" on-center each way bent 12" into footing, 36" into slab.

Slab-on-Grade Subgrade

Approximately four inches of sand for EI=21-90, or two inches of sand for EI=0-21, should be placed across the slab subgrade, with a vapor retarder placed on top of the sand in all areas where moisture penetration of the slab is undesirable. The vapor retarder should consist of at least 10 mil thick, polyolefin plastic that complies with specifications in the present version of ASTM E1745. Concrete for the floor slab should be placed directly upon the vapor retarder.

The vapor retarder should be placed in general conformance with ASTM E1643 – 10. The permeance (propensity to transmit water) and strength (i.e. Class A, B or C) of the vapor retarder, as well as the water/cement ratio, mix design and strength of the concrete, will influence a variety of things, including slab finishing, construction schedules, moisture released from the slab, and floor coverings. Project design and construction professionals should consider these factors when developing specifications for, and/or selecting materials for, the vapor retarder, concrete, and floor covering.

Retaining Walls

Seismic Increment of Earth Pressure

As required by CBC §1803.5.12 geotechnical reports for structures assigned to Seismic Design Category D, E or F must include information regarding lateral pressures on foundation walls and retaining walls due to earthquake motions. Recent writings such as Lew et al. (2010), Al Atik et al. (2010), and Agusti and Sitar (2013) attempt to address the appropriate means to implement this code requirement. These works conclude in part that seismic earth pressures can be neglected when the peak ground acceleration is equal to or less than 0.4g. For this site, the peak ground acceleration PGA_M is considered to be 0.911g.

For retaining walls, the following design criteria are provided considering the findings presented in Agusti and Sitar (2013) for walls founded on non-saturated, level ground conditions. Per CBC §1803.5.12 item 1, the seismic earth pressure increment need only be included in design when walls support more than six feet of backfill. When this criterion is met, cantilever walls free to move and rotate can be designed for a seismic earth pressure increment considering an equivalent fluid pressure of **27** pcf (triangular pressure distribution). Walls restricted from moving or rotating, such as basement walls, can be designed for a seismic earth pressure increment considering an equivalent fluid pressure of **43** pcf (triangular pressure distribution). The resultant of this seismic earth pressure increment is considered to act at one-third H above the base of the wall. The seismic earth pressure increment should be applied to the active earth pressure for both the free-to-rotate and restrained cases. Often, for the case of walls restricted from moving or rotating, this combination of active earth pressure and seismic earth pressure increment will not exceed the at-rest earth pressure for the static case when considering factored loads used for the basic load combinations prescribed in the California Building Code.

STORMWATER INFILTRATION

As discussed in the response to review comment 8 above, post-grading seismic settlement is estimated to be in the range of one to two inches. Saturation of the onsite soils by use of stormwater infiltration Best Management Practices (BMPs) may increase the potential magnitude of seismic settlement, which the reviewer has already pointed out to be significant in their comments 7 and 8. Due to the potential to cause increased seismic settlements, we consider onsite infiltration of stormwater to be infeasible.

CLOSURE

This geotechnical report has been prepared in accordance with generally accepted engineering practices at this time and location. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

Thank you for this opportunity to be of service. Please do not hesitate to call if you have any questions regarding this report.

Respectfully submitted,
GEOLABS-WESTLAKE VILLAGE

CERTIFIED ENGINEERING GEOLOGIST
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LKS:RP:af

ENCLOSURE LIST:

Reference List....................................R
Geologic Map ....Plate 1 (in pocket)
MMC Table 1809.7.............................Plate 2.1-2.2
Review Letter ....Appendix A
Seismicity ....Appendix B

XC: (3) Addressee

REFERENCES

Al Atik, L, and Sitar, N. (2010). "Seismic Earth Pressures on Cantilever Retaining Structures." J. Geotech. Geoenviron. Eng., 136(10), 1324-1333.

Agusti, G.C. and Sitar, N. (2013). "Seismic Earth Pressures on Retaining Structures in Cohesive Soils." Report submitted to the California Department of Transportation (Caltrans) under Contract No. 65A0367 and NSF-NEES-CR Grant No. CMMI-0936376. Report No. UCB GT 13-02.

Farley-Judkins, Shanna, February 5, 2021; "Re: Everett Street – Peer Review Comments." Message to John Newton. E-mail.

Geolabs-Westlake Village, December 2, 2005; Preliminary Geotechnical Investigation of 2-Acre Parcel, Northeast Corner of Everett Street and Walnut Canyon Road, Moorpark, California.

..., July 22, 2015; Update Geotechnical Investigation for Proposed Everett Street Terraces Apartment Complex, Everett Street and Walnut Canyon Road, City of Moorpark, California.

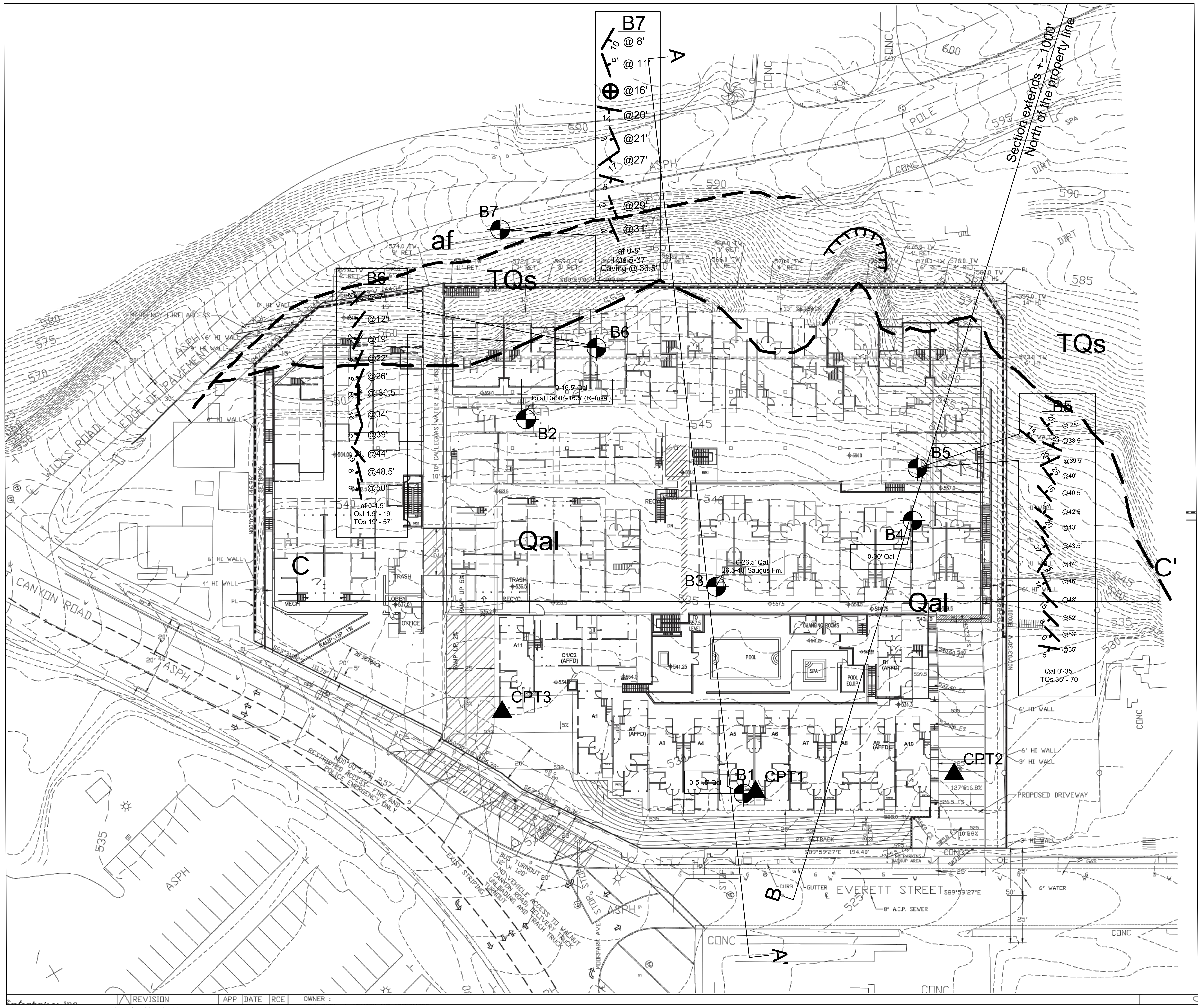
..., December 30, 2015; Response to City of Moorpark Incompleteness Letter, Tentative Tract 5739, Everett Street and Walnut Canyon Road, Everett Street Terraces Apartment Complex, City of Moorpark, California.

Geosyntec Consultants, June 29, 2018; Ventura County Technical Guidance Manual for Stormwater Quality Control Measures.

Haley & Aldrich, Inc., November 20, 2020; Third-Party Review of Preliminary Geotechnical Investigation and Update Letter, Everett Street Terraces Apartment Complex, City of Moorpark, California.

Lew, M., Sitar, N., Al Atik, L. (2010). "Seismic Earth Pressures: Fact or Fiction." Proc., 2010 Earth Retention Conference 3, ASCE, Bellevue, WA., 656-673.

Lew, M., Sitar, N., Al Atik, L., Pourzanjani, M., Hudson, M.B. (2010). "Seismic Earth Pressures on Deep Building Basements." Proc., SEAOC 2010 Convention, SEAOC, Indian Wells, CA.



GEOLOGIC MAP

EVERETT STREET TERRACES

City of Moorpark, CA

EXPLANATION

af	Artificial Fill
Qal	Alluvial deposits
TQs	Saugus Formation
	Geologic contact
	Scarp area
B1	Boring location
CPT1	CPT location
45	Strike and dip of bedding
45	Strike and dip of fault/shear
45	Strike and dip of fracture
A A'	Cross Section

40 20 10 0 40 80 120

1"=40'

Revised 2/9/2021

Geolabs – Westlake Village
GEOLOGY AND SOIL ENGINEERING

DATE 1/22/2021 BY RMP
SCALE 1"=40' W.O. 8953

PLATE 1

P:\8953 Everett Terraces\8953 geo Update 2015.dwg (Layout: Geo Map 2015.07.22)

Table 1809.7—Foundations for Stud Bearing Walls—Minimum Requirements[1,10,11,12]

Weighted expansion index	Foundation for slab and raised floor systems[2,5,7]							Concrete slabs		Pre-moistening of soils under footings, piers and slabs[5,6]	Restrictions on piers under raised floors
	No. of stories	Stem Thickness[8]	Footing width[9]	Footing thickness	All perimeter footings[6]	Interior footings for slab and raised floors[6]	Reinforcement for continuous foundations[3,8]	3-1/2″ minimum thickness 4″ with E.I. over 51			
					Depth below natural surface of ground and finish grade			Reinforcement[4]	Total thickness of sand		
		Inches									
0—20 Very low non expansive	1 2 3	6 6 10	12 15 18	6 7 8	12 18 24	12 18 24	1-#4 Top and bottom	#4 @ 48″ o.c. each way or #3 @ 36″ o.c. each way	2″	Moistening of ground prior to placing concrete is recommended	Piers allowed for single floor loads only
21—50 Low	1 2 3	6 8 10	12 15 18	6 7 8	15 18 24	12 18 24	1-#4 Top and bottom		4″	3% over optimum moisture required to a depth of 18″ below lowest adjacent grade. Testing required.	Piers allowed for single floor loads only
51—90 Medium	1 2	6 8	12 15	8 8	21 21	12 18	1-#4 top and bottom	#3 @ 24″ o.c. each way	4″	3% over optimum moisture required to a depth of 18″ below lowest adjacent grade. Testing required.	Piers not allowed
	3	10	18	8	24	24	#3 bars @ 24″ o.c. each way 12″ into footing, 36″ into slab[10]				
91—130 High	1 2	6 8	12 15	8 8	27 27	12 18	2-#4 top and bottom	#3 @ 24″ o.c. each way	4″	3% over optimum moisture required to a depth of 18″ below lowest adjacent grade. Testing required.	Piers not allowed
	3	10	18	8	27	24	#3 bars @ 24″ o.c. each way 12″ into footing, 36″ into slab[10]				
Above 130 Very high	Special design by a licensed Architect or Engineer required										

Footnotes to Table 1809.7:

1. Pre-moistening is required where specified in Table CBC 1809.7 in order to achieve maximum and uniform expansion of the soil prior to construction and thus limit structural distress caused by uneven expansion and shrinkage. Other systems, which do not include pre-moistening, may be approved by the building official, when such alternatives are shown to provide equivalent safeguards against the adverse effects of expansive soil.
2. Under-floor access crawl holes shall be provided with curbs extending not less than six (6) inches above adjacent grade to prevent surface water from entering the foundation area.
3. Reinforcement for continuous foundations shall be placed not less than three (3) inches above the bottom of the footing and not less than three (3) inches below the top of the stem.
4. Slab reinforcement shall be placed at mid-depth and continue to within two (2) inches of the exterior face of the exterior face of the exterior footing walls.
5. Moisture content of soils shall be maintained until foundations and piers are poured and a vapor barrier is installed. Test shall be taken within twenty-four (24) hours of each slab pour.
6. Crawl spaces under raised floors need not be pre-moistened except under interior footings. Interior footings which are not enclosed by a continuous perimeter foundation system or equivalent concrete or masonry moisture barrier shall be designed and constructed as specified for perimeter footings in Table CBC 1809.7.

7. A grade beam not less than twelve (12) inches by twelve (12) inches in cross-sectional area, reinforced as specified for continuous foundations in Table CBC 1809.7, shall be provided at garage door openings.
8. Foundation stem walls which exceed a height of three (3) times the stem thickness above lowest adjacent grade shall be reinforced in accordance with Sections 18 and 19 in the CBC, or as required by engineering design, whichever is more restrictive.
9. Footing widths may be reduced upon submittal of calculations by a registered civil or structural engineer or licensed architect, but shall be a minimum of twelve (12) inches for one and two-story structures and fifteen (15) inches for three-story structures.
10. Bent reinforcing bar between exterior footing and slab shall be omitted when floor is designed as an independent, "floating" slab.
11. Fireplace footings shall be reinforced with a horizontal grid located three (3) inches above the bottom of the footing and consisting of not less than No. 4 bars at twelve (12) inches on center each way. Vertical chimney reinforcing bars shall be hooked under the grid.
12. Underground utility conduits shall be installed prior to foundation inspection and shall extend beyond the foundation.

(Ord. 474 § 3, 2019)

PLATE 2.2

APPENDIX A
Review Letter

January 22, *2021*
W.O. 8953
Revised February 9, 2021

HALEY ALDRICH

HALEY & ALDRICH, INC.
5333 Mission Center Road
Suite 300
San Diego, CA 92108
619.280.9210

20 November 2020
File No. 135537-002

Chambers Group, Inc.
5 Hutton Centre Drive, Suite 750
Santa Ana, California 92707

Attention: Meghan Gibson
Project Manager/Senior Environmental Planner

Subject: Third-Party Peer Review of Preliminary Geotechnical Investigation and Update Letter
Everett Street Terraces Apartment Complex
City of Moorpark, California

Ladies and Gentlemen:

This letter summarizes Haley & Aldrich, Inc.'s (Haley & Aldrich) third-party review of the following geotechnical investigation documents prepared by Geolabs-Westlake Village, both completed under file 8953:

- "Preliminary Geotechnical Investigation, Proposed Everett Street Terraces Apartment Complex, Northeast Corner of Everett Street and Walnut Canyon Road, Moorpark, California," dated 2 December 2005 (Preliminary Report); and
- "Update Geotechnical Investigation for Proposed Everett Street Terraces Apartment Complex, Everette Street and Walnut Canyon Road, City of Moorpark, California," dated 22 July 2015 (Update Letter).

These documents were prepared to provide geotechnical considerations for the proposed development in Everett Street and Walnut Canyon in the City of Moorpark, California (the "Site"). A letter prepared by the City of Moorpark dated 24 November 2015 was also provided, including comments regarding the incompleteness of a development application.

The proposed development includes construction of a sixty-unit complex on a sloped, 2.4-acre property. The development is anticipated to include multi-story residential structures with subterranean basements. The residential structures are expected to be podium-style (wood frame over a reinforced concrete ground level) with slab-on-grade concrete floors and shallow, spread foundations. The preliminary investigation included advancing three cone penetration tests to unreported depths and seven borings of unreported methods and depths. The Update Letter addresses seismic ground motions, foundation systems, slabs-on-grade, and seismic parameters for retaining walls. As stated in the Update Letter, Geolabs-Westlake Village concluded that the primary concerns for the Site development include the potential for liquefaction induced settlement, the potential for settlement from collapsible soil, the presence of expansive soils, and slope stability.

Haley & Aldrich reviewed the Update Letter, and we have several comments as presented below:

- Update Letter, Cover Page: Reference is made to the 2013 California Building Code (CBC). The 2019 CBC was adopted on 1 January 2020. Unless a waiver has been provided, methodologies from the 2019 CBC should be used.
- Update Letter, Seismic Ground Motion Values: Recommendations for ground motion values were presented based on the 2013 CBC. Those values are derived based on America Society of Civil Engineers (ASCE) 7-10,[1] which has been updated with ASCE 7-16.[2] The methodologies should be updated to reflect current methodologies.
- Update Letter, Foundation Systems: The Update Letter states, "Once specific building types and foundation loads and locations are known, project specific foundation recommendations can be prepared." The Update Letter only includes preliminary recommendations and proposes that a final design-level study should be performed. Haley & Aldrich agrees that physical samples of the exposed soils should be collected after completing the rough grading to confirm the selection of the geotechnical recommendations.
- Update Letter, Conventional Foundations: It is recommended that the concrete be placed against in-place, undisturbed material. This appears to contradict earlier-referenced recommendations for overexcavation to address hydro-consolidation and slope stability, which implies that the foundations would be supported on engineered fill rather than in-place, undisturbed material.
- Update Letter, Conventional Foundations: Recommendations are provided that the expansive properties of the near-pad grade materials should be evaluated after completing the rough grading. The section then provides two separate sets of recommendations for various expansive conditions, including differing allowable bearing capacities, lateral resistance, and coefficients of friction. Haley & Aldrich agrees that physical samples of the exposed soils should be collected after completing the rough grading to confirm the selection of the geotechnical recommendations.
- Update Letter, Slab-on-Grade: The proposed 2- to 4-inch thickness of sand below the slab does not appear to be sufficiently thick enough to mitigate the potential for swell if the soils have a high expansive index. A detailed discussion should be provided.
- Preliminary Report, Liquefaction-Induced Settlement Potential: Liquefaction analysis resulted in an estimated 3½ inches of settlement from a seismic event. This is a significant amount of potential settlement for a residential structure. New methodologies for evaluating earthquake parameters have also been developed that may modify the presented findings by Geolabs-Westlake Village. These are discussed under comments for Seismic Ground Motion Values above.

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[1] ASCE (2010) Minimum Design Loads for Buildings and Other Structures. ASCE/SEI Standard 7-10

[2] ASCE (2016) Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI Standard 7-16

- Preliminary Report, Settlement: Total settlement of up to 4 ½ inches and differential settlement of up to 2 ¼ inches, including static and seismic conditions, were reported. These values exceed conventional limits of 1 inch of total settlement and ½ inch of differential settlement for most structures. Mitigation measures should be provided to reduce the settlement or the structural engineer should confirm that the proposed structures are capable of tolerating such excessive movement.
- Preliminary Report, Retaining Wall Recommendations: An allowable passive resistance of 600 pounds per square foot per foot with a factor of safety of 1.5 was provided. Based on the laboratory testing provided, this value exceeds the engineering properties of the soils. Additional justification should be provided for the recommended passive pressure.

In summary, Haley & Aldrich recommends that additional services be performed. Design-level information should be updated to include current building codes and methodologies to evaluate the seismic hazards at the Site. In addition, laboratory testing should be performed after completing the rough grading to verify the properties of the near-pad grade materials. Finally, the structural engineer should confirm that the anticipated settlement under static and seismic conditions are within the tolerance of the structures or mitigation measures should be developed.

We appreciate the opportunity to provide our services to you on this project. If you have any questions or require any additional information, please call.

Sincerely yours,
HALEY & ALDRICH, INC.

Catherine H. Ellis

Catherine H. Ellis, PE, GE
Senior Associate, Geotechnical Engineer

Nancy E. Gardiner

Nancy E. Gardiner, CPESC, QSD, QISP
Senior Associate

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APPENDIX B
Seismicity

January 22, *2021*
W.O. 8953
Revised February 9, 2021

Everett Street Terraces

Latitude, Longitude: 34.2880, -118.8821

Date	12/29/2020, 2:58:08 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.9	MCE_R ground motion. (for 0.2 second period)
S_1	0.701	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.9	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.266	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.828	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.911	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.9	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.129	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.358	Factored deterministic acceleration value. (0.2 second)
S1RT	0.701	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.786	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.76	Factored deterministic acceleration value. (1.0 second)
PGAd	0.962	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.892	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.892	Mapped value of the risk coefficient at a period of 1 s

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https://earthquake.usgs.gov/hazards/interactive/

Search

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.85416217 g

Recovered targets

Return period: 2920.1363 yrs
Exceedance rate: 0.00034244977 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 6.97
r: 8.31 km
ε_0: 1.28 σ

Mode (largest m-r bin)

m: 7.52
r: 9.29 km
ε_0: 1.07 σ
Contribution: 19.06 %

Mode (largest m-r-ε_0 bin)

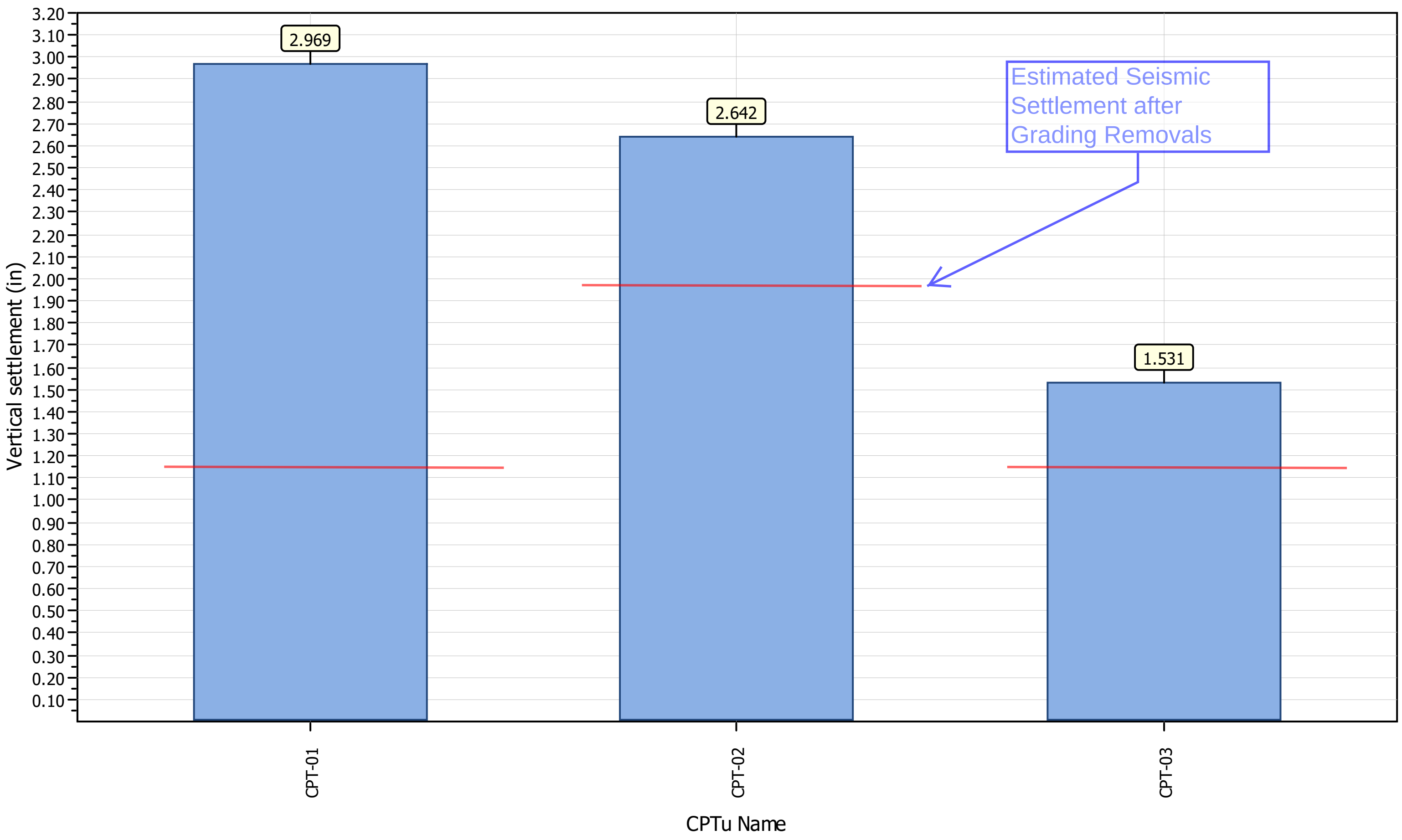
m: 7.52
r: 8.66 km
ε_0: 0.82 σ
Contribution: 10.99 %

GEOLABS - WESTLAKE VILLAGE
Engineering Geology and Soils Engineering
31119 Via Colinas, Suite 502
(818) 889-2562

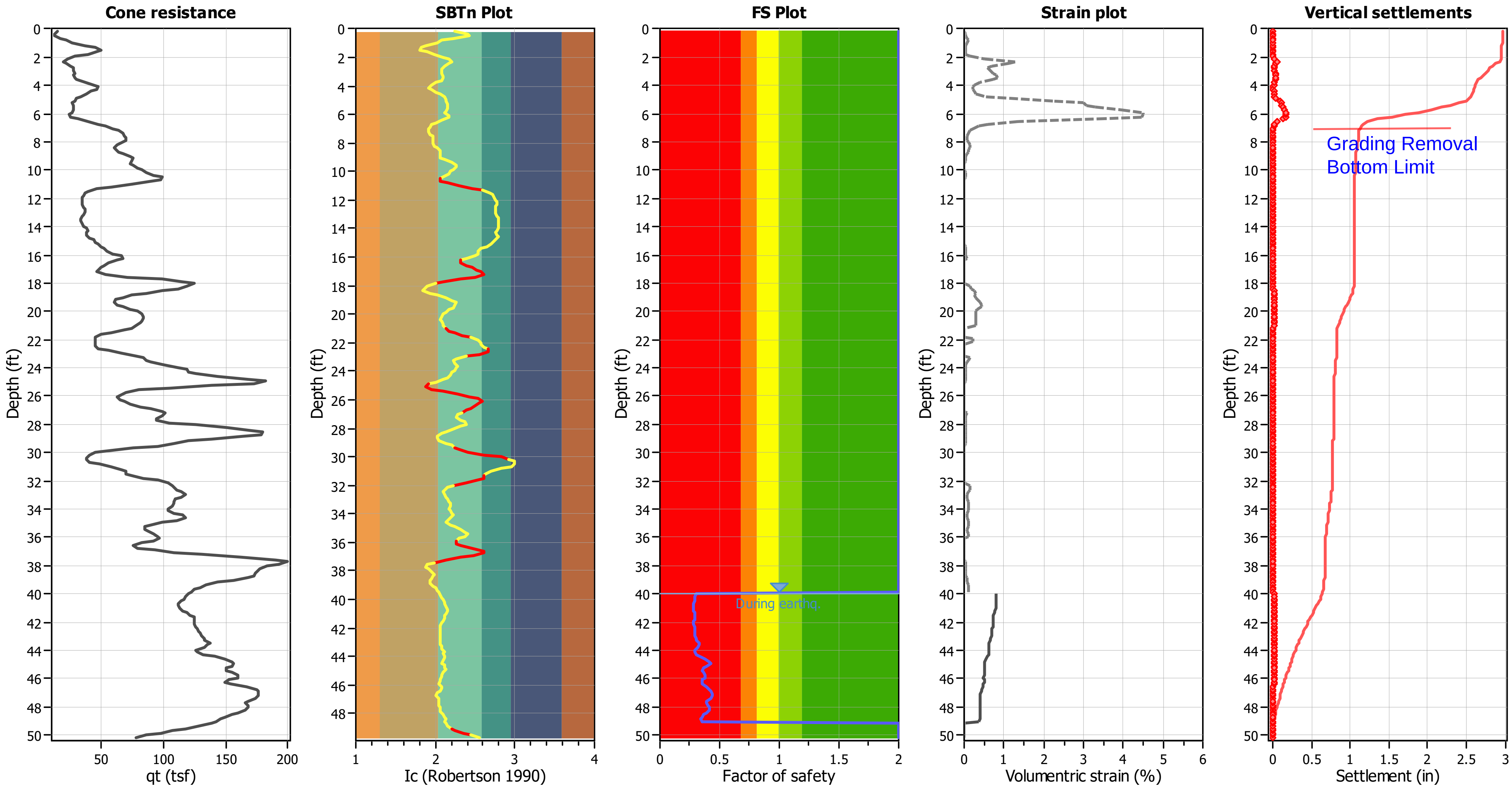
Project title :

Location :

Overall vertical settlements report



Estimation of post-earthquake settlements



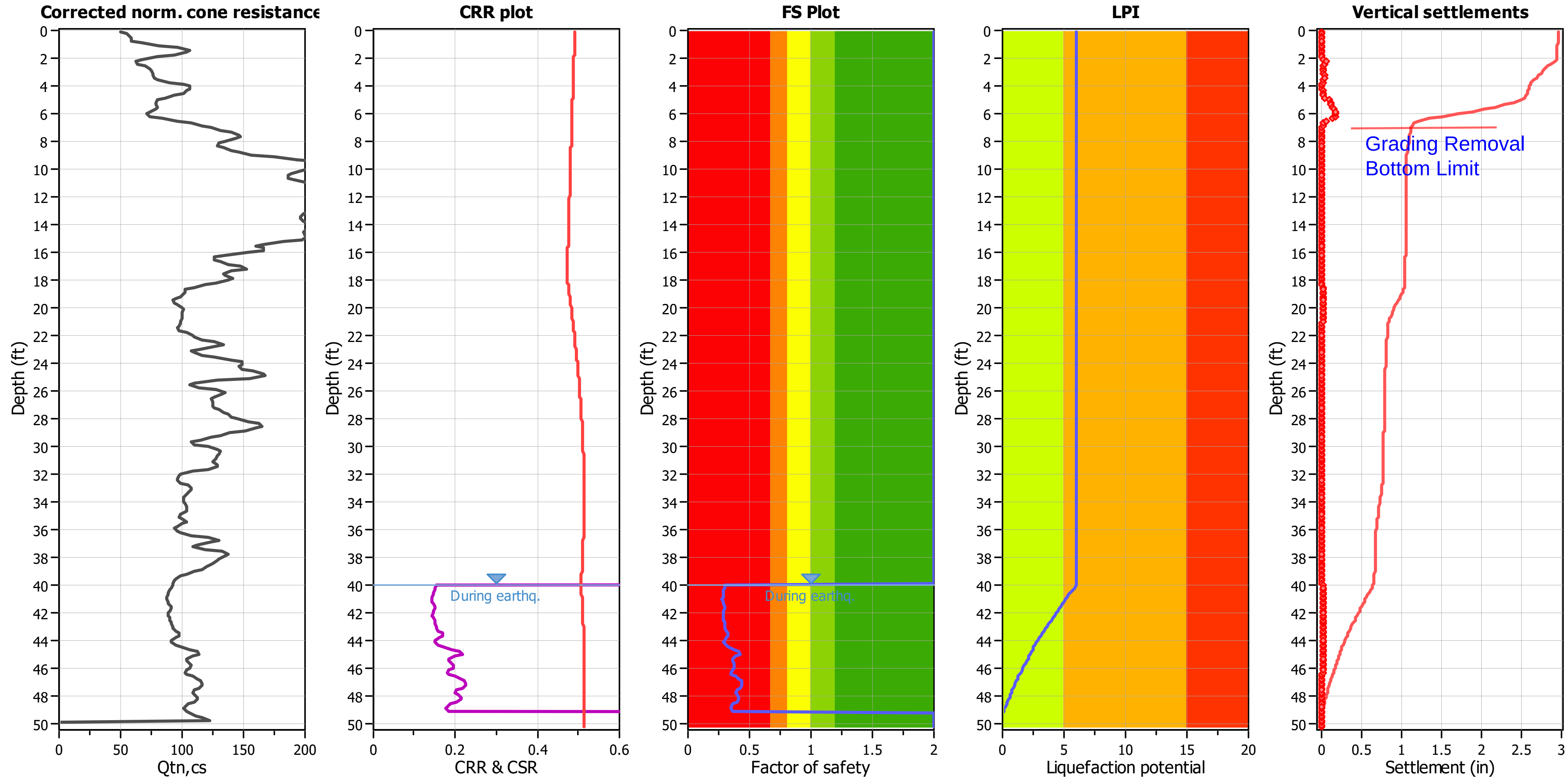
Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

GEOLABS - WESTLAKE VILLAGE
Engineering Geology and Soils Engineering
31119 Via Colinas, Suite 502
(818) 889-2562

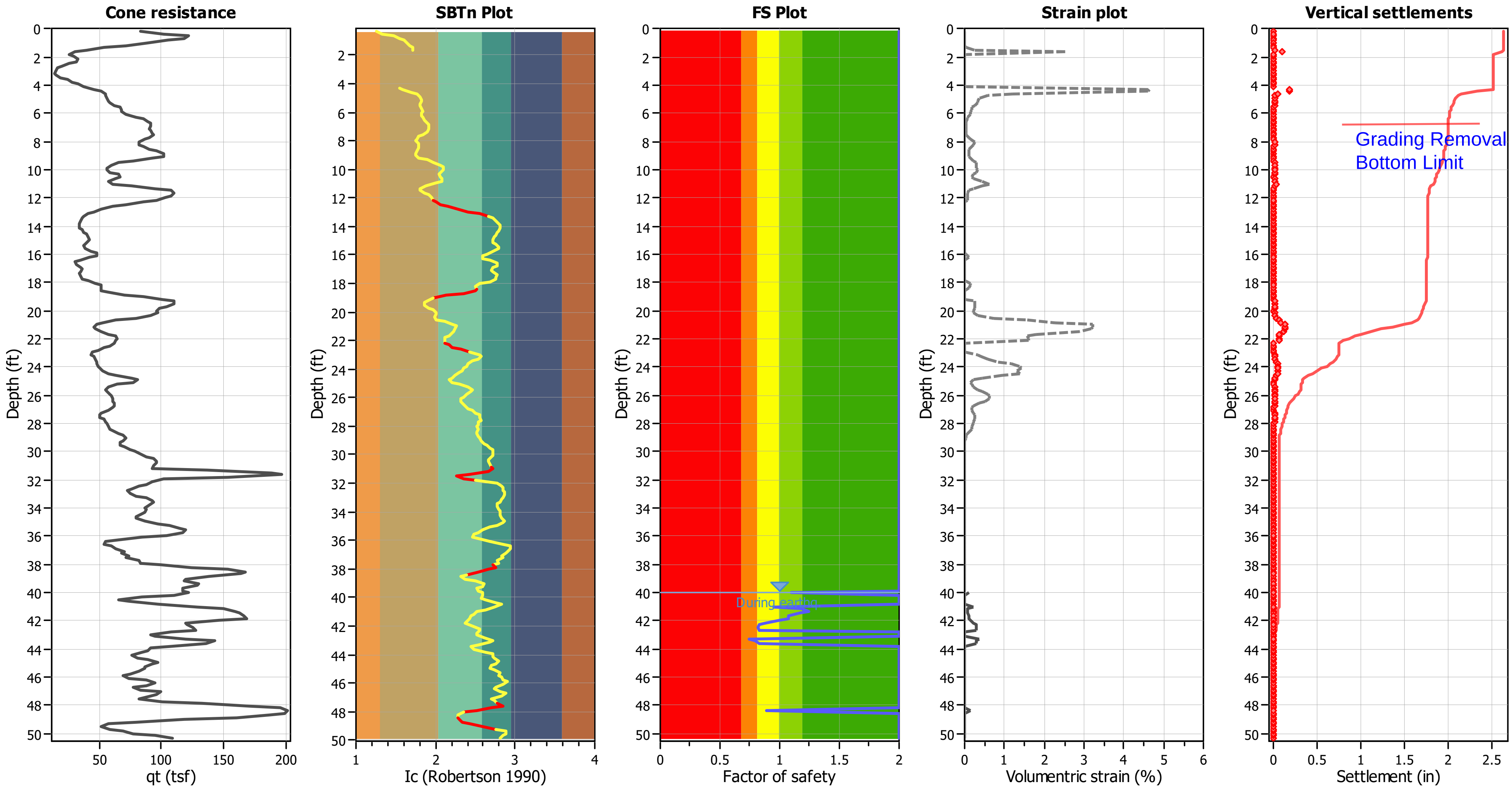
Project:
Location:

CPT: CPT-01
Total depth: 50.20 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	40.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.97	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

GEOLABS - WESTLAKE VILLAGE
Engineering Geology and Soils Engineering
31119 Via Colinas, Suite 502
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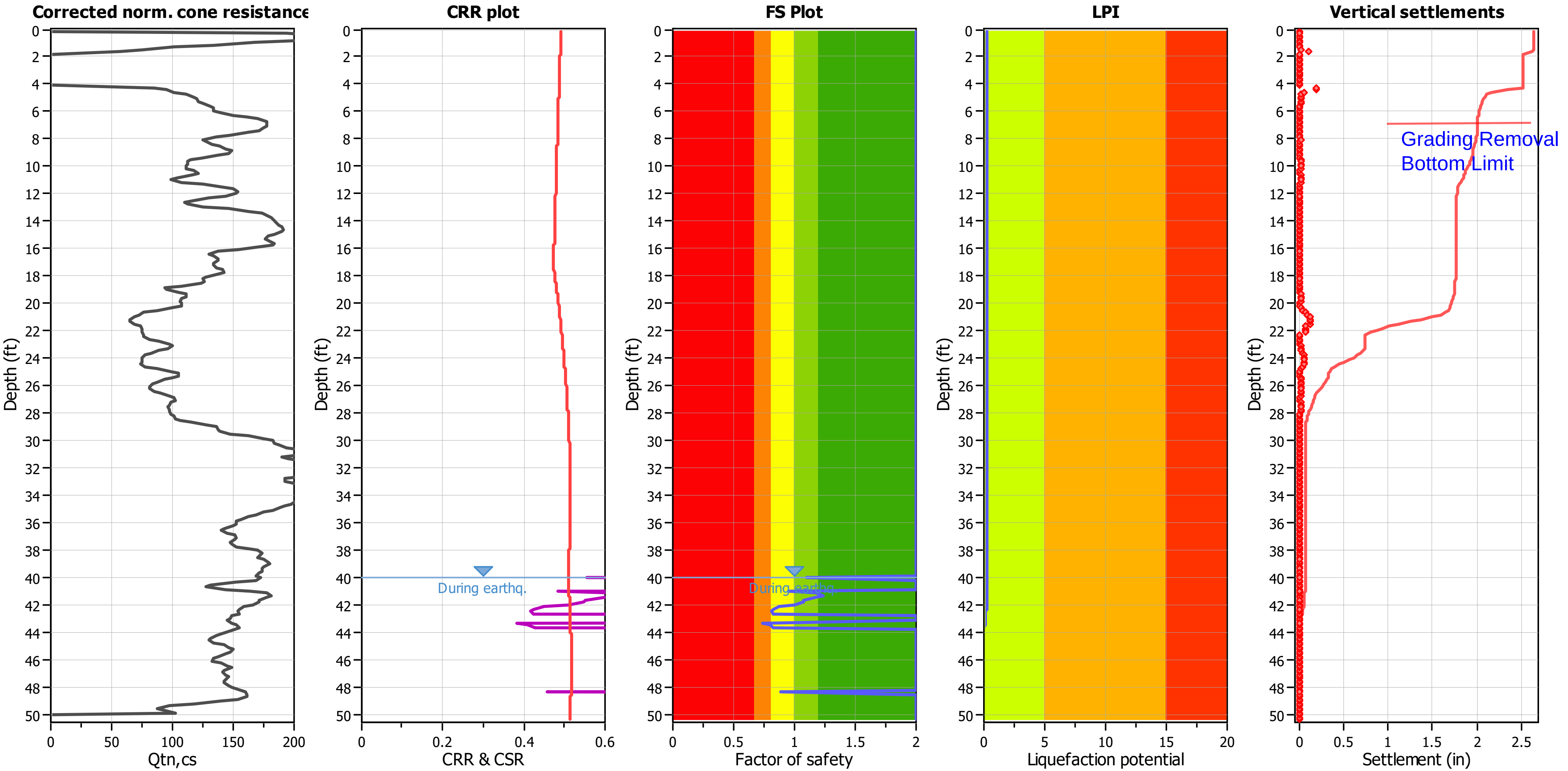
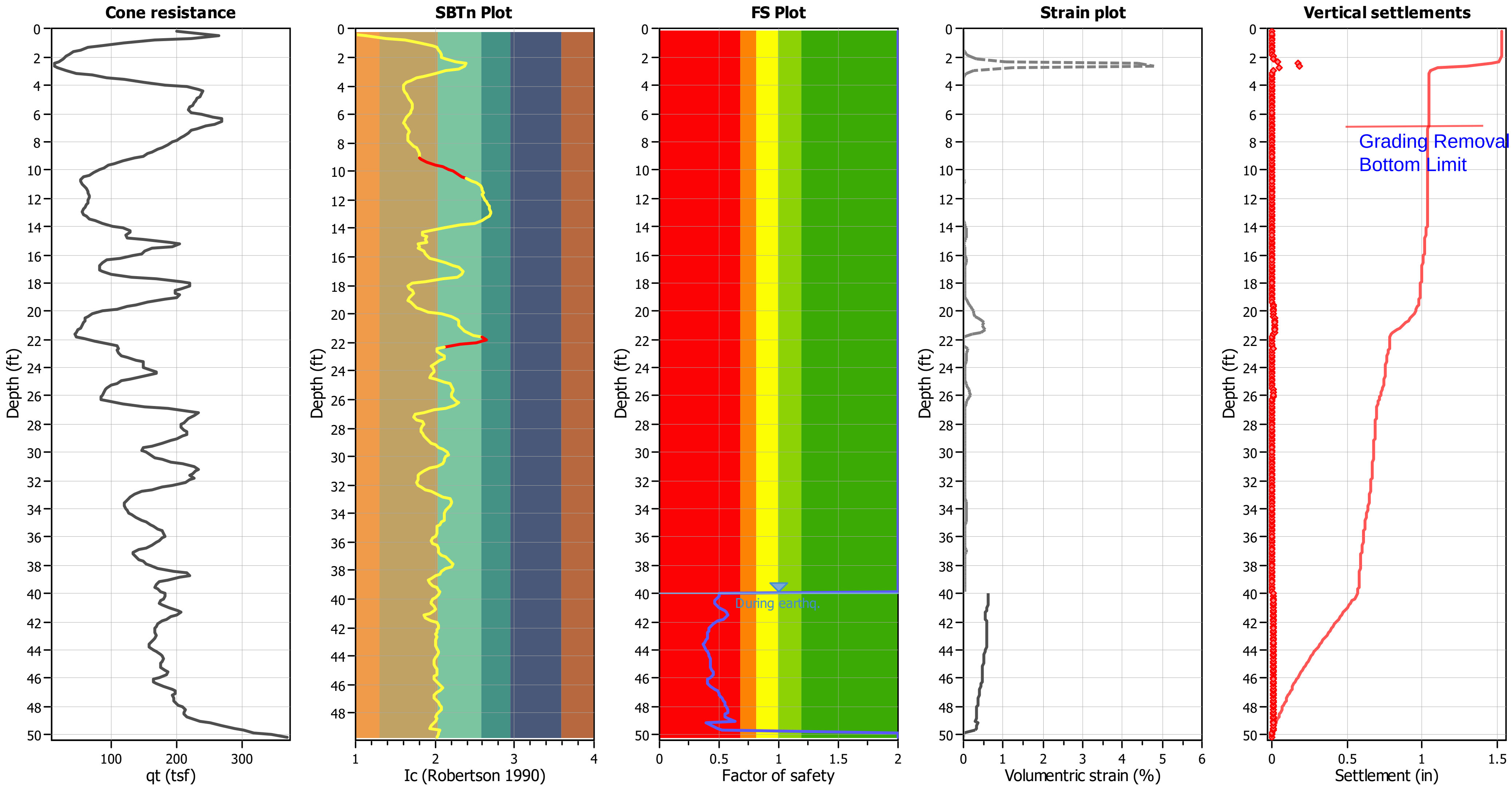
Project:
Location:

CPT: CPT-02
Total depth: 50.36 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	40.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.97	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

GEOLABS - WESTLAKE VILLAGE
Engineering Geology and Soils Engineering
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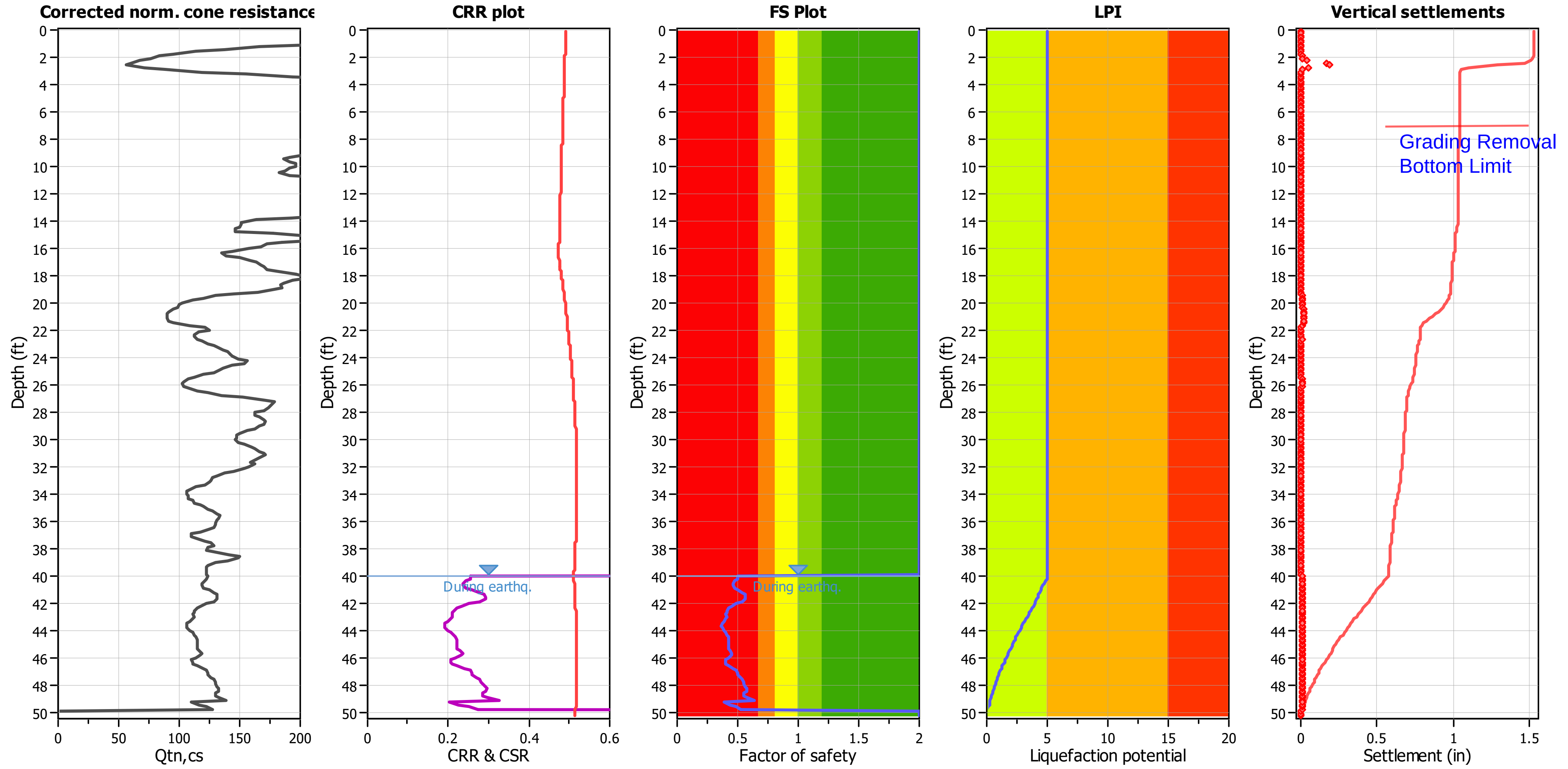
Project:
Location:

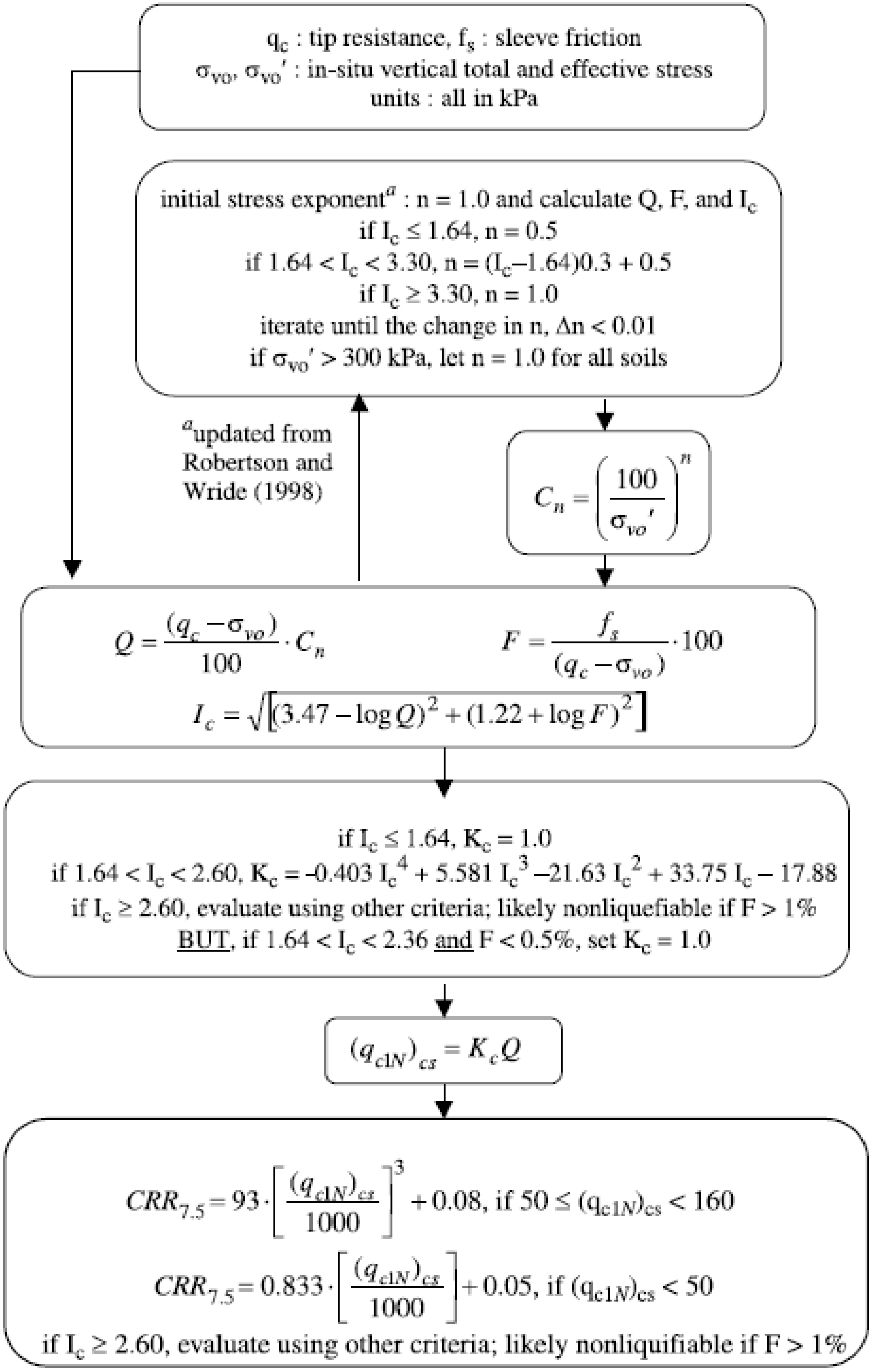
CPT: CPT-03
Total depth: 50.20 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	40.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.97	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot \tau_{a}$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

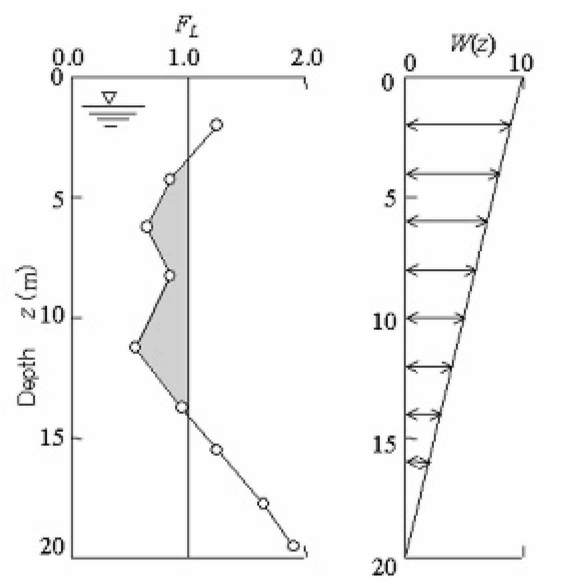
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_{0}^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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