Appendix C

Geotechnical Engineering Investigation

Geotechnologies, Inc.
Consulting Geotechnical Engineers

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

May 22, 2019
File Number 21671

Balboa Cove Group, LP
c/o Jack Nourafshan
6420 Wilshire Boulevard, Suite 1500
Los Angeles, California 90048

Attention: Jack Nourafshan

Subject: Update of Geotechnical Engineering Investigation
Proposed Mixed-Use Development
3443 South Sepulveda Boulevard, Los Angeles, California

Reference: *Report by Geotechnologies, Inc. (File No. 21086):*
Preliminary Geotechnical Engineering Investigation, dated March 18, 2016.

Dear Mr. Nourafshan,

This letter transmits the Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully Submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
REINARD T KNUR
No 2755
Exp. 12/31/20
GEOTECHNICAL
STATE OF CALIFORNIA

REINARD T. KNUR
G.E. 2755

RTK:km

Distribution: (3) Addressee

Email to: [jack@reliableprop.com], Attn: Jack Nourafshan
[Oscar@img-cm.com], Attn: Oscar Uranga

TABLE OF CONTENTS

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

UPDATE OF GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
3443 SEPULVEDA BOULEVARD
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of an updated geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to evaluate the distribution and engineering properties of the geologic materials underlying the site, and to provide geotechnical recommendations for the proposed development.

This investigation included drilling one boring, performing a percolation test, laboratory testing, obtaining and review of documents from the City of Los Angeles permit files, and preparation of this report.

This firm performed and earlier investigation that that included four borings, collection of representative samples, laboratory testing, engineering analysis, review of available geotechnical engineering information and preparation of a preliminary report dated March 18, 2016. The results of that report are incorporated into this report.

The site location is shown on the enclosed Vicinity Map, and the boring locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by Oscar Uranga of IMG Construction Management. The newly-proposed development has been modified to consist of seven stories over three levels of subterranean parking. The first two floors will provide retail establishments, parking, and miscellaneous building facilities (e.g. storage units, utilities, trash enclosures). The upper five floors will be comprised of residential units. Other improvements anticipated for this development include driveways for vehicular access, parking lots, and landscaping. The location of the proposed development relative to surrounding streets and structures is shown on the enclosed Plot Plan in the Appendix of this report.

Column loads are estimated to be between 300 and 1,100 kips. Wall loads are estimated to be between 5 and 20 kips per lineal foot. These loads reflect the dead plus live load, of which the dead load is approximately 75 percent. Grading will consist of excavations up to 42 feet in depth for the proposed subterranean parking levels and foundation elements.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The site is roughly rectangular in shape and approximately 2.7 acres in area. The site bounded to the north by an at-grade three level parking structure, to the east by Sepulveda Boulevard, to the south by Palms Boulevard and to the west by the 405 Freeway. The ground surface descends to the northwest ranging in elevation from 141 feet at the southeast corner to 132 feet at the northwest corner for a total elevation difference of 9 feet over a distance of 510 feet. The ground surface gradient is approximately 60 to 1 (horizontal to vertical).

It must be noted that the survey provided to this firm appears to have an elevation 90 feet less than that shown on the City of Los Angeles Topographic map. For purposes of discussion in this report, 100 feet was added to the elevations shown on the attached topographic map.

The site is developed with a single-story retail structure with at-grade paved parking surrounding the building. There is no vegetation on the site. Drainage occurs by sheetflow towards the northwest.

At-Grade Parking Structure

The parking structure on the north side of the site is 2-levels in height and is constructed at-grade. Based on review of the permit files from the City of Los Angeles, (described in the following section) the building is supported on conventional foundations bearing in compacted fill.

Palms Boulevard Overpass

Palms Boulevard bridges over the 405 Freeway on the south side of the site. A bridge support is located adjacent to the southwest corner of the site. Detailed plans showing the construction of the foundation elements for the overpass have not been provided to this office. The plans should be reviewed to determine the interaction of the footings and shoring between the proposed structure and the bridge.

405 Freeway

The 405 Freeway borders the site to the west and is found at elevation 110 to 115 feet. Therefore, the elevation difference between the site and the freeway ranges from 25 feet on the south side of the site to approximately 17 feet on the north side. From the site, a paved sloped surface descends to a retaining wall of variable height along the freeway. No information was obtained from the city or client files that describe the design of the retaining all along the freeway.

DOCUMENT REVIEW

This firm performed a document review on the site located to the north, 3415 South Sepulveda Boulevard. The documents are summarized below:

Converse Foundation Engineering Company, February 11, 1960, Proposed Kingpin Lanes Bowling Center 3415 South Sepulveda Boulevard and Rose Avenue, Los Angeles, California, Project No. 60-032-A.

The investigation included drilling 3 borings to depths of 25 and 31 feet. Boring 1 is shown on the attached Plot Plan. Fill soils consisting of Silty Clay and Clay extending to depths of 11 to 12 feet were found in each of the borings. Alluvium consists of interlayered silty clay and clayey silt was identified. Water was not identified in the borings

The report states that prior to 1956 the area was used as a dump. In 1956 the trash was removed and replaced with clean fill soils and compacted under the observation of Converse Foundation Company. A copy of the report is included in the Appendix.

Soils International, September 8, 1980, Preliminary Soil Investigation, File No L-1176-FG.

Three borings were drilled as part of this investigation to depths of 68 and 69 feet. Boring 1 is shown on the attached Plot Plan. The borings encountered fill soils that consist of sandy and silty clay that contains concrete fragments. The fill extended to depths ranging from 8 ½ to 12 ½ feet. The alluvium consist of interlayered clay, sand and silty gravel. The alluvium consists of Clay, silty sand silty gravel. The borings identified “bedrock” at depth of 45 to 55 feet. It is the opinion of this firm that geologic materials were misidentified and should be described as alluvium. Seepage was encountered in two of the borings at depths of 47 and 38 feet.

City of Los Angeles, September 12, 1980, Review Letter.

This letter was prepared to summarize the review of the Seismology Report included in the report by Soils International dated September 8, 1980. The letter approves only the soils-geology-seismology portion of the referenced report.

City of Los Angeles, September 30, 1980, Review Letter.

This letter states that approval of the foundation investigation portion of the report by Soils International dated September 8, 1980, after specific foundation recommendations are given. Downdrag effects on piles and differential settlement recommendations are specifically requested.

Soils International, January 8, 1991, Letter, File No. L-1776-F.

This letter presents foundations recommendations based upon the encountered soil type.

City of Los Angeles, February 17, 1981, Letter.

This letter approves of the report by Soils International dated September 8, 1980.

Soils International, March 4, 1981, Letter, File No. L-1776-F.

This letter provides responses to a City of Los Angeles Review Letter regarding unshored cuts and foundation recommendations.

Soils International, June 17, 1981, Letter, File No. L-1776-F.

This letter responds to another question regarding temporary cuts.

Soils International, January 24, 1983, Compaction Report, File no L-1776-I.

This report presents the results of compaction testing performed on the site. As part of grading, a pre-existing at-grade structure was demolished. The report states that the excavation extended to an elevation of approximately 108.8 feet. A plan showing the location of the compaction tests was not included in the report. A City of Los Angeles approval letter for the fill was not identified in the records.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was recently explored on March 13, 2019 by drilling one boring. For the previous investigation, the site was explored on October 28, 2015 by drilling four borings. The borings varied in depth from 30 to 100 feet. The borings were drilled with a truck-mounted rig equipped with 8-inch diameter hollowstem augers. Soil samples were taken at regular intervals with a California-modified split-spoon sampler lined with 2.5 inch diameter brass rings and standard penetration test equipment. The samplers were advanced with and automatic hammer dropping a 140 pound weight from a height of 30 inches. The boring locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-5. Cross Sections that show the subsurface distribution of the geologic materials are presented on Cross Sections A-A’ and B-B’

The boring locations were determined by measurement from hardscape features shown on the Plot Plan. Elevations were estimated by interpolation of the elevation contours shown on the Plot Plan. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

It must be noted that the survey provided to this firm appears to have an elevation 90 feet less than that shown on the City of Los Angeles Topographic map. For purposes of discussion in this report, 100 feet was added to the elevations shown on the attached topographic map.

Geologic Materials

The geologic materials consist of fill soil and natural alluvium.

Fill

The fill consists of sandy silt and silty clay, and silty sand with minor amounts of gravel. The fill is dark brown and grayish brown, moist and is medium dense and stiff. The fill extends to depths ranging from 3 to 17 1/2feet. This thickest fill was found in Borings B1 and B4 on the north side of the site.

Alluvium

The alluvium consists of silty sand, sandy silt and sand, and clayey silt. The alluvium is dark brown and grayish brown, moist and dense to very dense. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was not encountered within the 100-foot depth explored in March 2019 and the 70 foot depth explored in October 2015.

Based on a review of Seismic Hazard Evaluation Report for the Beverly Hills 7.5-Minute Quadrangle (CDMG, 2005), the historically highest groundwater level is on the order of 40 feet below grade.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during exploration due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

Percolation Testing

A percolation test was performed after the completion of drilling in Boring 1 from this investigation. The boring was drilled to a depth of 100 feet and no water was encountered. The boring was backfilled with cuttings to a depth of 74 feet and a 1-foot-thick layer of hydrated bentonite chips was added to the boring. A 2-inch diameter PVC pipe was inserted into the hole. The lower 20 feet of the pipe was perforated and the upper 55 feet was solid. A sand pack consisting of #3 Monterey Sand was poured into the annular space around the perforated portion of the casing.

After the casing was installed, the borehole was filled with water for the purpose of pre-soaking for a minimum of 4 hours. After presoaking, the borehole was refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 5 times or until a stabilized rate of drop was obtained, whichever occurred first. The percolation testing was performed within the native alluvial soils. . An uncorrected percolation rate of 12 inches per hour was measured.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject site is located in the Los Angeles Basin within the Transverse Ranges Geomorphic Province. The Transverse Ranges are characterized by roughly east-west trending mountains and the northern and southern boundaries are formed by reverse fault scarps. The convergent deformational features of the Transverse Ranges are a result of north-south shortening due to plate tectonics. This has resulted in local folding and uplift of the mountains along with the propagation of thrust faults (including blind thrusts). The intervening valleys have been filled with sediments derived from the bordering mountains.

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

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

Surface rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazards Maps of the State of California (CDMG, 1999), does not classify the site as part of the potentially “Liquefiable” area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. However, the liquefaction analysis was performed as required by the reviewing agency.

Groundwater was not encountered during exploration, conducted to a maximum depth of 100feet below the ground surface. The According to the Seismic Hazard Zone Report of the Beverly-Hills 7½-Minute Quadrangle (CDMG, 1998, Revised 2005), the historic-high groundwater level for the site is 40 feet below the ground surface. The historically highest groundwater level was conservatively utilized for the enclosed liquefaction analysis.

The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS websites, using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) and the U.S. Seismic Design Maps tool (USGS, 2013). A Site Class "D" (Stiff Soil Profile) and a published shear wave velocity of 230 meters per second were utilized for Vs30 (Tinsley and Fumal, 1985) in the USGS seismic programs. A modal magnitude (M_W) of 6.6 is obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration of 0.67g was obtained using the U.S. Seismic Design Maps tool. These parameters are used in the enclosed liquefaction analyses.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on Boring B3. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. Based on the collected SPT data, the enclosed liquefaction analysis indicates that the soils underlying the site would not be capable of liquefaction during the design-based earthquake.

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

Lateral Spreading

The enclosed liquefaction analysis included in the Appendix, indicates that site soils would not be capable of liquefaction during 2475 year return period ground motion. Therefore, lateral spreading is considered to be remote.

Dynamic Dry Settlement

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

Some seismically-induced settlement of the proposed structures should be expected as a result of strong ground-shaking.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Based on review of the Tsunami Inundation Map for the Beverly Hills Quadrangle (CalEMA, USC and CGS, 2009), indicates the site does not lie within a mapped tsunami inundation boundary.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within mapped inundation boundaries due to a breach in the Stone Canyon Dam upgradient reservoir.

A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

The site is underlain by fill soil and alluvium. The fill consists of sandy silt, clayey silt and silty clay that is dark brown moist and stiff. The maximum depth of fill identified in the borings was 17 1/2 feet. Deeper fill may occur elsewhere on the site. The fill soil was found to be deepest along the northern side of the site near the parking structure and appears to deepen towards the north. Natural alluvium consisting of interlayered silty sand sandy silt sand and silty clay underlies the fill soil. The alluvial soils are generally firm and dense. Groundwater was not identified within the 100 foot depth explored. The historically highest groundwater depth is 40 feet below grade.

The fill soil is not suitable for support of the proposed foundations, floor slabs or additional fill. Excavation of the proposed subterranean levels will remove the unsuitable materials in the building area.

The proposed structure may be supported on conventional foundations bearing in the alluvial soil that are anticipated at the subgrade elevation.

The elevation difference between the site and the freeway is as much as 25 feet. The footings should be deepened as appropriate on the west side, along the 405 freeway so as to not surcharge the retaining wall along the freeway. Conversely, additional information should be obtained from CALTRANS in order to identify the presence or absence of tiebacks on the site and to design for the surcharge pressure from the retaining wall, The surcharge from the adjacent 2 level parking structure to the north must be considered in the design of any retaining walls on the north side of the site. Additional detailed information regarding the depth and design of the footing supporting the Palms Avenue Bridge must be obtained in order to design the proposed shoring and footings

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

On-site stormwater infiltration may be performed beneath the structure. The drywell should begin percolation no less than 20 feet below the proposed footings.

SEISMIC DESIGN CONSIDERATIONS

2016 California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a “Stiff Soil” Profile, according to Table 20.3-1 of ASCE 7-10. This information and the site coordinates were input into the Structural Engineers Association OSHPD USGS U.S. Seismic Design Maps tool (Version 3.1.0) to calculate the ground motions for the site.

2013 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.960g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.960g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.307g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.696g
Site Coefficient (F_v)	Null
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	Null
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	Null

FILL SOILS

The maximum depth of fill encountered on the site was 17 ½ feet. This fill soil will be removed during the excavation for the subterranean levels and removed from the site.

EXPANSIVE SOILS

The onsite geologic materials are in the moderate expansion range. The Expansion Index was found to be 86 for a sample from Boring 1 taken from a depth of 1 to 5 feet and remolded to 90 percent of the laboratory maximum density. Reinforcing beyond the minimum required by the City of Los Angeles Department of Building and Safety is not required.

WATER-SOLUBLE SULFATES

The sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomena wherein soils lose volume when they are saturated. This can result in settlement of structures bearing thereon. The hydroconsolidation potential of the site soils was considered by assessing the consolidation tests of the undisturbed soil samples. The tests did not show collapse upon saturation of the sample. Based on the laboratory testing, it is the opinion of Geotechnologies, Inc. that the potential for damaging settlement due to hydrocollapse insignificant.

DEWATERING

The historic high groundwater level is approximately 40 feet below grade. Groundwater was not encountered within the 100 foot depth explored. The proposed basement of the structure will extend approximately 20 feet below grade, therefore the structure will not encounter groundwater. Therefore, a permanent dewatering system is not needed. However, in order to relieve hydrostatic pressure from nuisance water sources, the retaining wall will require drainage.

METHANE ZONES

Based on a review of the City of Los Angeles Methane and Methane Buffer Zones map, the subject site is not located within a Methane or Methane Buffer Zone (City of Los Angeles, 2003).

GRADING GUIDELINES

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. The soils tested by this firm may require the 95 percent compaction requirement.

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

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

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

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed.

Utility Trench Backfill

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

Wet Soils

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

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

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 10 and 15 percent should be anticipated when excavating and recompacting the existing fill on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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

LEED Considerations

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

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

The demolition debris should be limited to concrete, asphalt and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials and wood.

For structural fill applications, the materials should be crushed to 6 inches in maximum dimension or smaller. The crushed materials should be thoroughly blended and mixed with onsite soils prior to placement as compacted fill. The amount of crushed material should not exceed 20 percent. The blended and mixed materials should be tested by this office prior to placement to insure it is suitable for compaction purposes. The blended and mixed materials should be tested by Geotechnologies, Inc. during placement to insure that it has been compacted in a suitable manner.

FOUNDATION DESIGN

Conventional

Conventional foundations may bear in the natural alluvial soils found at the subgrade elevation. All conventional foundations for a structure should bear in the same material.

Continuous foundations may be designed for a bearing capacity of 4,400 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 5,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 250 pounds per square foot. The bearing capacity increase for each additional foot of depth is 250 pounds per square foot. The maximum recommended bearing capacity is 7,000 pounds per square foot.

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

Miscellaneous Foundations

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

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

Foundation Reinforcement

Based on City of Los Angeles minimum requirements all continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.38 may be used with the dead load forces.

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

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

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1.25 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed 0.5 inch.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory geologic materials, if necessary.

Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

Cantilever Retaining Walls (Active Pressure)

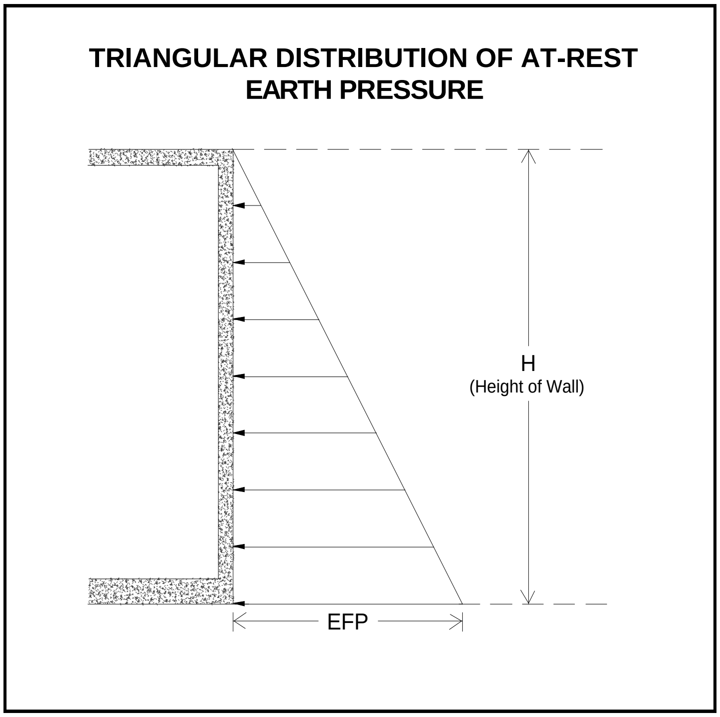
Cantilevered retaining walls supporting a level backslope may be designed utilizing an active pressure with a triangular distribution. Cantilever retaining walls may be designed for an Equivalent Fluid Pressure as identified in the following table.

HEIGHT OF CANTILEVERED RETAINING WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 20	39
20 to 30	45
30 to 40	48
40 to 50	50

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist an at-rest pressure with a triangular distribution as indicated in the diagram below. The at-rest pressure for design purposes would be 64 pounds per cubic foot. Additional earth pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.



In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to the parking structure to the north, the Freeway to the west, and street to the east and south, should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by existing buildings on the adjacent property.

Retaining Wall Drainage

Retaining walls should be provided with a subdrain covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. The onsite geologic materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent of the maximum density as determined by the most recent revision of ASTM D 1557.

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

Some municipalities do not allow the use of flat-drainage products. The use of such a product should be researched with the building official.

Where shoring will not allow the installation of a standard subdrainage system outside the wall rock pockets may be utilized. The rock pockets with should drain through the wall. The pockets should be a minimum of 12 inches in length, width and depth. The pocket should be filled with gravel. The rock pockets should be no more than 8 feet on center.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration to a depth of 1000 feet which corresponds to 60 feet below the base of the proposed structure. Therefore the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. It is considered improbable that groundwater level would rise to the subgrade elevation during the design life of the structure to affect the retaining wall backdrainage system. Therefore the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally the site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 21.2 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Surcharge from Adjacent Structures

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

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

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

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

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

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

Waterproofing

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

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Excavations on the order 45 feet in vertical depth will be required for the subterranean levels. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1 to 1 slope gradient. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

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

SHORING DESIGN

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

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Another method of shoring consists of steel soldier piles vibrated into place. Either of these methods is acceptable to Geotechnologies, Inc. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tied-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation may be assumed to be 400 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

Groundwater was not encountered during exploration to a depth of 100 feet below grade. It is not anticipated that groundwater will be encountered in the shoring pile excavations. However if seepage water greater than 3 inches of water accumulates at the bottom of the pile excavation, concrete placement will require the use of a tremie. A tremie shall consist of a water-tight tube having a diameter of not less than 4 inches connected to a concrete pump. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.38 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 250 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Lagging

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge.

A skin friction of 2,000 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

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

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

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

Anchor Installation

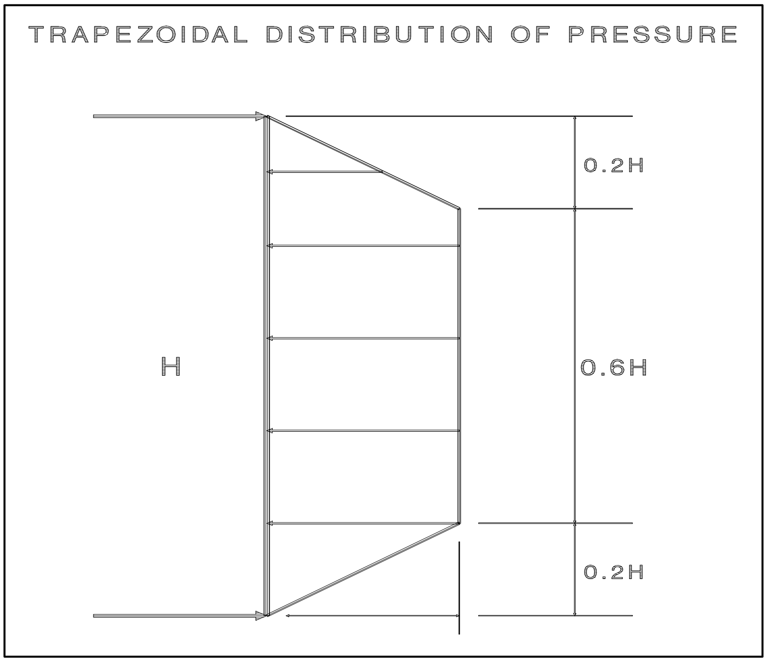
Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete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

Lateral Pressures

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

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 20	30
20 to 30	36
30 to 40	40
40 to 50	42

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs, with the trapezoidal distribution as shown in the diagram below.



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

HEIGHT OF SHORING "H" **(feet)**	**DESIGN SHORING FOR** **(Where H is the height of the wall)**
Up to 20	19H
20 to 30	23H
30 to 40	25H
40 to 50	26H

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

Deflection

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

The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 plane drawn upward from the base of the excavation.

Monitoring

Because of the depth of the excavation, some mean of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

Raker Brace Foundations

An allowable bearing pressure of 5,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 4 feet in width and length as well as 4 feet in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 4 inches in thickness. Slabs-on-grade should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 3 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way.

Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended based on an R-value for the subgrade of 20 and for the aggregate base of 60.

Service	**Asphalt Pavement Thickness Inches**	**Base Course Inches**
Passenger Cars (TI=5)	4	6
Moderate Truck (TI=6)	5	6.5

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform with Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), 1991 Edition.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

Concrete paving may be used on the project. Based on the highway design manual, for Traffic Index of 7 concrete paving should be 8 inches of concrete over 4 inches of compacted base.

The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer. Concrete paving should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

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

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

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

Introduction

Recently, regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Percolation Testing

Based on results of the percolation tests, a percolation rate of 12 inches per hour may be utilized for design purposes. It is recommended that stormwater should only percolate into natural alluvial soil. It should be noted that the recommended percolation rate is based on testing at the discrete locations and the overall percolation rate of the system could vary considerably.

Based on results of the percolation testing, a percolation rate of 12 inches per hour may be utilized for design of the proposed deep infiltration dry well systems. No safety factors or reduction factors have been applied to this percolation rate. The civil engineer must apply the required factors of safety to the percolation rate provided herein.

The Proposed System

The location for potential stormwater disposal has not been specifically addressed on this site. It is the opinion of this office that stormwater infiltration is possible on this site, however until the plan achieves more definition, and this office can address the impacts, stormwater infiltration is not recommended.

With regards to deep infiltration at the site, it is the opinion of this firm that any infiltration of stormwater in close proximity to structures should occur below the influence zone of the proposed foundations. Foundation influence zones would be expected to extend to depths correlating to roughly twice the width of the largest pad footing and approximately 4 times the width of wall footings. Assuming a typical 10 foot square pad footing that is founded at a depth of 45 feet, this would correlate to an influence depth of 20 feet below the bottom of pad footing, or approximately 65 feet below the ground surface.

The soils encountered on the site should allow stormwater to percolate in a generally vertical manner. Therefore, there is no potential for creating a perched water condition.

The soils are in the moderate expansion range. The onsite soils are not susceptible to significant hydroconsolidation.

The facility is not located in a hillside area and no slopes are nearby. The project will not be serviced by below grade retaining walls. No infiltration is planned into fill.

The Seismic Hazards Maps of the State of California (CDMG, 1999), does not classify the site as part of the potentially “Liquefiable” area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

Groundwater was not encountered during exploration, conducted to a maximum depth of 100 feet below the ground surface. According to the Seismic Hazard Zone Report of the Inglewood 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historic-high groundwater level for the site was 40 feet below the ground surface. The historic highest groundwater level was conservatively utilized for the enclosed liquefaction analysis.

A site specific liquefaction analysis was included with the referenced report. That analysis concluded that the liquefaction potential for the site was remote based on the design earthquake. It is, therefore, the opinion of this firm that the proposed infiltration of stormwater will not materially impact the liquefaction potential of the site.

Recommendations

The design and construction of stormwater infiltration facilities is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:

Open infiltration basins have many negative associated issues. Such a design must consider attractive nuisance, impacts to growing vegetation, impacts to air quality and vector control.

All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.

All connections associated with stormwater infiltration devices should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.

Excavations proposed for the installation of stormwater facilities should comply with the “Temporary Excavations” sections of this (the referenced) reports well as CalOSHA Regulations where applicable.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

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

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

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

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

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

The City of Los Angeles does not require corrosion testing. However, if corrosion sensitive improvements are planned, it is recommended that a comprehensive corrosion study should be commissioned. The study will develop recommendations to avoid premature corrosion of buried pipes and concrete structures in direct contact with the soils.

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Grain Size Distribution

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

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

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

Moisture and Density Relationships

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

Direct Shear Testing

Shear tests are performed by the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

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

Consolidation Testing

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

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000.

Laboratory Compaction Characteristics

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

REFERENCES

California Department of Conservation, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, California Geological Survey.

California Department of Conservation, Division of Mines and Geology, 1999, Seismic Hazard Zones Map, Beverly Hills 7½-minute Quadrangle, CDMG Seismic Hazard Zone Mapping Act of 1990.

California Department of Conservation, Division of Mines and Geology, 1998 (Revised 2005), Seismic Hazard Zone Report of the Beverly Hills 7½-Minute Quadrangle, Los Angeles County, California., C.D.M.G. Seismic Hazard Zone Report 023, map scale 1:24,000.

California Geological Survey, 2018, Earthquake Zones of Required Investigation, Beverly Hills Quadrangle, Map scale 1:24,000.

California Emergency Management Agency (CalEMA), USC, CGS, 2009, Tsunami Inundation Map for Emergency Planning. Map Scale 1:24,000.

City of Los Angeles, September 12, 1980, Review Letter.

City of Los Angeles, September 30, 1980, Review Letter.

City of Los Angeles, February 17, 1981, Letter.

City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.

Converse Foundation Engineering Company, February 11, 1960, Proposed Kingpin Lanes Bowling Center 3415 South Sepulveda Boulevard and Rose Avenue, Los Angeles, California, Project No. 60-032-A.

Dibblee, T.W. Jr., 1991, Geologic Map of the Beverly Hills Quadrangle, Division of Mines and Geology, DF Map #DF-31, Map Scale 1:24,000.

Leighton and Associates, Inc., 1990, Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.

Poland, J.F., Garell, A.A., AND Sinott, A., 1959, Geology, Hydrology, and Chemical Character of Groundwaters in the Torrance-Santa Monica area, California; U.S. Geological Survey, Water Supply Paper 1461.

SEOC/OSHPD, 2019, Seismic Design Maps Tool, https://seismicmaps.org/

REFERENCES - continued

Soils International, September 8, 1980, Preliminary Soil Investigation, File No L-1176-FG.

Soils International, January 8, 1991, Letter, File No. L-1776-F.

Soils International, March 4, 1981, Letter, File No. L-1776-F.

Soils International, June 17, 1981, Letter, File No. L-1776-F.

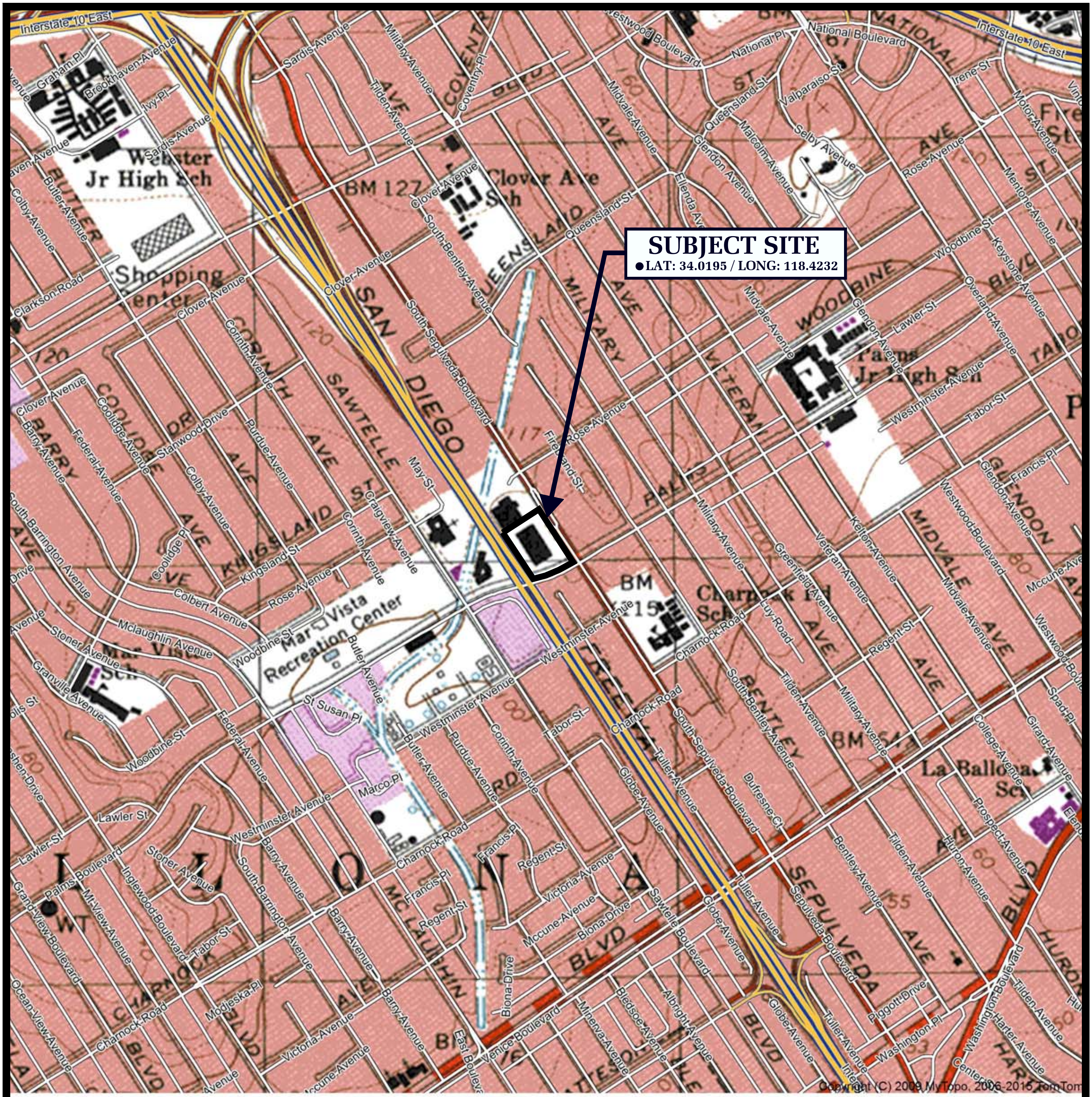
Soils International, January 24, 1983, Compaction Report, File no L-1776-I.

State of California Division of Oil, Gas, and Geothermal Resources, Online Mapping System, http://maps.conservation.ca.gov/doms/doms-app.html.

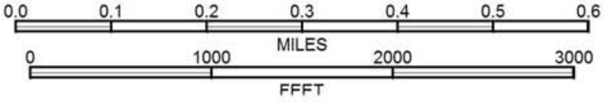
Stewart, J.P., Blake, T.F., and Hollingsworth, R.A., 2003, a screen analysis procedure of seismic slope stability: Earthquake Spectra, v. 19, n. 3, p. 697-712.

Tinsley, J.C., and Fumal, T.E., 1985, Mapping Quaternary Sedimentary Deposits for Areal Variations in Shaking Response, in Evaluation Earthquake Hazards in the Los Angeles Region- An Earth Science Perspective, U.S. Geological Survey Professional Paper 1360, Ziony, J.I. ed., pp 101-125

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E., Vedder, J.G., Geology of the Los Angeles Basin, Southern California-An Introduction, U.S. Geological Professional Paper 420-A.



SCALE 1:12000



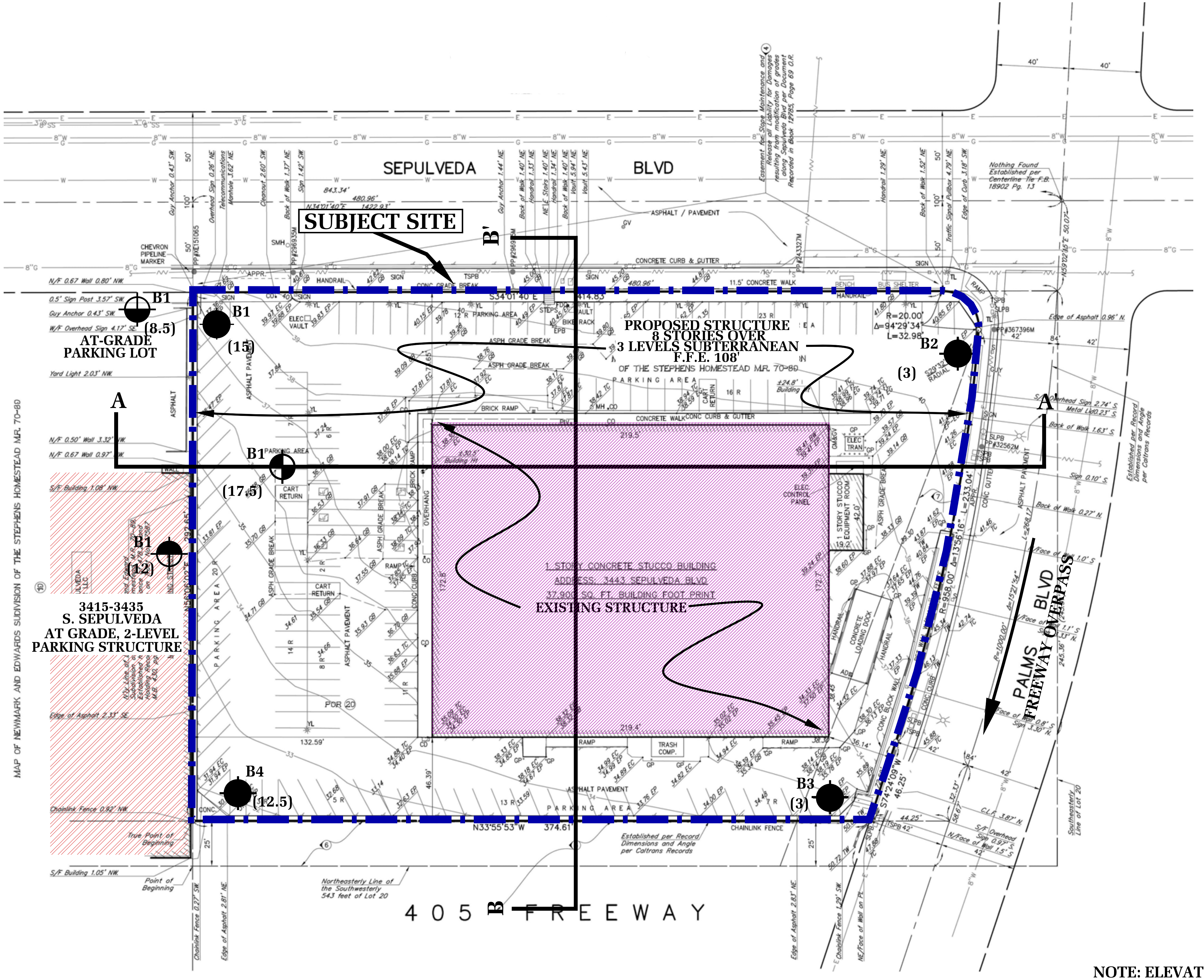
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
BEVERLY HILLS, CA QUADRANGLE

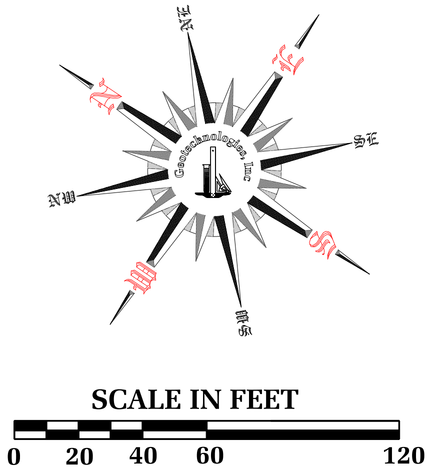
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP.

FILE NO. 21671





LEGEND

B1 (17.5)	BORING LOCATION AND NUMBER SHOWING DEPTH OF FILL THIS INVESTIGATION
B4 (15)	LOCATION & NUMBER OF BORING SHOWING DEPTH OF FILL PREVIOUS INVESTIGATION (FILE NO. 21086)
B1 (12)	LOCATION & NUMBER OF BORING SHOWING DEPTH OF FILL CONVERSE (PROJECT NO. 60-032-A)
B1 (8.5)	LOCATION & NUMBER OF BORING SHOWING DEPTH OF FILL SOILS INTERNATIONAL (FILE NO. L-1176-FG)

NOTE: ELEVATIONS SHOWIN NOT TRUE ELEVATIONS ABOVE MEAN SEA LEVEL

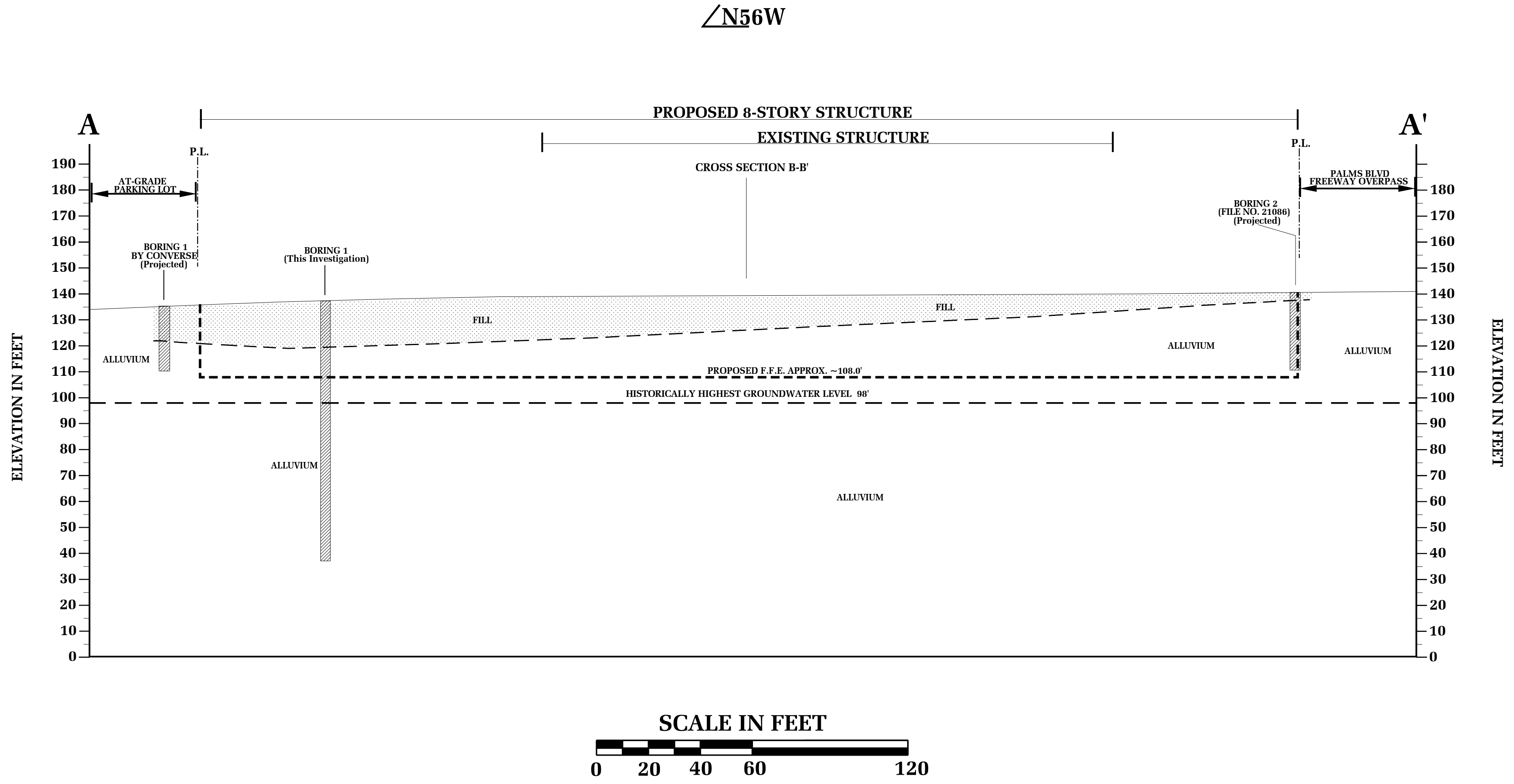
REFERENCE: ALTA/ACSM LAND TITLE & TOPOGRAPHIC SURVEY BY PSOMAS
DATED AUGUST 29, 2005

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP

FILE No. 21671
DRAWN BY: TC
DATE: May 2019
SHEET: 1 of 1



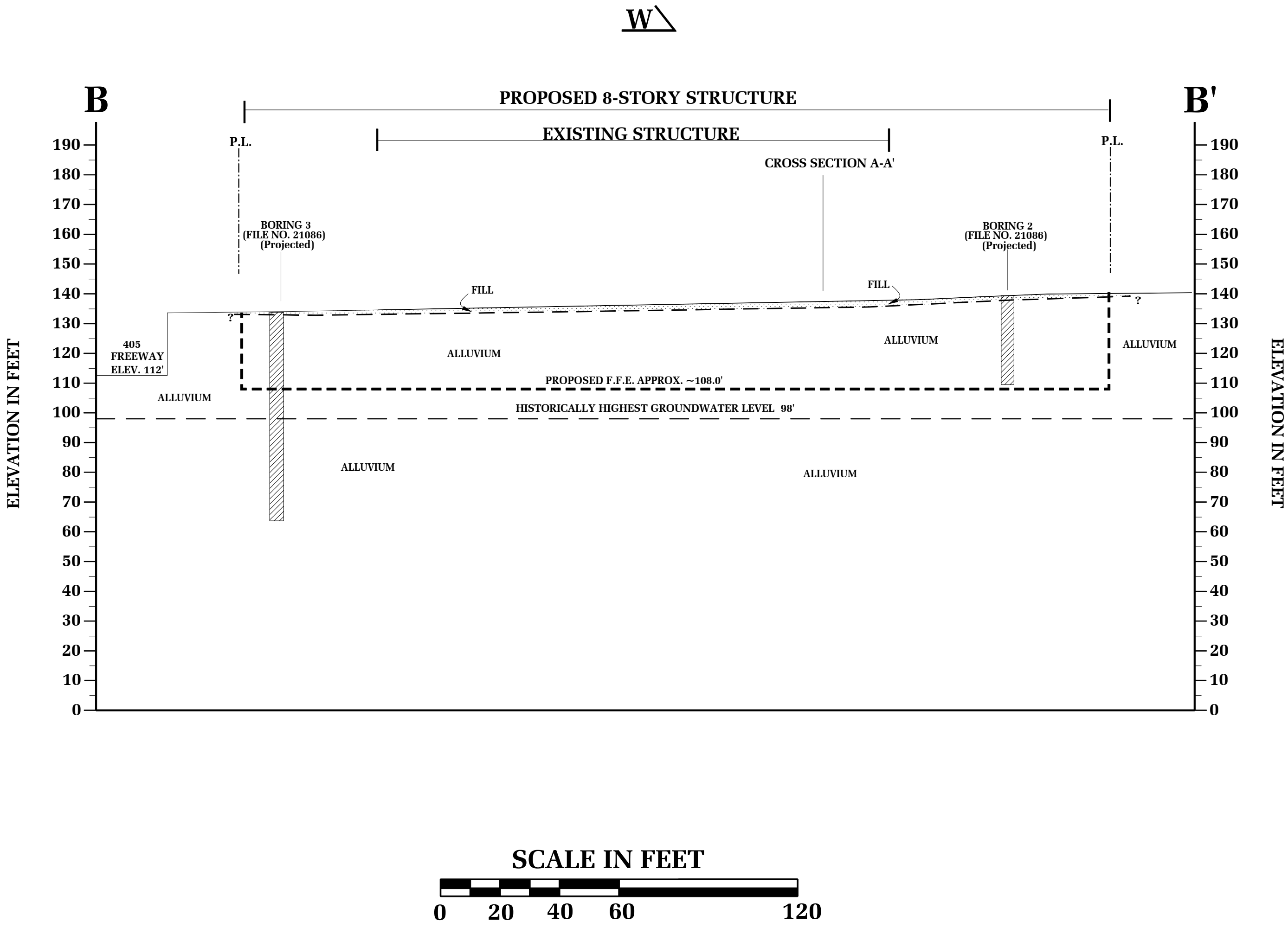
NOTE: ELEVATIONS SHOWN NOT TRUE ELEVATION ABOVE MEAN SEA LEVEL

CROSS SECTION A-A'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP

FILE No. 21671	DRAWN BY: AE
DATE: May 2019	SHEET: 1 of 1



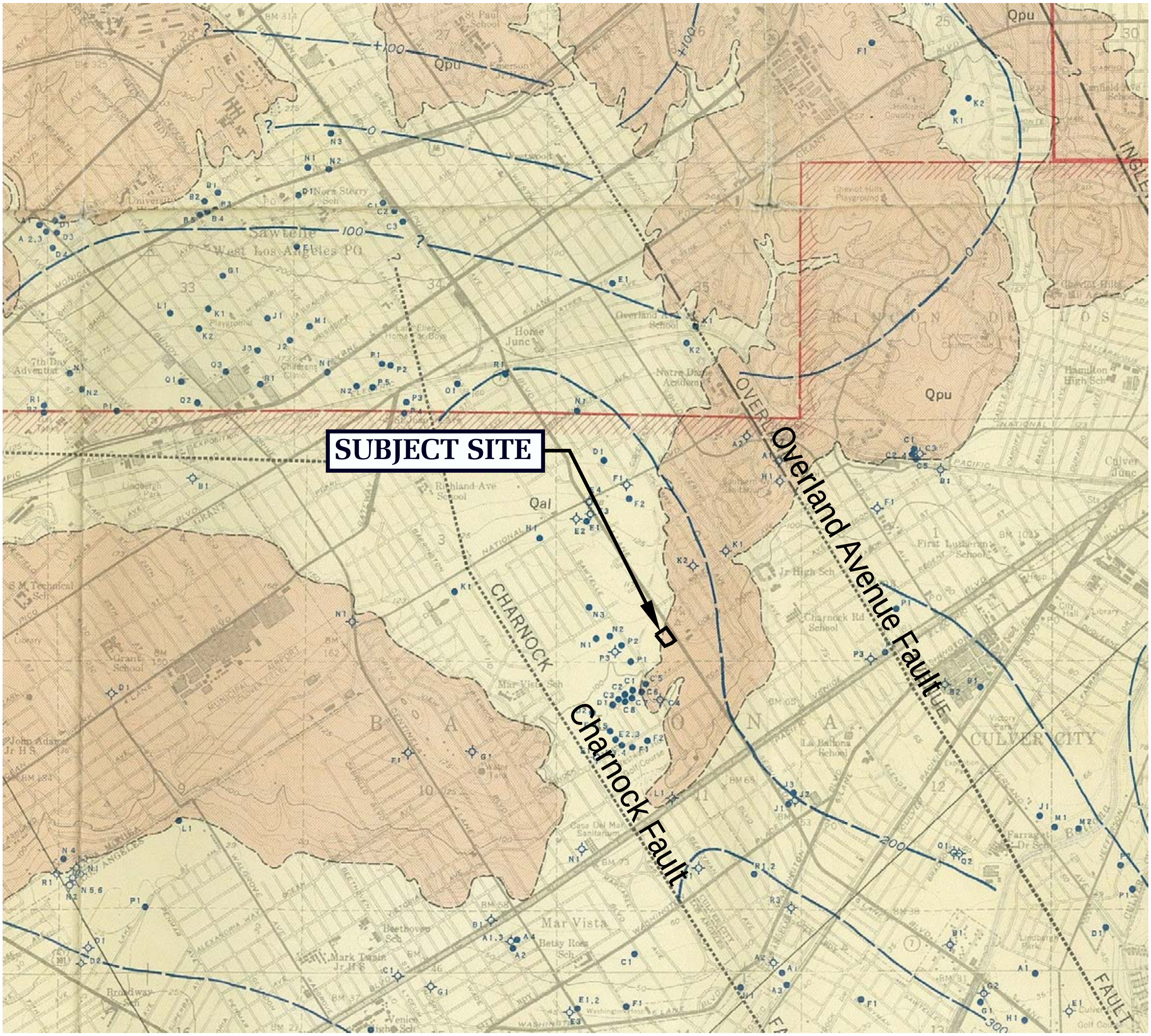
NOTE: ELEVATIONS DO NOT CORRESPOND TO TRUE ELEVATION ABOVE MEAN SEA LEVEL

CROSS SECTION B-B'

BALBOA COVE GROUP, LP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FILE No. 21671	DRAWN BY: AE
DATE: May 2019	SHEET: 1 of 1



SCALE IN MILES

0 2 MI

LEGEND

Qal: Alluvial and coastal deposits - sand, gravel and silt in stream channels and beneath flood plains

Qpu: Terrace Cover - reddish-brown sand, nonfossiliferous and probably largely continental

◄--+--- Folds - arrow on axial trace of fold indicates direction of plunge

—••••? Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

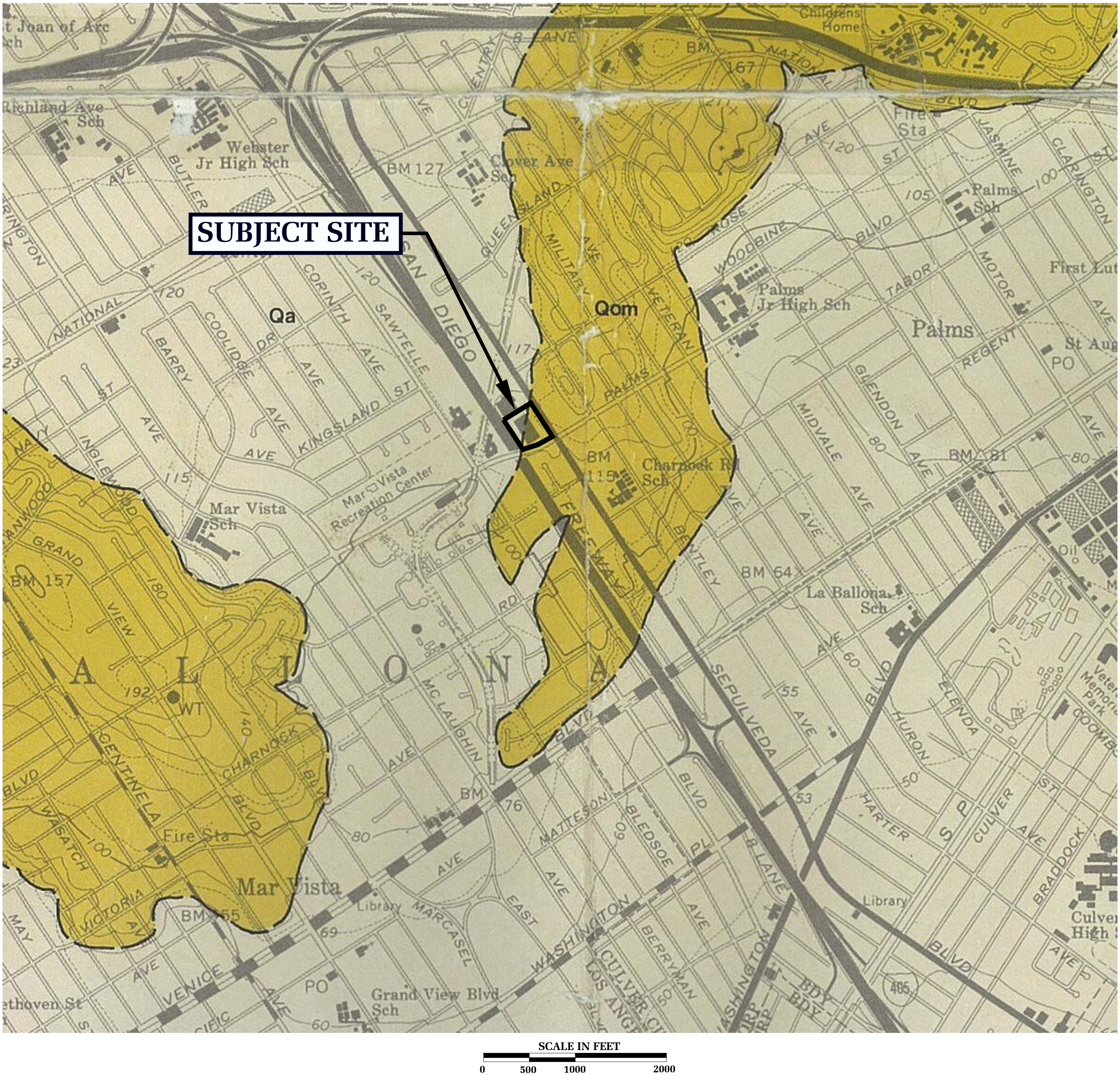
REFERENCE: POLAND, GARRETT, SINNOTT (1959); USGS WSP 1461

LOCAL GEOLOGIC MAP - POLAND

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP.

FILE NO. 21671



LEGEND

Qa: Surficial Sediments - alluvial gravel, sand and silt-clay

Qom Shallow Marine Sediments - marine deposits of Hoots 1931: light gray to light borwn sand, pebbly sand gravel & silt

--·-|--- Folds - arrow on axial trace of fold indicates direction of plunge

—·····? Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

REFERENCE: DIBBLEE, T.W., (1992) GEOLOGIC MAP OF THE CALABASAS QUADRANGLES (#DF-37)

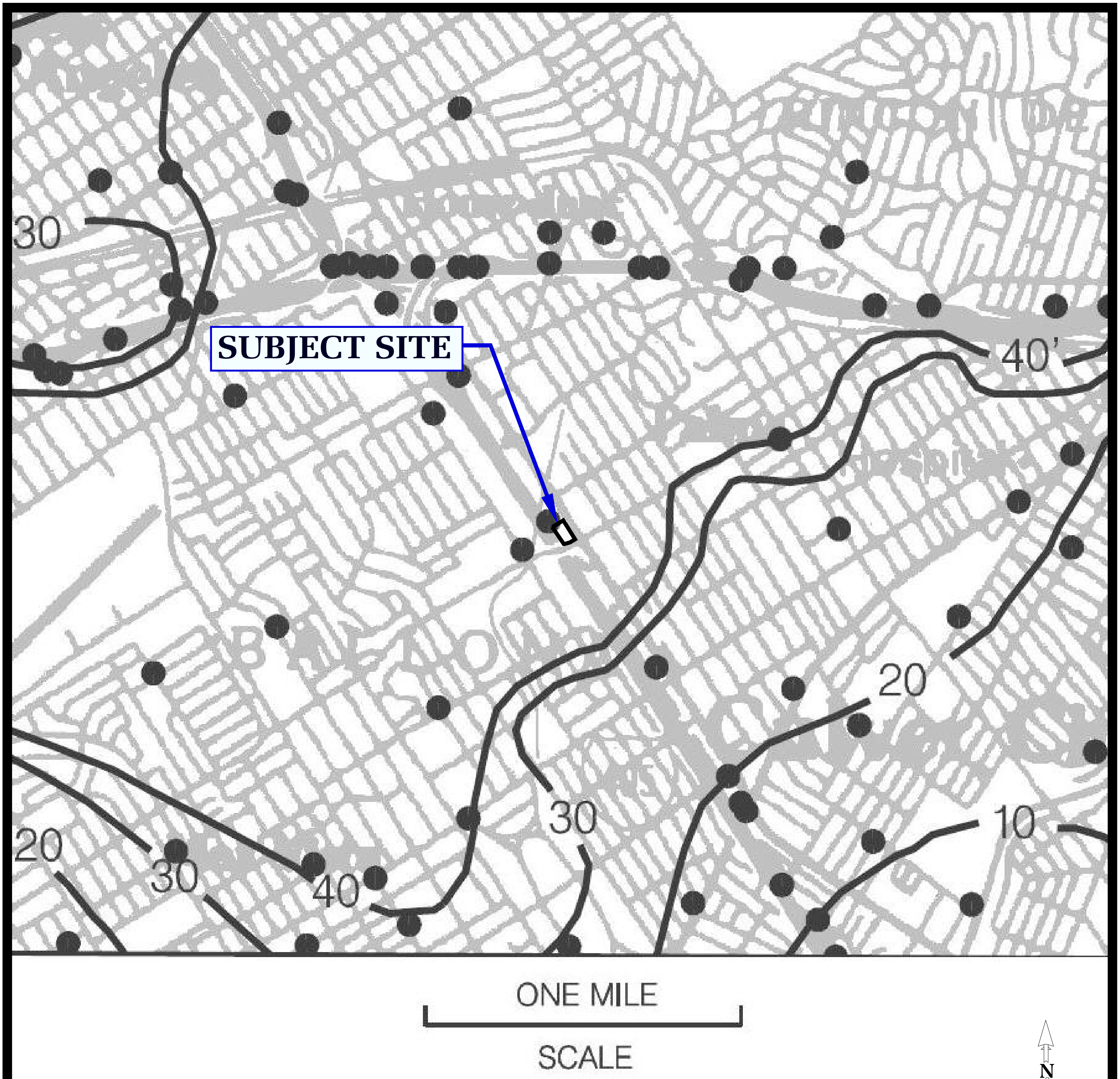
N

LOCAL GEOLOGIC MAP - DIBBLEE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP.

FILE NO. 21671



20 Depth to groundwater in feet

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

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP.

FILE No. 21671

BORING LOG NUMBER 1

Balboa Cove Group **Date: 03/13/19** **Elevation: 136'***

File No. 21671 **Method: 8-inch diameter Hollow Stem Auger**

km

*Ref: Alta/ACSM Land Title & Topographic Survey by PSOMAS, dated 8/29/05

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3-inch Asphalt over 8-inch Base
				-		
				1 --		
				-		FILL: Clayey Silt to Silty Clay, dark brown, moist, stiff
				2 --		
				-		
3	23	13.5	119.0	3 --		
				-		
				4 --		Sandy to Clayey Silt, dark brown, moist, stiff
				-		
5	28	15.5	104.7	5 --		
				-		Sandy Silt to Silty Clay, dark brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	27	16.4	116.9	-		
				8 --		Sandy to Clayey Silt, dark and yellowish brown, moist, stiff
				-		
				9 --		
				-		
10	18	13.3	115.4	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	29	13.0	115.9	-		
				13 --		Sandy Silt, dark brown, moist, stiff
				-		
				14 --		
				-		
15	18	11.6	119.0	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	19	9.0	112.7	-		
				18 --	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				-		
				19 --		
				-		
20	18	15.0	100.7	20 --		
				-	SP	Sand, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 1

Balboa Cove Group

File No. 21671

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	85	9.3	124.4	30 --		
				-	SM/SP	Silty Sand to Sand, dark brown and yellowish brown, very
				31 --		dense, fine grained, gravel to 1/4" (slate)
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	30/6"	15.1	100.6	40 --		
	50/5"			-	SM	Silty Sand, yellowish brown, very dense, fine grained
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	45	19.0	103.2	50 --		
	50/5"			-		dark gray and gray

BORING LOG NUMBER 1

Balboa Cove Group

File No. 21671

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	75	32.3	90.9	55 --		
				-	SP/ML	Sand to Clayey Silt, dense, fine grained, stiff
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	48/6"	9.4	105.5	60 --		
	50/4"			-	SP	Sand, very dense, fine grained
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	45/6"	17.3	113.8	65 --		
	50/3"			-	SM/SP	Silty Sand to Sand, fine grained
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	100/7"	4.3	102.9	70 --		
				-	SP	Sand, yellowish brown, fine grained
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	100/9"	9.3	99.0	75 --		
				-		

BORING LOG NUMBER 1

Balboa Cove Group

File No. 21671

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		NOTE: The stratification lines represent the approximate
				77 --		boundary between earth types; the transition may be gradual.
				-		
				78 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				79 --		Modified California Sampler used unless otherwise noted
				-		
80	76	17.8	109.6	80 --		
				-	ML	Clayey Silt, dark gray, very stiff
				81 --		
				-		
				82 --		
				-		
				83 --		
				-		
				84 --		
				-		
85	40/6"	23.8	104.2	85 --		
	50/5"			-		
				86 --		
				-		
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
90	48/6"	14.8	119.3	90 --		
	50/3"			-	SM/ML	Silty Sand to Sandy Silt, dark brown and yellowish brown,
				91 --		very dense, fine grained, very stiff
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
95	15/6"	12.2	115.9	95 --		
	50/4"			-	SM	Silty Sand, dark gray and gray, very dense, fine grained,
				96 --		minor gravel
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
100	40/6"	23.6	102.1	100 --		
	50/4"			-		Total Depth 100 feet
						No Water
						Fill to 17½ feet

BORING LOG NUMBER 1

Reliable Properties **Date: 10/28/15** **Elevation: 137.5'***

File No. 21086 **Method: 8-inch diameter Hollow Stem Auger**

km

***Reference: ALTA Plan by PSOMAS, dated 8/29/05**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		5-inch Asphalt over 5-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt to Silty Clay, dark brown, moist, stiff, minor slate fragments
				2 --		
2.5	37	13.5	110.8	-		
				3 --		
				-		
				4 --		
				-		
5	20	16.2	109.6	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	32	15.0	118.7	-		Sandy Silt to Silty Clay, dark and grayish brown, moist, stiff, minor gravel
				8 --		
				-		
				9 --		
				-		
10	33	14.6	118.8	10 --		
				-		Sandy to Clayey Silt to Silty Sand, dark and grayish brown, moist, stiff, medium dense, fine grained
				11 --		
				-		
				12 --		
12.5	34	14.7	110.3	-		
				13 --		Clayey Silt to Silty Clay, dark brown, moist, stiff
				-		
				14 --		
				-		
15	28	12.9	115.3	15 --		
				-	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
17.5	67	9.1	128.9	-		
				18 --		Silty Sand, dark brown, moist, very dense, fine grained
				-		
				19 --		
				-		
20	76	10.5	129.5	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	38/6"	12.9	122.5	25 --		
	50/5"			-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense, fine grained

BORING LOG NUMBER 1

Reliable Properties

File No. 21086

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	92	11.1	Disturbed	30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	100/7"	18.3	104.5	35 --		
				-	SM	Silty Sand, dark and yellowish brown, moist, very dense, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	47/6"	9.4	94.6	40 --		
	50/5"			-	SP	Sand, yellow and grayish brown, moist, very dense, fine grained
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	100/7"	23.9	98.9	45 --		
				-		Sand, yellow and grayish brown, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-	SM/SP	Silty Sand to Sand, gray to dark gray, moist, very dense, fine grained
50	47/6"	22.6	98.9	50 --		
	50/6"			-		Total Depth 50 feet No Water Fill to 15 feet

BORING LOG NUMBER 2

Reliable Properties **Date: 10/28/15** **Elevation: 141'***

File No. 21086 **Method: 8-inch diameter Hollow Stem Auger**

km

*Reference: ALTA Plan by PSOMAS, dated 8/29/05

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		4-inch Asphalt over 8-inch Base
				-		
				1 --		
				-		FILL: Sandy to Clayey Silt, dark brown, moist, stiff
				2 --		
2.5	23	15.8	111.7	-		
				3 --		
				-	ML	ALLUVIUM: Sandy to Clayey Silt, dark brown, moist, stiff, minor slate fragments
				4 --		
				-		
5	22	19.5	104.8	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	20/6"	7.1	120.8	-		
	50/5"			8 --	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, very dense, fine grained
				-		
				9 --		
				-		
10	74	5.7	116.5	10 --		
				-	SM	Silty Sand, dark and grayish brown, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	65	14.3	119.0	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark and grayish brown, moist, dense, fine grained, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	52	13.3	118.8	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	70	21.7	112.0	25 --		
				-	SM	Silty Sand, dark and grayish brown mottling, moist, very dense, fine grained

BORING LOG NUMBER 2

Reliable Properties

File No. 21086

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	28/6"	10.5	125.4	30 --		
	50/5"			-		Total Depth 30 feet
				31 --		No Water
				-		Fill to 3 feet
				32 --		
				-		
				33 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				34 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				35 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 3

Reliable Properties **Date: 10/28/15** **Elevation: 136'***

File No. 21086 **Method: 8-inch diameter Hollow Stem Auger**

km ***Reference: ALTA Plan by PSOMAS, dated 8/29/05**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
						Surface Conditions: Asphalt Parking Lot
				0 --		4-inch Asphalt over 2-inch Base
				-		
				1 --		FILL: Sandy to Clayey Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	14	17.4	107.4	-		
				3 --		
				-	ML/SM	ALLUVIUM: Clayey Silt to Silty Sand, dark brown, moist, stiff, medium dense, fine grained
				4 --		
				-		
5	7	18.4	SPT	5 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	15	16.7	110.1	-		
				8 --		
				-		
				9 --		
				-		
10	22	19.7	SPT	10 --		
				-		Sandy Silt to Silty Clay, dark and grayish brown, moist, stiff
				11 --		
				-		
				12 --		
12.5	42	20.5	108.7	-		
				13 --		
				-		
				14 --		
				-		
15	74	14.7	SPT	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, yellowish brown mottling, moist, very dense, fine grained, very stiff
				16 --		
				-		
				17 --		
17.5	43	14.6	115.7	-		
				18 --		
				-		
				19 --		
				-		
20	36	10.1	SPT	20 --		
				-	SM	Silty Sand, dark and grayish brown, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
22.5	30/6" 50/4"	7.6	98.2	-		
				23 --	SP	Sand, yellow and grayish brown, moist, very dense, fine grained
				-		
				24 --		
				-		
25	38	6.3	SPT	25 --		
				-		

BORING LOG NUMBER 3

Reliable Properties

File No. 21086

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	40/6"	17.7	108.1	-		
	50/3"			28 --	SM	Silty Sand, dark and grayish brown, moist, very dense, fine grained
				-		
				29 --		
				-		
30	26	13.1	SPT	30 --		
				-	SM/SP	Silty Sand to Sand, dark and grayish brown, moist, medium dense, fine grained
				31 --		
				-		
				32 --		
32.5	38/6"	13.3	121.8	-		
	50/5"			33 --	SM/ML	Silty Sand to Sandy Silt, dark and grayish brown, moist, very dense, fine grained, very stiff
				-		
				34 --		
				-		
35	35	12.0	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	100/9"	18.8	117.6	-		
				38 --	SM	Silty Sand, dark and grayish brown, moist, very dense, fine grained
				-		
				39 --		
				-		
40	48	14.1	SPT	40 --		
				-	ML	Sandy to Clayey Silt, dark and grayish brown, moist, stiff
				41 --		
				-		
				42 --		
42.5	100/9"	6.2	110.2	-		
				43 --	SP	Sand, yellowish brown, moist, very dense, fine to medium grained
				-		
				44 --		
				-		
45	37	5.3	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	37/6"	3.7	108.9	-		
	50/5"			48 --		
				-		
				49 --		
				-		
50	60	14.5	SPT	50 --		
				-		

BORING LOG NUMBER 3

Reliable Properties

File No. 21086
km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	45/6"	27.7	98.4	-		
	50/3"			53 --	SM/ML	Silty Sand to Sandy Silt, dark and grayish brown, moist, very dense, fine grained, very stiff
				-		
				54 --		
				-		
55	19	28.9	SPT	55 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark and grayish brown, moist, stiff
				56 --		
				-		
				57 --		
57.5	32/6"	27.4	96.0	-		
	50/4"			58 --	ML	Sandy to Clayey Silt, dark and grayish brown, moist, very stiff
				-		
				59 --		
				-		
60	40	30.9	SPT	60 --		
				-	SP/ML	Sand to Sandy Silt, gray and dark brown, moist, medium dense, fine grained, stiff
				61 --		
				-		
				62 --		
62.5	30/6"	26.8	100.2	-		
	50/4"			63 --	SW/SP	Silty Sand to Sand, dark and grayish brown, moist, very dense, fine grained
				-		
				64 --		
				-		
65	42	21.1	SPT	65 --		
				-	SM/CL	Silty to Clayey Sand, dark and grayish brown, moist, medium dense to dense, fine grained
				66 --		
				-		
				67 --		
67.5	100/8"	4.4	117.2	-		
				68 --	SM/SP	Silty Sand to Sand, dark and grayish brown, moist, very dense, fine grained
				-		
				69 --		
				-	SP	Sand, yellow and grayish brown, moist, very dense, fine grained
70	28/6"	4.5	SPT	70 --		
	50/5"			-		Total Depth 70 feet
				71 --		No Water
				-		Fill to 3 feet
				72 --		
				-		
				73 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				74 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				75 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
						SPT=Standard Penetration Test

BORING LOG NUMBER 4

Reliable Properties **Date: 10/28/15** **Elevation: 130'***

File No. 21086 **Method: 8-inch diameter Hollow Stem Auger**

km

*Reference: ALTA Plan by PSOMAS, dated 8/29/05

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		5-inch Asphalt over 6-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt, dark brown, moist, stiff
				2 --		
2.5	27	15.6	116.9	-		
				3 --		Sandy Silt to Silty Sand, dark and grayish brown, moist, stiff, medium dense, fine grained
				-		
				4 --		
				-		
5	15	15.4	110.3	5 --		
				-		Sandy to Clayey Silt, dark and grayish brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	36	14.5	115.9	-		
				8 --		Silty Sand to Sandy Silt, dark brown to dark gray, moist, stiff, medium dense, fine grained
				-		
				9 --		
				-		
10	19	11.3	108.2	10 --		
				-		Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
12.5	13	10.3	110.0	-		
				13 --	SM	ALLUVIUM: Silty Sand, dark and yellowish brown, moist, medium dense, fine grained
				-		
				14 --		
				-		
15	25	16.4	113.5	15 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	44	20.1	105.6	20 --		
				-	CL	Silty Clay, dark brown, moist, stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	27	13.1	118.0	25 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff

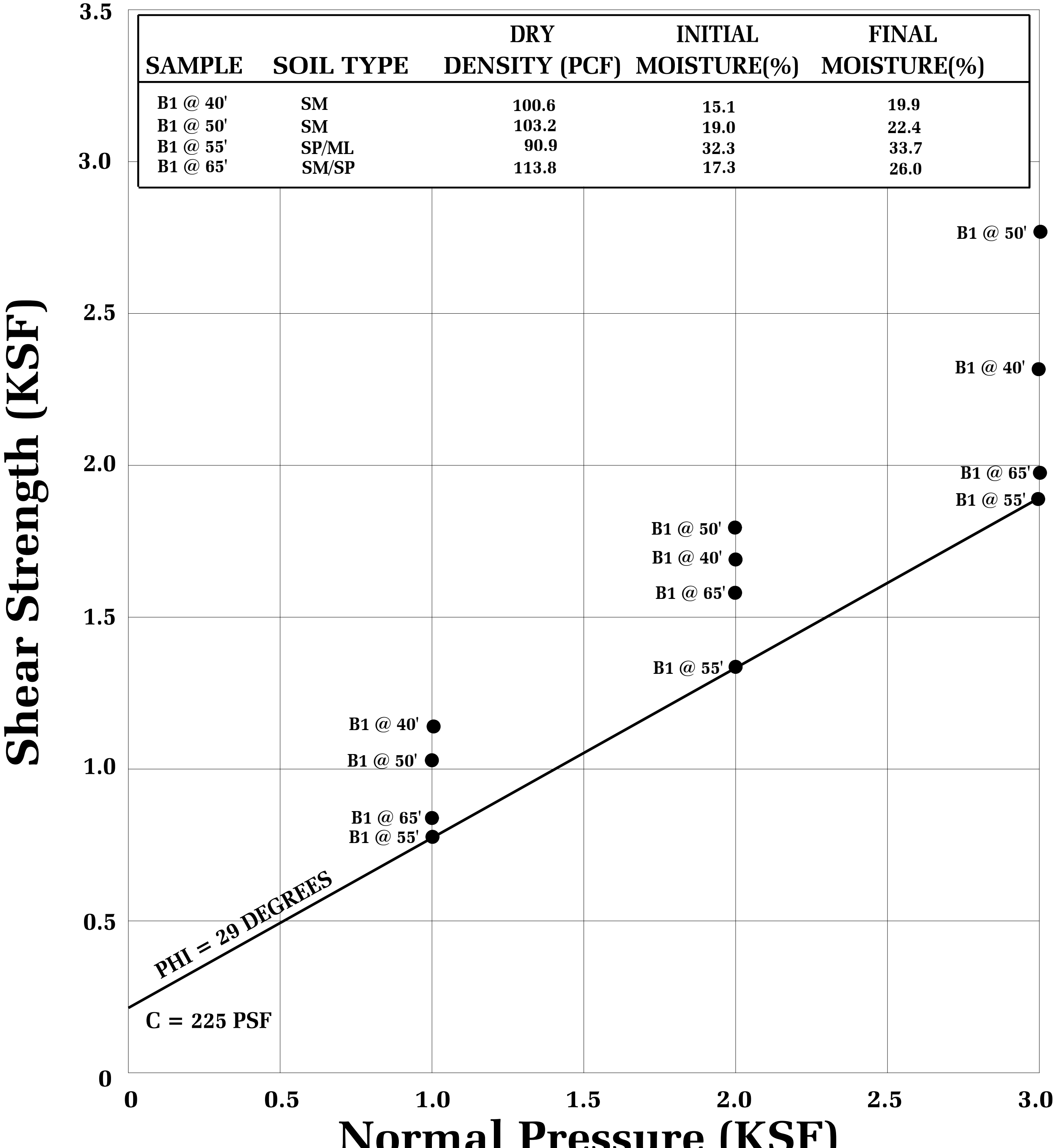
BORING LOG NUMBER 4

Reliable Properties

File No. 21086

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --	SW	Gravelly Sand, dark and grayish brown, moist, dense, fine to coarse grained
				-		
30	96	6.3	130.6	30 --		
				-		Total Depth 30 feet
				31 --		No Water
				-		Fill to 12½ feet
				32 --		
				-		
				33 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				34 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				35 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 40'	SM	100.6	15.1	19.9
B1 @ 50'	SM	103.2	19.0	22.4
B1 @ 55'	SP/ML	90.9	32.3	33.7
B1 @ 65'	SM/SP	113.8	17.3	26.0

● Direct Shear, Saturated

SHEAR TEST DIAGRAM

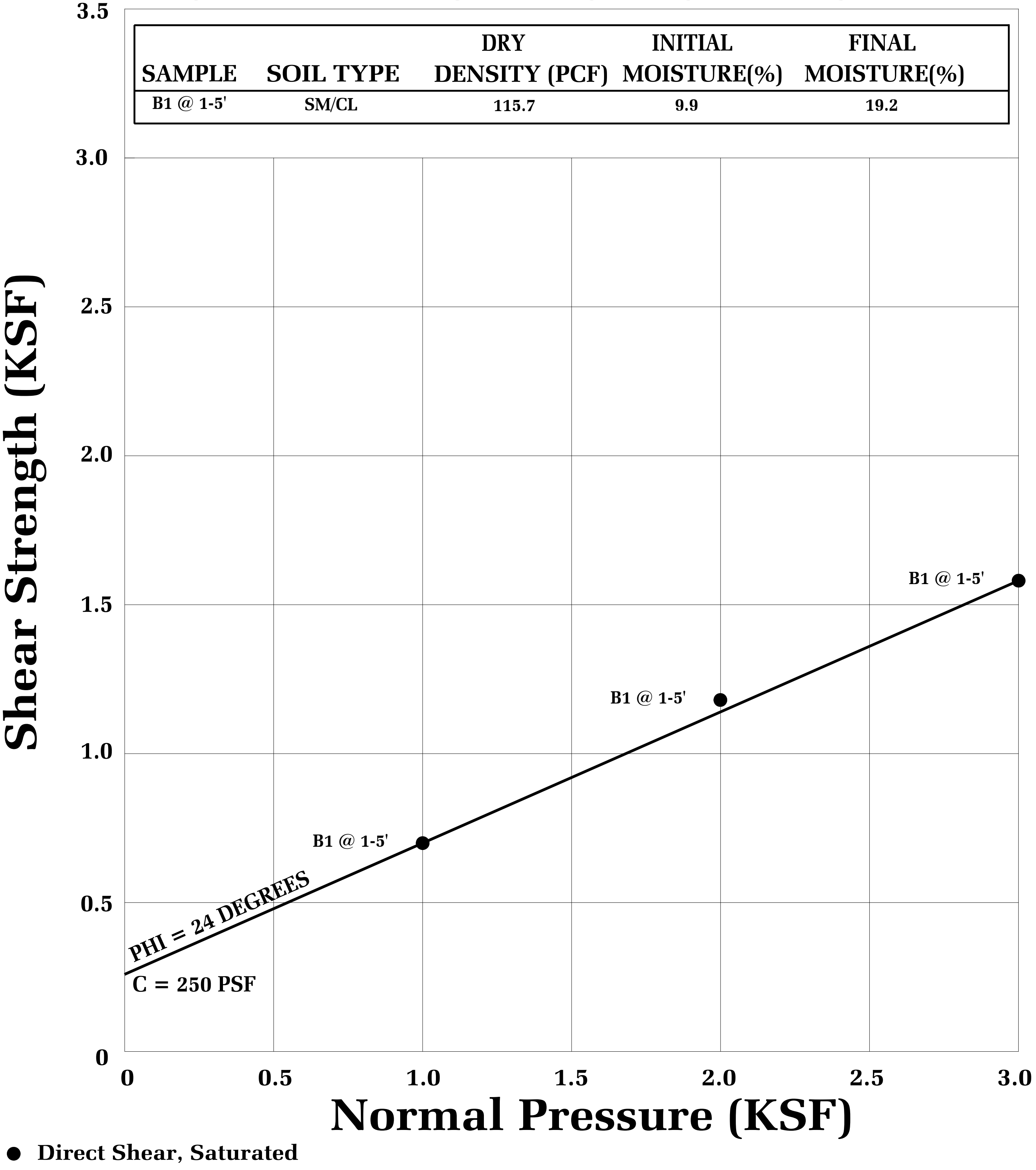
Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21671

PLATE: B-1

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	SM/CL	115.7	9.9	19.2

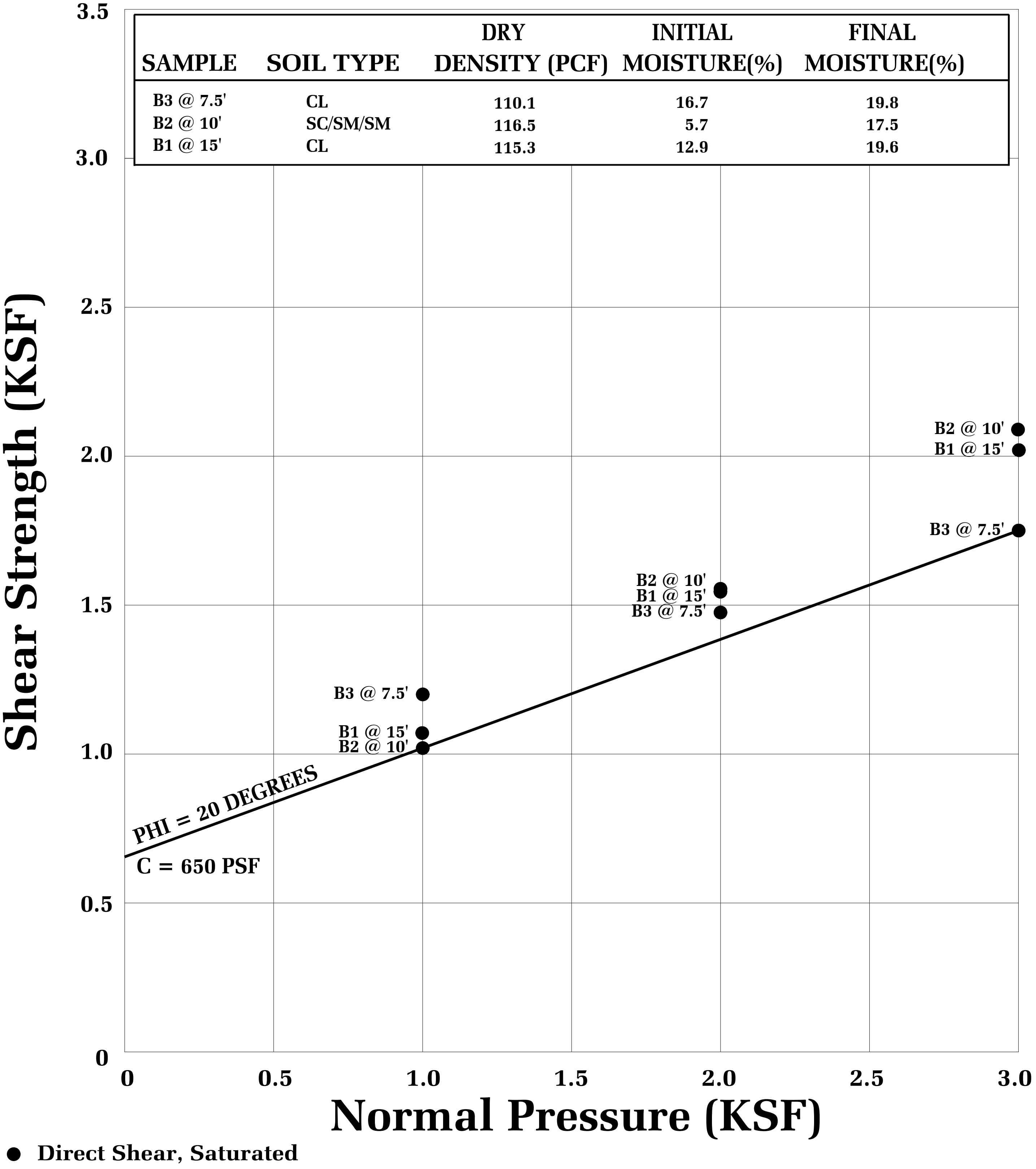
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086

PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B3 @ 7.5'	CL	110.1	16.7	19.8
B2 @ 10'	SC/SM/SM	116.5	5.7	17.5
B1 @ 15'	CL	115.3	12.9	19.6

SHEAR TEST DIAGRAM

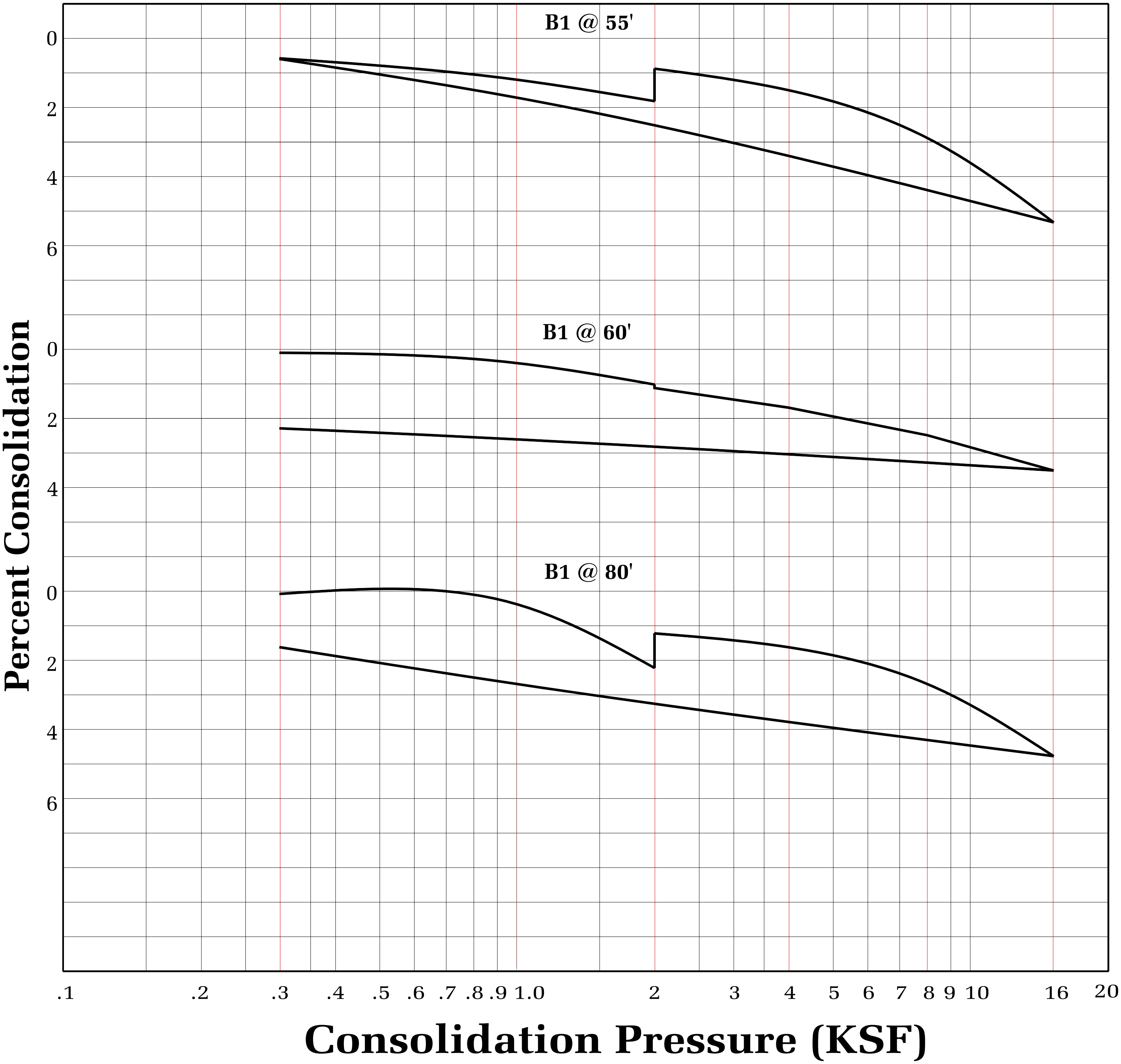
Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086

PLATE: B-2

WATER ADDED AT 2 KSF

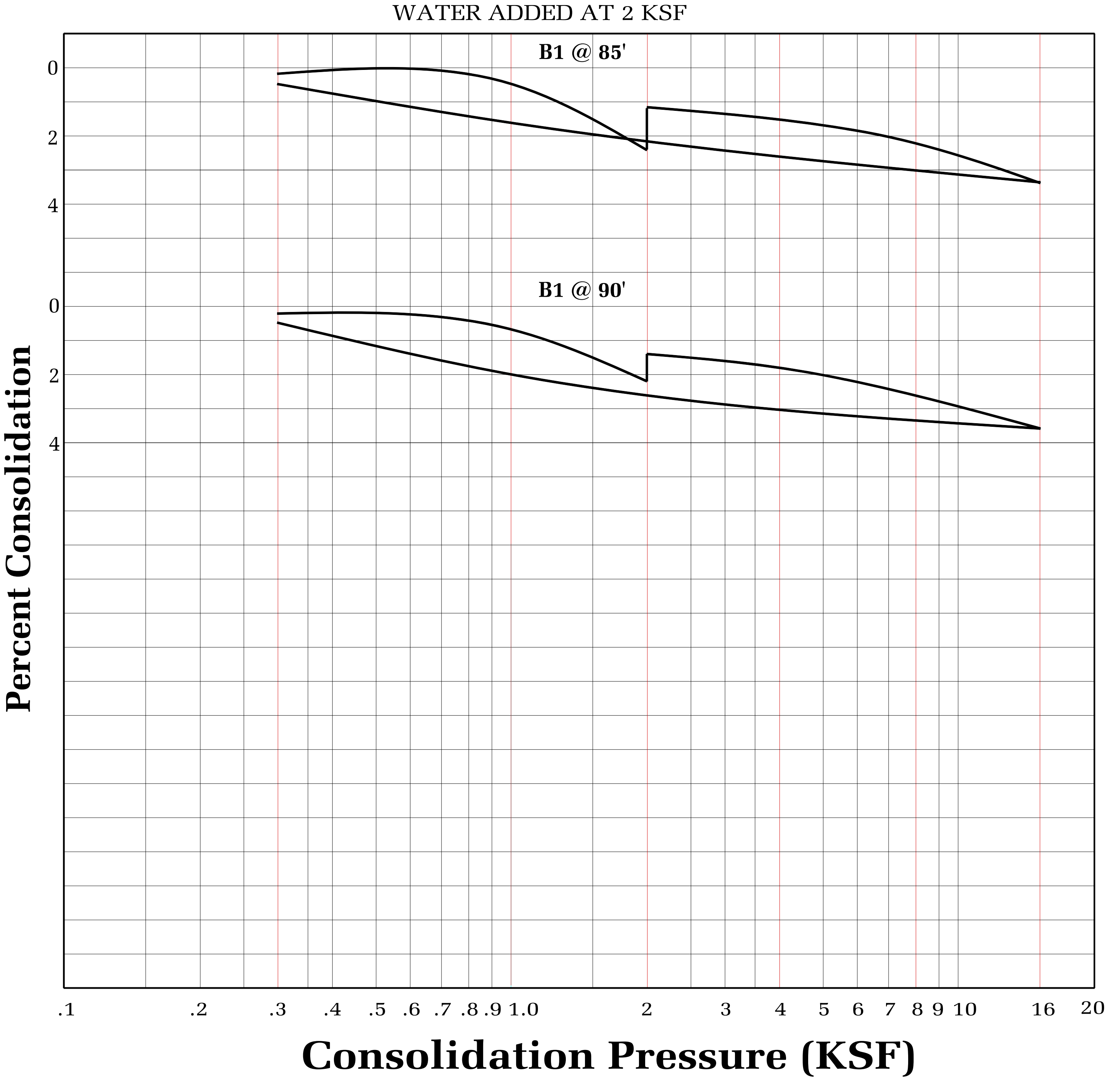


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP

FILE NO. 21671	**PLATE: C-1**



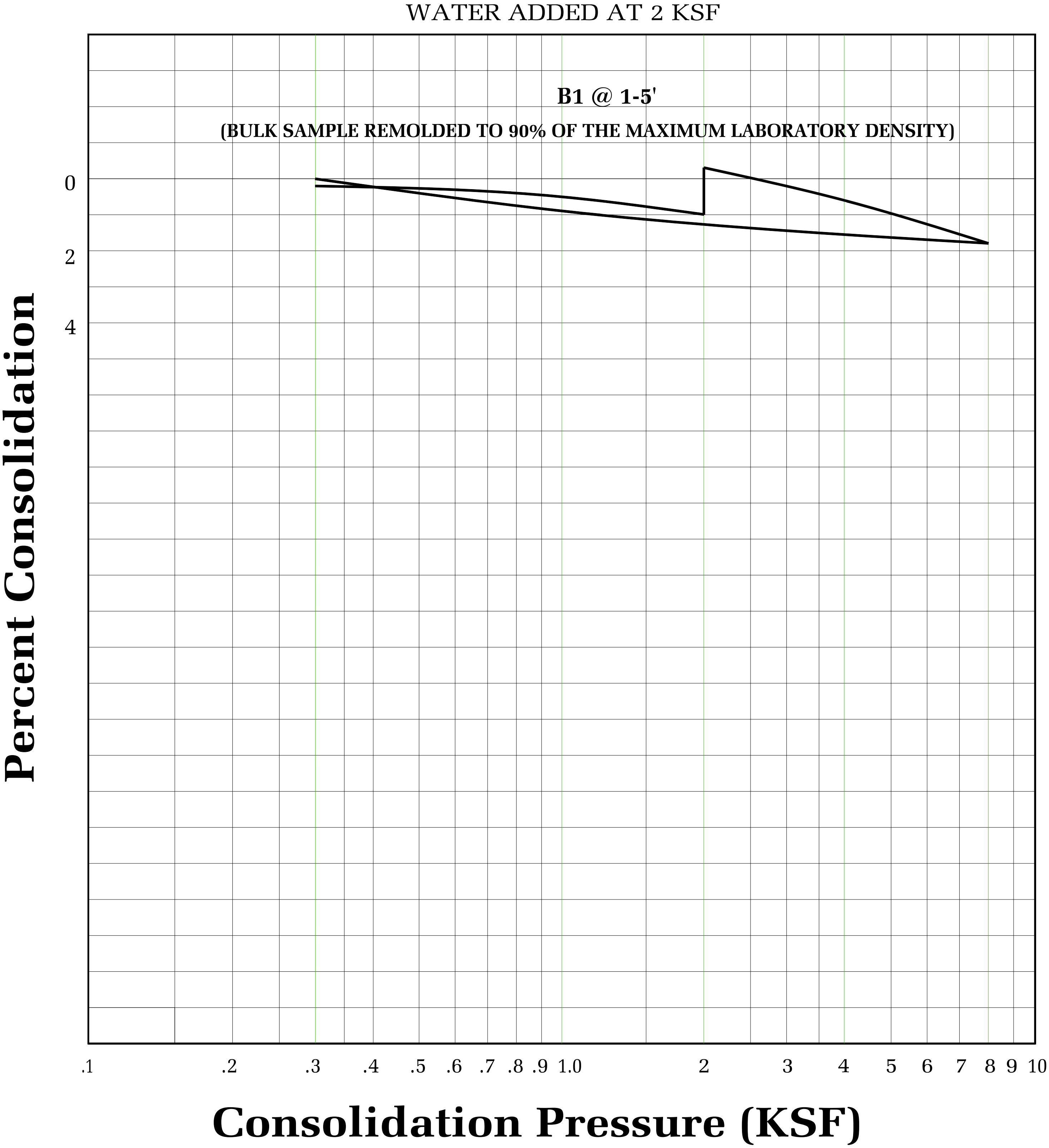
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BALBOA COVE GROUP, LP

FILE NO. 21671

PLATE: C-2



CONSOLIDATION TEST

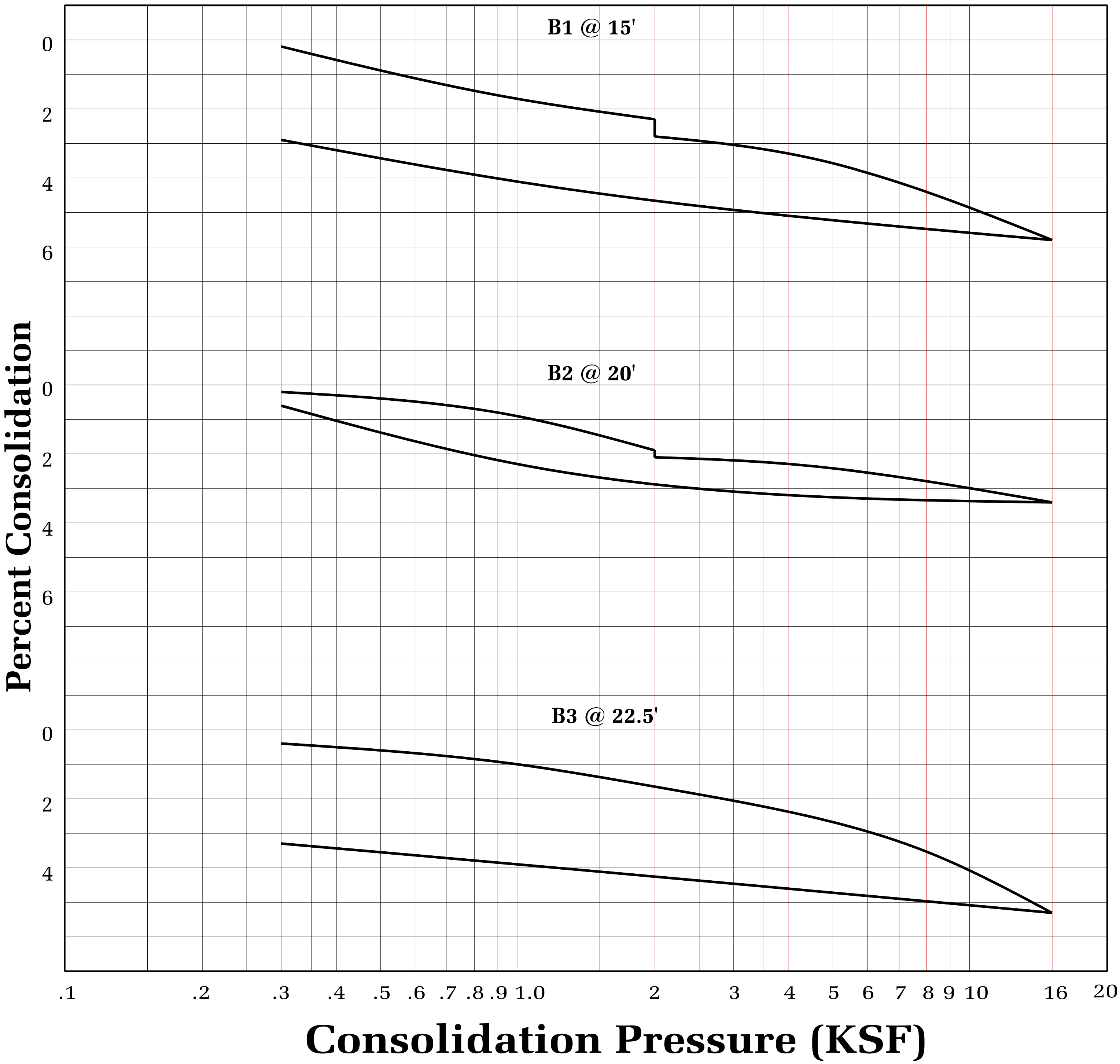
Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086

PLATE: C-1

WATER ADDED AT 2 KSF



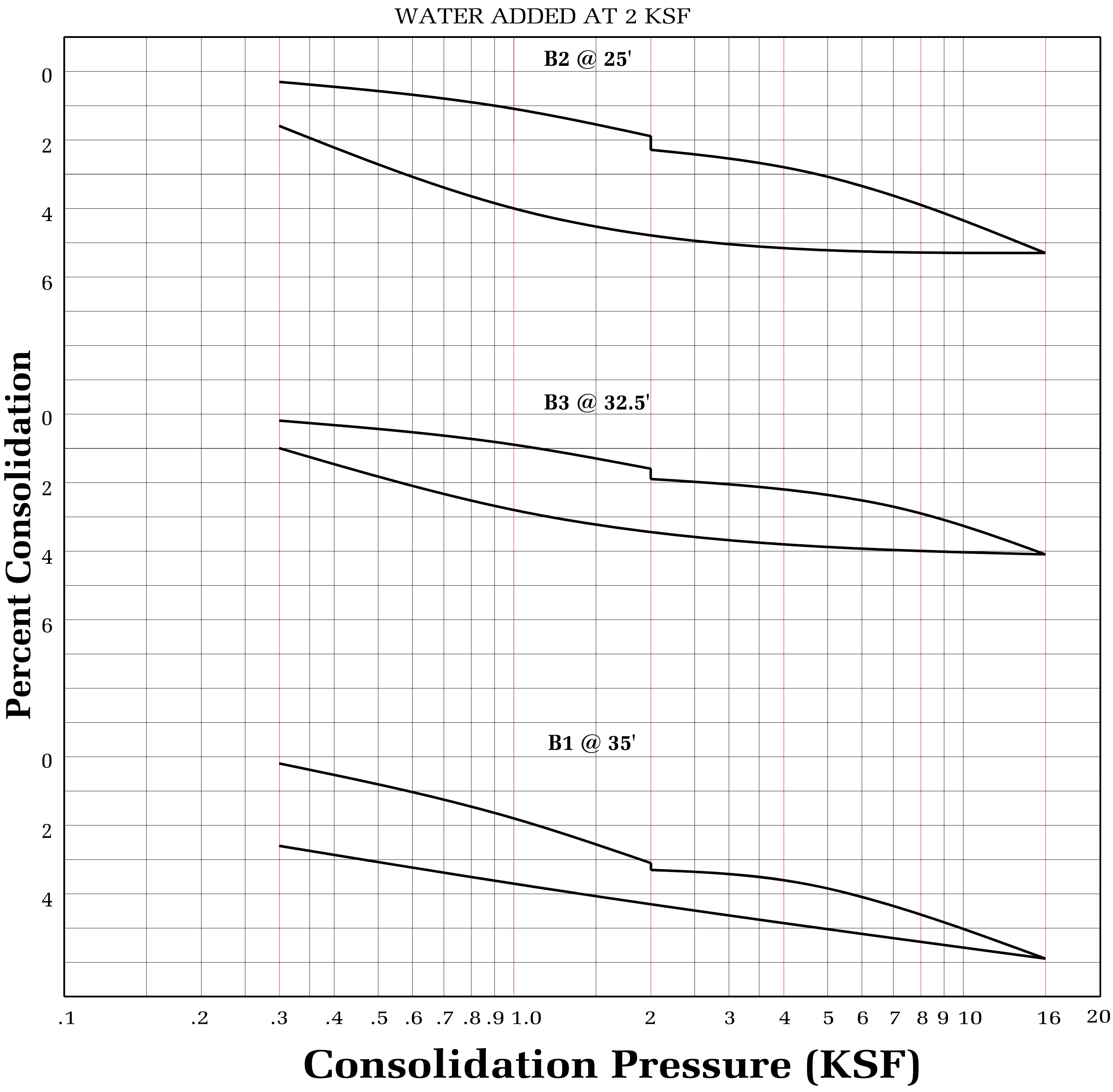
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086

PLATE: C-2



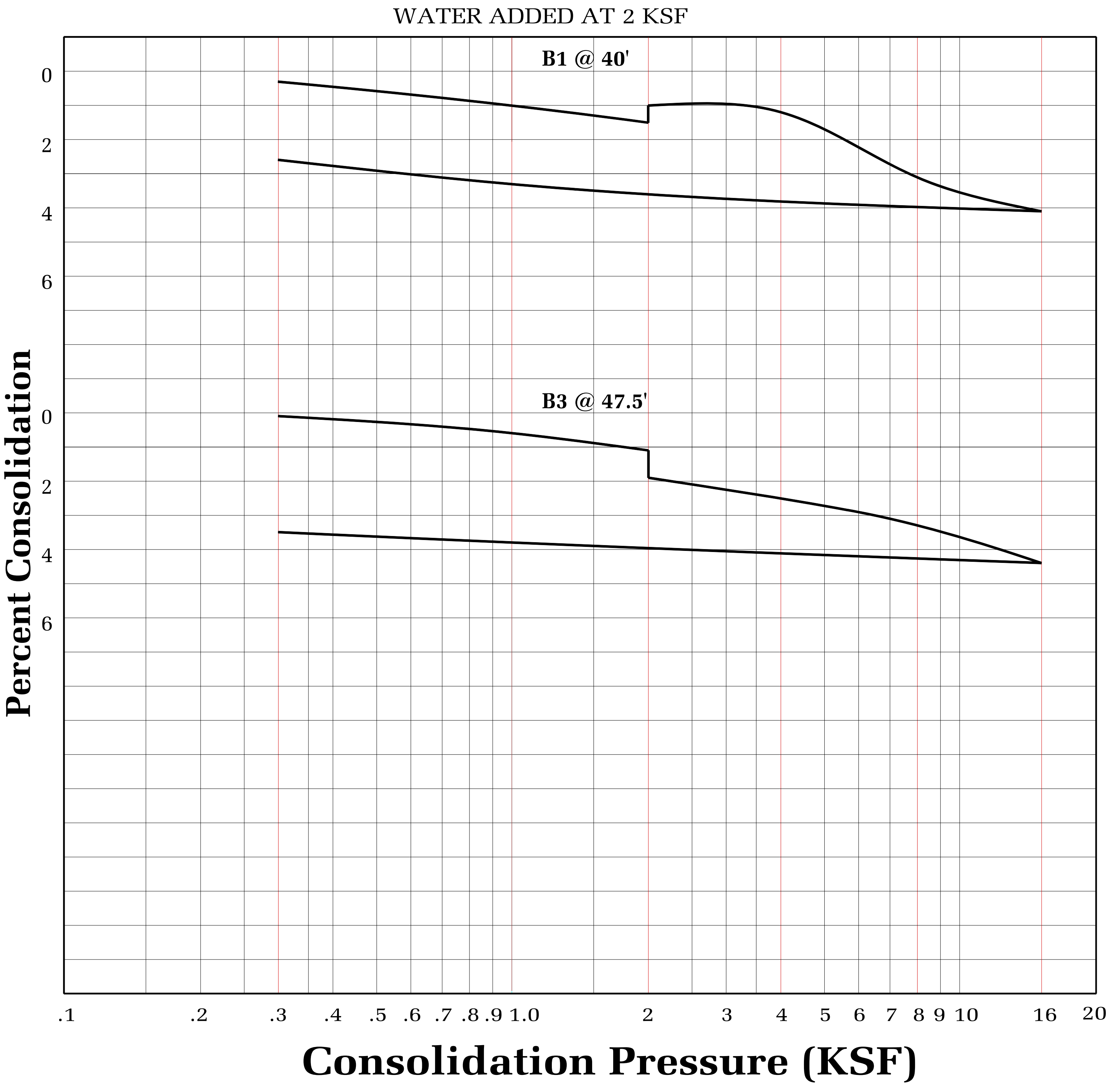
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086

PLATE: C-3



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086 **PLATE: C-4**

ASTM D-1557

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/CL
MAXIMUM DENSITY pcf.	128.5
OPTIMUM MOISTURE %	9.9

ASTM D 4829

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/CL
EXPANSION INDEX UBC STANDARD 18-2	86
EXPANSION CHARACTER	MODERATE

SULFATE CONTENT

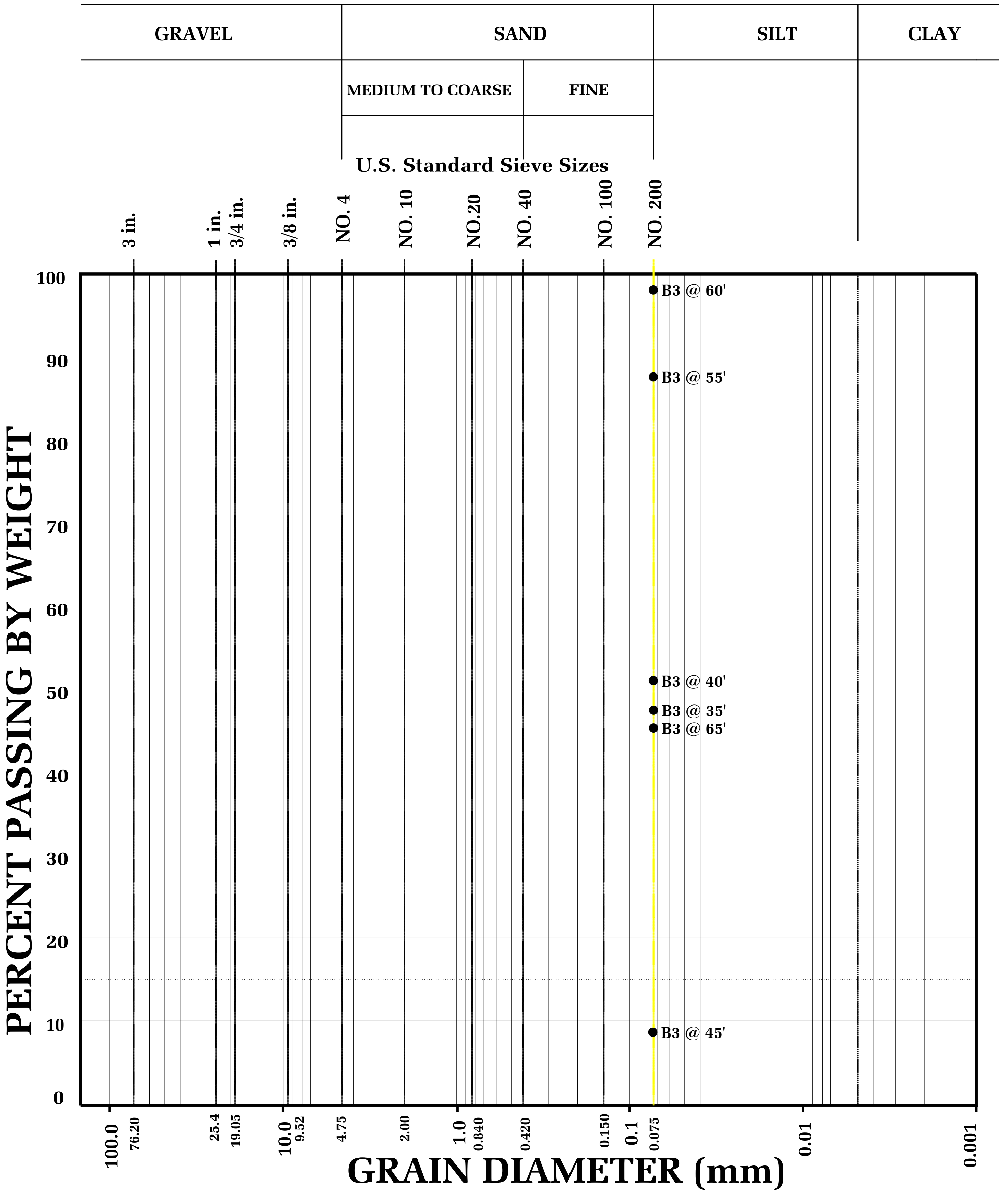
SAMPLE	B1 @ 1-5'	B1 @ 15'	B2 @ 10'	B3 @ 7.5'	B3 @ 22.5'
SULFATE CONTENT: (percentage by weight)	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086 **PLATE: D**



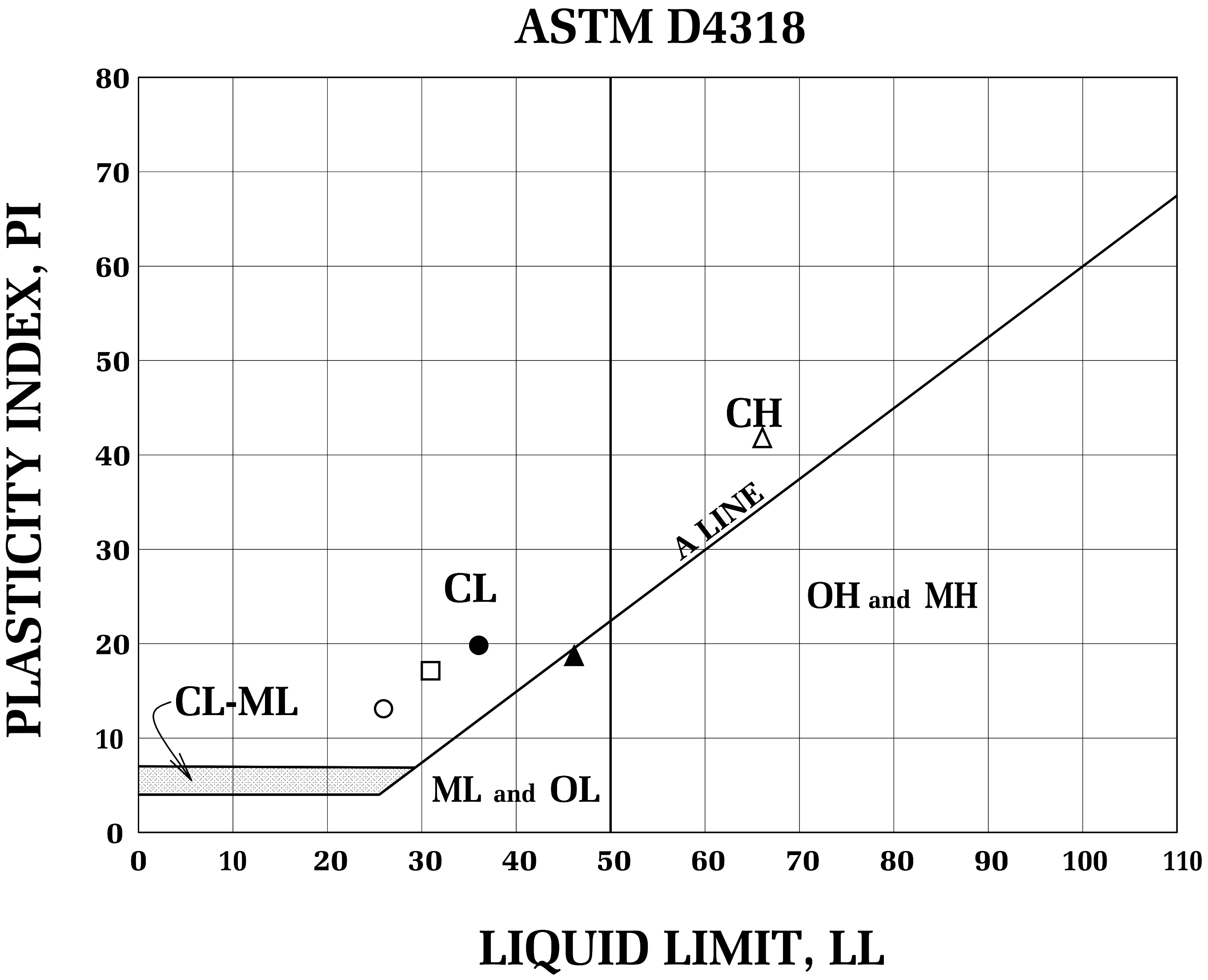
GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086

PLATE: E



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B3	35	○	26	13	13	CL
B3	40	●	36	16	20	CL
B3	55	△	66	24	42	CH
B3	60	▲	46	27	19	CL
B3	65	□	31	14	17	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RELIABLE PROPERTIES

FILE NO. 21086

PLATE: F

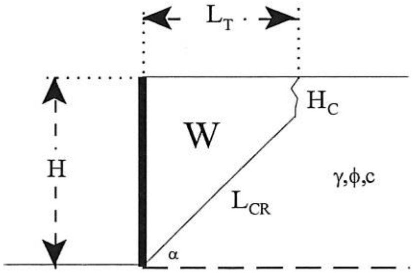
Geotechnologies, Inc.

Project: Balboa cove Group
File No.: 21671
Description: alluvium

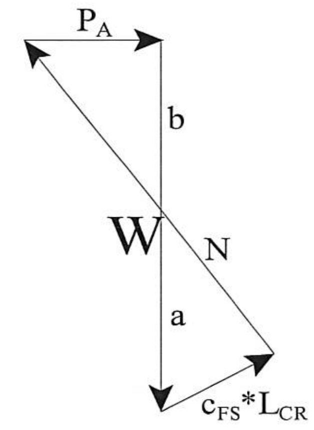
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	20.3 degrees
	(c_{FS})	150.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.5	226	27130.6	24.1	10032.0	17098.6	6128.5
41	4.4	219	26277.8	23.8	9462.0	16815.8	6360.5
42	4.3	212	25443.3	23.5	8941.9	16501.5	6573.0
43	4.2	205	24628.1	23.2	8466.2	16161.9	6766.9
44	4.1	199	23832.5	23.0	8030.3	15802.2	6942.9
45	4.0	192	23056.6	22.7	7630.0	15426.6	7101.6
46	3.9	186	22300.0	22.4	7261.5	15038.5	7243.6
47	3.8	180	21562.3	22.1	6921.6	14640.7	7369.5
48	3.8	174	20843.0	21.8	6607.5	14235.5	7479.7
49	3.7	168	20141.4	21.6	6316.6	13824.7	7574.7
50	3.7	162	19456.8	21.3	6046.7	13410.1	7654.8
51	3.6	157	18788.5	21.0	5795.8	12992.7	7720.3
52	3.6	151	18135.8	20.8	5562.0	12573.7	7771.4
53	3.6	146	17497.9	20.5	5344.0	12153.9	7808.3
54	3.6	141	16874.1	20.3	5140.1	11734.0	7831.2
55	3.6	136	16263.8	20.0	4949.2	11314.6	7840.1
56	3.6	131	15666.2	19.8	4770.0	10896.2	7835.1
57	3.6	126	15080.6	19.6	4601.6	10479.0	7816.1
58	3.6	121	14506.5	19.3	4443.0	10063.4	7783.2
59	3.6	116	13943.1	19.1	4293.4	9649.7	7736.1
60	3.7	112	13389.9	18.9	4151.9	9238.0	7674.7
61	3.7	107	12846.3	18.6	4017.8	8828.5	7598.7
62	3.8	103	12311.7	18.4	3890.5	8421.2	7508.0
63	3.8	98	11785.5	18.2	3769.3	8016.3	7402.1
64	3.9	94	11267.3	17.9	3653.5	7613.7	7280.6
65	3.9	90	10756.4	17.7	3542.8	7213.6	7143.2



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 7840.1 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 39.2 pcf

Design Wall for an Equivalent Fluid Pressure: 39 pcf Cantelevered, Triangular Distrib

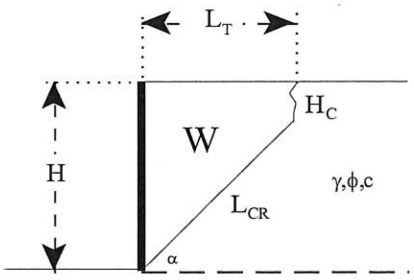
Geotechnologies, Inc.

Project: Balboa cove Group
File No.: 21671
Description: alluvium

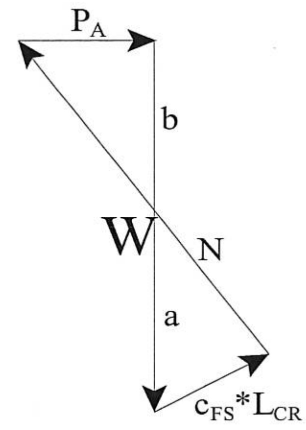
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	30.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	20.3 degrees
	(c_{FS})	150.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.5	524	62883.2	39.6	16519.6	46363.6	16617.7
41	4.4	507	60788.9	39.0	15524.0	45264.8	17121.1
42	4.3	490	58761.7	38.5	14624.1	44137.6	17581.2
43	4.2	473	56799.2	37.9	13808.1	42991.1	18000.1
44	4.1	457	54898.4	37.4	13065.7	41832.8	18379.6
45	4.0	442	53056.6	36.8	12388.4	40668.2	18721.4
46	3.9	427	51270.6	36.3	11768.8	39501.8	19026.9
47	3.8	413	49537.7	35.8	11200.5	38337.2	19297.3
48	3.8	399	47855.1	35.3	10678.0	37177.1	19533.9
49	3.7	385	46220.0	34.8	10196.4	36023.5	19737.6
50	3.7	372	44629.8	34.4	9751.7	34878.1	19909.2
51	3.6	359	43082.0	33.9	9340.0	33742.0	20049.5
52	3.6	346	41574.3	33.5	8958.2	32616.1	20158.9
53	3.6	334	40104.5	33.1	8603.4	31501.1	20237.9
54	3.6	322	38670.4	32.6	8273.0	30397.4	20286.9
55	3.6	311	37270.0	32.2	7964.9	29305.1	20306.0
56	3.6	299	35901.4	31.9	7677.1	28224.4	20295.3
57	3.6	288	34562.8	31.5	7407.6	27155.2	20254.7
58	3.6	277	33252.5	31.1	7154.9	26097.6	20184.2
59	3.6	266	31968.9	30.8	6917.6	25051.3	20083.3
60	3.7	256	30710.4	30.4	6694.3	24016.1	19951.8
61	3.7	246	29475.6	30.1	6483.8	22991.7	19789.1
62	3.8	236	28263.0	29.7	6285.1	21977.9	19594.5
63	3.8	226	27071.3	29.4	6097.0	20974.3	19367.3
64	3.9	216	25899.3	29.1	5918.6	19980.6	19106.4
65	3.9	206	24745.6	28.8	5749.1	18996.5	18810.9



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$
$b = W-a$
$P_A = b*\tan(\alpha-\phi_{FS})$
$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 20306.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$
EFP 45.1 pcf

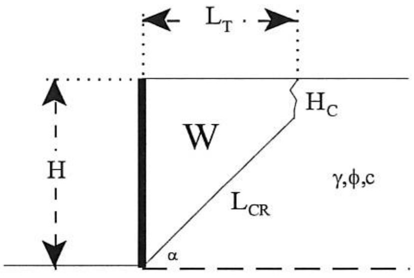
Design Wall for an Equivalent Fluid Pressure: 45 pcf Cantelevered, Triangular Distrib

Geotechnologies, Inc.

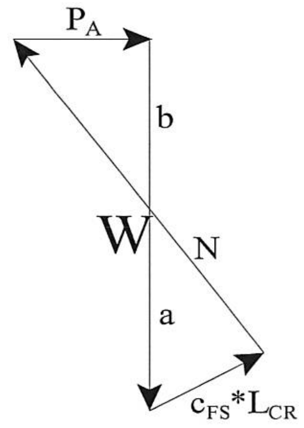
Project: Balboa cove Group
File No.: 21671
Description: alluvium

Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:		
Retaining Wall Height	(H)	40.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	20.3 degrees
	(c_{FS})	150.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.5	941	112936.8	55.2	23007.1	89929.8	32232.7
41	4.4	909	109104.3	54.3	21586.1	87518.3	33103.1
42	4.3	878	105407.4	53.4	20306.4	85101.0	33898.1
43	4.2	849	101838.7	52.6	19149.9	82688.8	34621.3
44	4.1	820	98390.7	51.7	18101.0	80289.7	35276.0
45	4.0	792	95056.6	51.0	17146.8	77909.7	35865.4
46	3.9	765	91829.6	50.2	16276.1	75553.4	36391.9
47	3.8	739	88703.4	49.5	15479.4	73224.0	36857.8
48	3.8	714	85672.1	48.8	14748.5	70923.6	37265.3
49	3.7	689	82730.0	48.1	14076.3	68653.7	37616.0
50	3.7	666	79871.9	47.4	13456.6	66415.3	37911.4
51	3.6	642	77092.9	46.8	12884.2	64208.7	38152.8
52	3.6	620	74388.3	46.2	12354.3	62034.0	38341.0
53	3.6	598	71753.8	45.6	11862.8	59891.0	38477.0
54	3.6	577	69185.2	45.0	11406.0	57779.2	38561.2
55	3.6	556	66678.7	44.4	10980.7	55698.0	38594.1
56	3.6	535	64230.8	43.9	10584.1	53646.7	38575.7
57	3.6	515	61838.0	43.4	10213.6	51624.4	38505.9
58	3.6	496	59497.1	42.9	9866.8	49630.2	38384.5
59	3.6	477	57205.1	42.4	9541.9	47663.2	38211.0
60	3.7	458	54959.1	42.0	9236.8	45722.4	37984.7
61	3.7	440	52756.5	41.5	8949.9	43806.7	37704.6
62	3.8	422	50594.8	41.1	8679.6	41915.1	37369.6
63	3.8	404	48471.4	40.6	8424.7	40046.7	36978.3
64	3.9	387	46384.0	40.2	8183.7	38200.3	36529.0
65	3.9	369	44330.6	39.8	7955.5	36375.1	36019.7



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 38594.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 48.2 pcf

Design Wall for an Equivalent Fluid Pressure: 48 pcf Cantelevered, Triangular Distrib

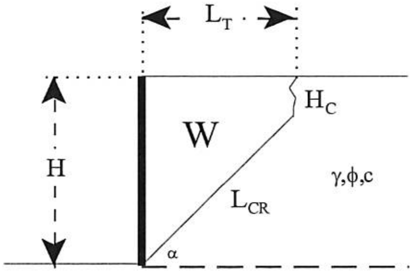
Geotechnologies, Inc.

Project: Balboa cove Group
File No.: 21671
Description: alluvium

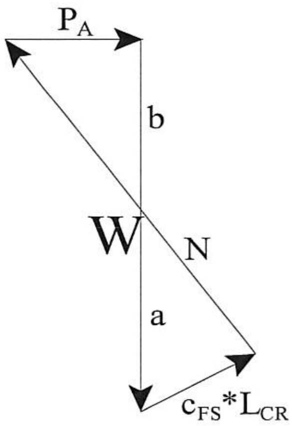
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	50.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	20.3 degrees
	(c_{FS})	150.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.5	1477	177291.5	70.7	29494.6	147797.0	52973.5
41	4.4	1427	171224.2	69.5	27648.1	143576.1	54306.6
42	4.3	1378	165380.5	68.4	25988.7	139391.8	55523.6
43	4.2	1331	159746.6	67.2	24491.7	135254.8	56630.3
44	4.1	1286	154309.3	66.1	23136.4	131172.9	57632.1
45	4.0	1242	149056.6	65.1	21905.2	127151.3	58533.5
46	3.9	1200	143976.7	64.1	20783.4	123193.3	59338.6
47	3.8	1159	139059.2	63.1	19758.3	119300.9	60051.0
48	3.8	1119	134293.9	62.2	18818.9	115474.9	60673.9
49	3.7	1081	129671.5	61.3	17956.1	111715.4	61209.9
50	3.7	1043	125183.3	60.5	17161.6	108021.7	61661.3
51	3.6	1007	120821.3	59.6	16428.4	104392.8	62030.1
52	3.6	971	116577.7	58.9	15750.4	100827.3	62317.8
53	3.6	937	112445.7	58.1	15122.2	97323.5	62525.6
54	3.6	903	108418.5	57.4	14538.9	93879.5	62654.3
55	3.6	871	104489.9	56.7	13996.5	90493.4	62704.4
56	3.6	839	100654.2	56.0	13491.1	87163.1	62676.3
57	3.6	808	96906.0	55.3	13019.5	83886.4	62569.7
58	3.6	777	93240.0	54.7	12578.8	80661.3	62384.3
59	3.6	747	89651.5	54.1	12166.1	77485.4	62119.2
60	3.7	718	86136.0	53.5	11779.2	74356.8	61773.3
61	3.7	689	82689.2	52.9	11415.9	71273.3	61345.3
62	3.8	661	79307.1	52.4	11074.2	68232.9	60833.3
63	3.8	633	75985.7	51.8	10752.4	65233.4	60235.1
64	3.9	606	72721.6	51.3	10448.8	62272.8	59548.2
65	3.9	579	69511.2	50.8	10161.9	59349.3	58769.5



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 62704.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 50.2 pcf

Design Wall for an Equivalent Fluid Pressure: 50 pcf Cantelevered, Triangular Distrib

Geotechnologies, Inc.

Project:	Balboa cove Group
File No.:	21671
Geologic Material	Alluvium

Soil Weight	γ	125	pcf
Internal Friction Angle	ϕ	29	degrees
Cohesion	c	225	psf
Height of Retaining Wall	H	45	feet

Cantilever Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.515
$\sigma'_v = \gamma H$	5625.0 psf

$\sigma'_h =$ 2897.9 psf

EFP = 64.4 pcf

$P_o =$ 65203.8 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 64 pcf

Geotechnologies, Inc.

Project: Balboa cove Group

File No.: 21671

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	45.0	feet
Retained Soil Unit Weight:	(γ)	120.0	pcf
Short Duration Acceleration	(SDs)	1.307	g
Horizontal Ground Acceleration: (1/2 of Sds/2.5)	(k_h)	0.26	g

Seismic Increment (ΔP_{AE}):

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

$\Delta P_{AE} =$ 23820.1 lbs/ft

Force applied at 0.6H above the base of the wall

Transfer load to 2/3 of the height of the wall

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

T = 21438.1 lbs/ft

$EFP = 2*T/H^2$

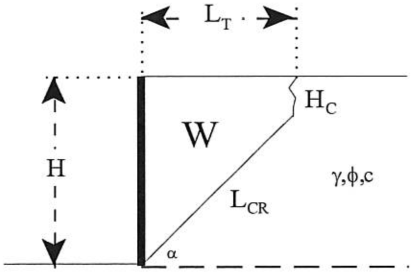
EFP = 21.2 pcf Triangular shape

Geotechnologies, Inc.
Project: Balboa cove Group
File No.: 21671
Description: alluvium

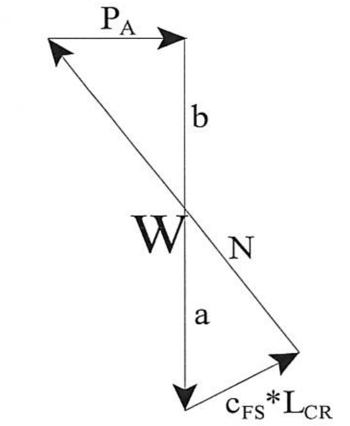
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	23.9 degrees
	(c_{FS})	180.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.5	213	25617.6	21.1	12509.5	13108.1	3779.8
41	6.2	208	24969.1	21.1	11794.5	13174.6	4049.3
42	5.9	203	24300.5	21.0	11134.7	13165.7	4299.5
43	5.7	197	23621.3	20.9	10526.7	13094.6	4530.6
44	5.6	191	22938.4	20.8	9966.5	12971.9	4743.3
45	5.4	185	22256.7	20.7	9450.1	12806.6	4937.8
46	5.3	180	21579.5	20.5	8973.8	12605.7	5114.9
47	5.1	174	20909.2	20.3	8533.8	12375.4	5274.8
48	5.0	169	20247.4	20.2	8126.8	12120.6	5418.1
49	4.9	163	19595.3	20.0	7749.9	11845.4	5545.1
50	4.9	158	18953.4	19.8	7400.2	11553.2	5656.2
51	4.8	153	18322.1	19.6	7075.1	11247.0	5751.7
52	4.7	148	17701.7	19.4	6772.5	10929.2	5832.0
53	4.7	142	17092.0	19.2	6490.3	10601.7	5897.3
54	4.7	137	16492.9	19.0	6226.5	10266.4	5947.7
55	4.6	133	15904.3	18.8	5979.7	9924.6	5983.4
56	4.6	128	15325.7	18.6	5748.2	9577.5	6004.5
57	4.6	123	14757.0	18.3	5530.6	9226.3	6011.2
58	4.6	118	14197.6	18.1	5325.8	8871.8	6003.3
59	4.6	114	13647.2	17.9	5132.5	8514.7	5980.9
60	4.7	109	13105.4	17.7	4949.8	8155.6	5944.0
61	4.7	105	12571.7	17.5	4776.5	7795.2	5892.3
62	4.7	100	12045.7	17.3	4611.9	7433.8	5825.8
63	4.8	96	11527.0	17.1	4455.0	7071.9	5744.2
64	4.9	92	11015.0	16.8	4305.1	6709.9	5647.3
65	4.9	88	10509.4	16.6	4161.4	6348.0	5534.9



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 6011.2 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 30.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 30 pcf Cantelevered, Triangular Distrib

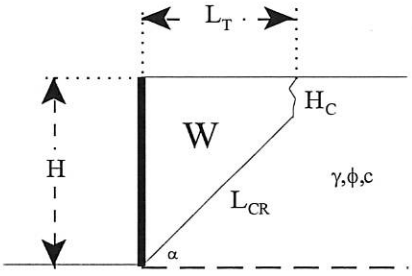
Geotechnologies, Inc.

Project: Balboa cove Group
File No.: 21671
Description: alluvium

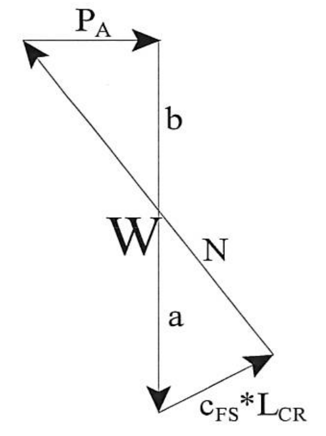
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	30.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	23.9 degrees
	(c_{FS})	180.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.5	511	61370.2	36.6	21748.7	39621.4	11425.1
41	6.2	496	59480.2	36.3	20331.5	39148.7	12032.7
42	5.9	480	57618.8	36.0	19056.3	38562.5	12593.2
43	5.7	465	55792.3	35.6	17905.6	37886.7	13108.5
44	5.6	450	54004.3	35.2	16864.0	37140.3	13580.6
45	5.4	435	52256.7	34.8	15918.5	36338.2	14011.0
46	5.3	421	50550.1	34.4	15057.7	35492.4	14401.3
47	5.1	407	48884.6	34.0	14271.8	34612.8	14753.1
48	5.0	394	47259.6	33.6	13552.5	33707.0	15067.5
49	4.9	381	45673.9	33.2	12892.5	32781.4	15345.6
50	4.9	368	44126.4	32.8	12285.2	31841.1	15588.6
51	4.8	355	42615.7	32.4	11725.3	30890.3	15797.3
52	4.7	343	41140.2	32.1	11207.9	29932.3	15972.5
53	4.7	331	39698.6	31.7	10728.7	28969.9	16114.7
54	4.7	319	38289.2	31.3	10283.9	28005.3	16224.5
55	4.6	308	36910.5	31.0	9870.3	27040.2	16302.2
56	4.6	296	35561.0	30.6	9484.8	26076.2	16348.2
57	4.6	285	34239.2	30.3	9124.8	25114.4	16362.6
58	4.6	275	32943.7	29.9	8788.0	24155.7	16345.5
59	4.6	264	31673.0	29.6	8472.3	23200.7	16296.8
60	4.7	254	30425.9	29.3	8175.7	22250.2	16216.3
61	4.7	243	29201.0	28.9	7896.5	21304.5	16103.8
62	4.7	233	27997.0	28.6	7633.2	20363.9	15958.8
63	4.8	223	26812.8	28.3	7384.2	19428.6	15780.9
64	4.9	214	25647.0	28.0	7148.2	18498.8	15569.3
65	4.9	204	24498.6	27.7	6924.0	17574.6	15323.3



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 16362.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 36.4 pcf

Design Shoring for an Equivalent Fluid Pressure: 36 pcf Cantelevered, Triangular Distrib

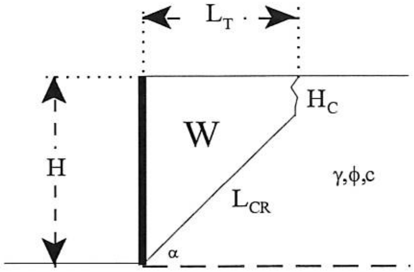
Geotechnologies, Inc.

Project: Balboa cove Group
File No.: 21671
Description: alluvium

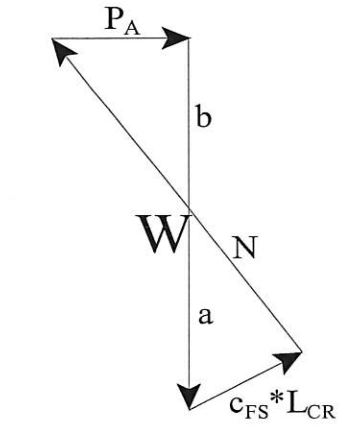
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	40.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	23.9 degrees
	(c_{FS})	180.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.5	929	111423.8	52.2	30988.0	80435.8	23194.2
41	6.2	898	107795.6	51.5	28868.4	78927.2	24258.9
42	5.9	869	104264.6	50.9	26977.9	77286.6	25239.1
43	5.7	840	100831.8	50.2	25284.5	75547.3	26138.8
44	5.6	812	97496.6	49.6	23761.6	73735.0	26961.6
45	5.4	785	94256.7	48.9	22386.9	71869.8	27711.0
46	5.3	759	91109.1	48.3	21141.6	69967.4	28389.8
47	5.1	734	88050.3	47.7	20009.9	68040.4	29001.0
48	5.0	709	85076.5	47.1	18978.2	66098.3	29546.8
49	4.9	685	82183.9	46.5	18035.0	64148.9	30029.4
50	4.9	661	79368.6	45.9	17170.3	62198.2	30450.7
51	4.8	639	76626.6	45.3	16375.6	60251.0	30812.4
52	4.7	616	73954.2	44.8	15643.3	58310.9	31115.8
53	4.7	595	71347.9	44.2	14967.1	56380.7	31362.1
54	4.7	573	68804.0	43.7	14341.3	54462.7	31552.2
55	4.6	553	66319.2	43.2	13760.8	52558.4	31686.8
56	4.6	532	63890.3	42.7	13221.3	50669.0	31766.4
57	4.6	513	61514.3	42.2	12718.9	48795.4	31791.4
58	4.6	493	59188.2	41.7	12250.2	46938.0	31761.8
59	4.6	474	56909.2	41.3	11812.0	45097.2	31677.5
60	4.7	456	54674.6	40.8	11401.6	43273.0	31538.1
61	4.7	437	52482.0	40.4	11016.5	41465.5	31343.3
62	4.7	419	50328.8	39.9	10654.4	39674.4	31092.2
63	4.8	402	48212.8	39.5	10313.3	37899.5	30783.9
64	4.9	384	46131.8	39.1	9991.3	36140.4	30417.2
65	4.9	367	44083.6	38.7	9686.7	34396.8	29990.7



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 31791.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 39.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 40 pcf Cantelevered, Triangular Distrib

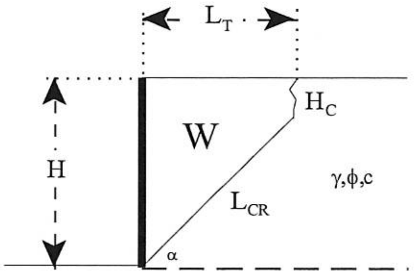
Geotechnologies, Inc.

Project: Balboa cove Group
File No.: 21671
Description: alluvium

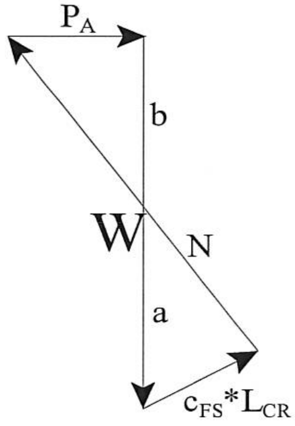
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	50.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	29.0 degrees
Cohesion of Retained Soils	(c)	225.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	23.9 degrees
	(c_{FS})	180.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.5	1465	175778.5	67.7	40227.2	135551.3	39087.0
41	6.2	1416	169915.5	66.8	37405.4	132510.1	40728.1
42	5.9	1369	164237.6	65.8	34899.5	129338.1	42237.3
43	5.7	1323	158739.7	64.9	32663.4	126076.3	43621.5
44	5.6	1278	153415.2	64.0	30659.2	122756.1	44886.5
45	5.4	1235	148256.7	63.1	28855.3	119401.4	46037.8
46	5.3	1194	143256.3	62.2	27225.6	116030.7	47080.4
47	5.1	1153	138406.1	61.4	25748.0	112658.1	48018.5
48	5.0	1114	133698.3	60.5	24403.9	109294.4	48856.0
49	4.9	1076	129125.4	59.7	23177.6	105947.9	49596.3
50	4.9	1039	124679.9	58.9	22055.4	102624.5	50242.5
51	4.8	1003	120354.9	58.2	21025.8	99329.1	50797.0
52	4.7	968	116143.7	57.4	20078.8	96064.9	51262.1
53	4.7	934	112039.8	56.7	19205.6	92834.2	51639.5
54	4.7	900	108037.3	56.1	18398.7	89638.6	51930.8
55	4.6	868	104130.4	55.4	17651.4	86479.0	52137.1
56	4.6	836	100313.8	54.7	16957.9	83355.9	52259.1
57	4.6	805	96582.3	54.1	16313.1	80269.2	52297.4
58	4.6	774	92931.1	53.5	15712.4	77218.7	52252.0
59	4.6	745	89355.6	52.9	15151.7	74203.9	52122.8
60	4.7	715	85851.5	52.4	14627.5	71224.0	51909.3
61	4.7	687	82414.6	51.8	14136.5	68278.2	51610.7
62	4.7	659	79041.1	51.3	13675.7	65365.4	51225.9
63	4.8	631	75727.2	50.7	13242.5	62484.7	50753.2
64	4.9	604	72469.3	50.2	12834.4	59634.9	50191.0
65	4.9	577	69264.2	49.7	12449.4	56814.8	49536.9



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 52297.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 41.8 pcf

Design Shoring for an Equivalent Fluid Pressure: 42 pcf Cantelevered, Triangular Distrib

Geotechnologies, Inc.

Project: Balboa Cove Group
File No.: 21671
Bedrock Type Alluvium

Bearing Capacity Calculations

Input

Soil Density	(γ)	120	pcf
Friction Angle	(ϕ)	30	degrees
Cohesion	(c)	225	psf
Footing Width	(B)	1	ft
Footing Depth	(D)	2	ft
N_γ		17.6	
N_q		19.6	
N_c		33.0	
Factor of Safety	(FS)	3	

Frictional Coefficient

$f_c = (\tan \phi)/1.5$ 0.38

Passive Pressure

$\sigma_{p,\,ult} = \gamma D^* \tan^2(45+\phi/2) + 2c^* \tan(45+\phi/2)$

$\sigma_{p,\,allow} = \sigma_{p,\,ult} / 2$

$\sigma_{p,\,allow} =$ 750 psf

Continuous Footing

$q_{ult} = cN_c + \gamma DN_q + \gamma BN_\gamma/2$

$q_{allow} = q_{ult}/FS$

$q_{allow} =$ 4,395 psf

Square Footing

$q_{ult} = 1.3cN_c + \gamma DN_q + 0.4\gamma BN_\gamma$

$q_{allow} = q_{ult}/FS$

$q_{allow} =$ 5,067 psf

Circular Footing

$q_{ult} = 1.3cN_c + \gamma DN_q + 0.6\gamma RN_\gamma$; (R=B/2)

$q_{allow} = q_{ult}/FS$

$q_{allow} =$ 4,997 psf

Increase per foot of Depth

$q_{increase,\,depth} =$ 784 psf

Increase per foot of Width

$q_{increase,\,width} =$ 282 psf

	Recommended Bearing Capacity (psf)	Bearing Increase per foot of Depth (psf)	Bearing Increase per foot of Width (psf)
Continuous Footing	500	250	250
Square Column Footing	5000	250	250
Circular Footing	3600	250	250

Geotechnologies, Inc.

Project: Reliable Properties
File No.: 21086
Sample: Fill/Alluvium
Depth: 0-5 feet

SHRINKAGE CALCULATIONS

Properties of In-situ Soils (Borrow)

. Dry Density = 103.6 pcf
Moisture Content = 14.9 %
Density Gravity Water = 62.4 pcf
Specific Gravity of Solids = 2.66

Calculations

VOL.		**Borrow**		WT.
	0.147	AIR	0.00	
1.141	0.282	WATER	17.61	118.22
	0.712	SOLIDS	118.22	

Properties of Engineered Fill Soils

Percent compaction = 92.0 %
Maximum Dry Density = 128.5 pcf
Dry Density = 118.2 pcf
Optimum Moisture Content = 9.9 %

Calculations

VOL.		**Fill**		WT.
	0.100	AIR	0.00	
1.000	0.188	WATER	11.70	129.92
	0.712	SOLIDS	118.22	

Shrinkage = 14.1%

Geotechnologies, Inc.

Project: Reliable Properties
File No.: 21086
Description: Liquefaction Analysis (2% Exceedance in 50 Years)
Boring No: 3

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.6
Peak Ground Horizontal Acceleration, PGA (g):	0.71
Calculated Mag.Wtg.Factor:	1.267

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	71.0
Historically Highest Groundwater Level* (ft):	40.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

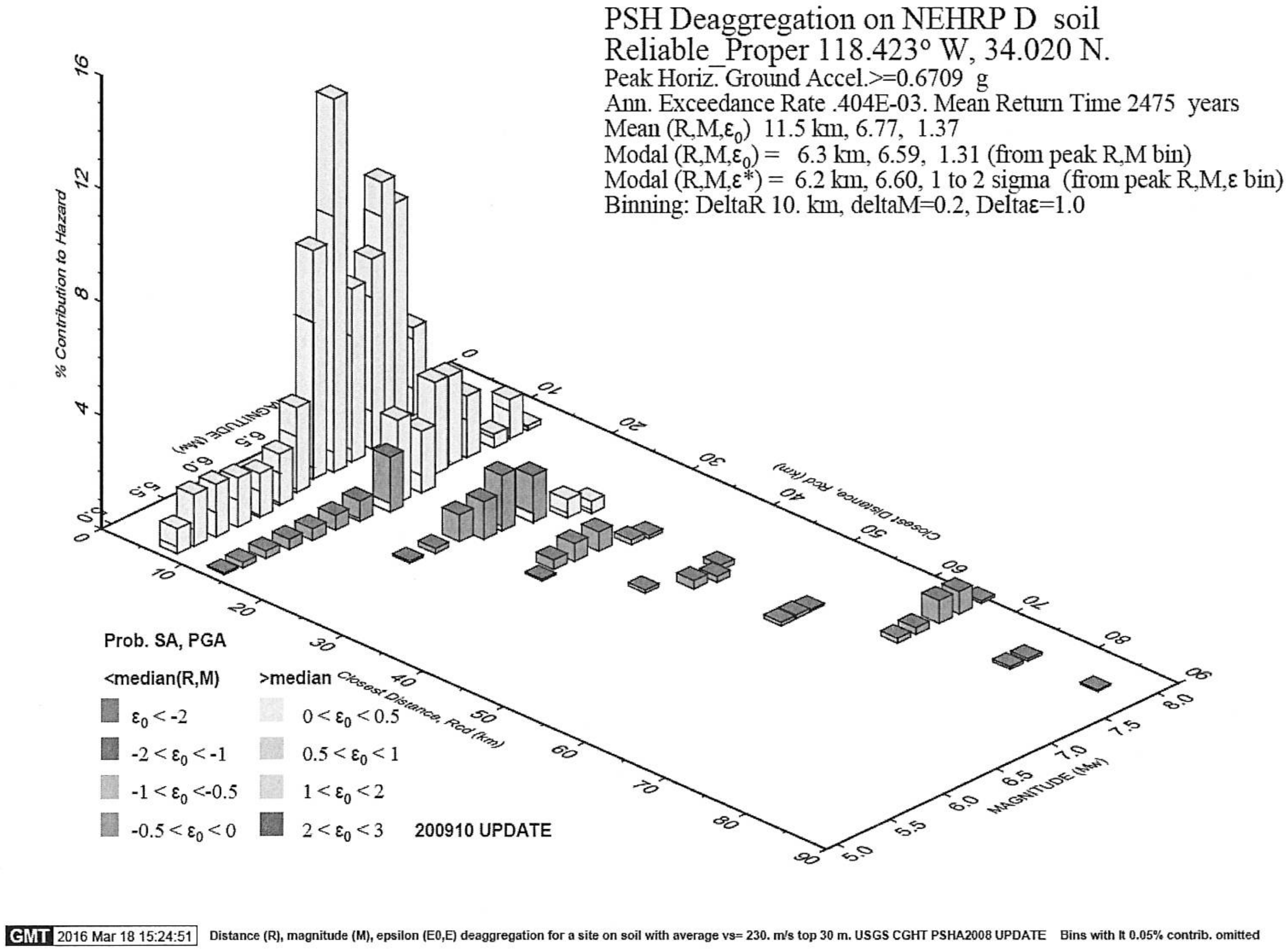
Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

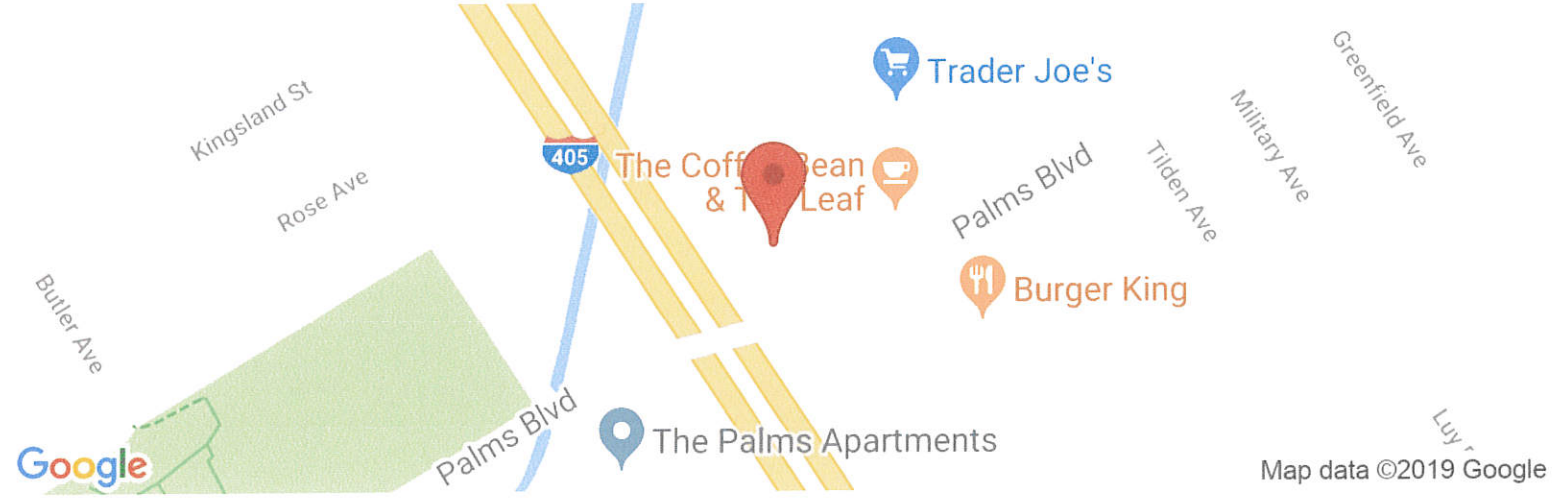
Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	126.0	Unsaturated	Unsaturated	7	5	0.0	0	126.0	126.0	14.5	1.00	0.460	0.212	Non-Liq.	0.00
2	126.0	Unsaturated	Unsaturated	7	5	0.0	0	252.0	252.0	14.5	1.00	0.459	0.212	Non-Liq.	0.00
3	128.5	Unsaturated	Unsaturated	7	5	0.0	0	380.5	380.5	14.5	1.00	0.457	0.212	Non-Liq.	0.00
4	128.5	Unsaturated	Unsaturated	7	5	0.0	0	509.0	509.0	14.5	0.99	0.455	0.212	Non-Liq.	0.00
5	128.5	Unsaturated	Unsaturated	7	5	0.0	0	637.5	637.5	15.6	0.99	0.453	0.224	Non-Liq.	0.00
6	128.5	Unsaturated	Unsaturated	7	5	0.0	0	766.0	766.0	15.6	0.99	0.452	0.224	Non-Liq.	0.00
7	128.5	Unsaturated	Unsaturated	7	5	0.0	0	894.5	894.5	14.6	0.98	0.450	0.212	Non-Liq.	0.00
8	128.5	Unsaturated	Unsaturated	7	5	0.0	0	1023.0	1023.0	13.5	0.98	0.448	0.196	Non-Liq.	0.00
9	128.5	Unsaturated	Unsaturated	7	5	0.0	0	1151.5	1151.5	13.4	0.97	0.446	0.192	Non-Liq.	0.00
10	128.5	Unsaturated	Unsaturated	22	10	0.0	0	1280.0	1280.0	41.2	0.97	0.443	2.000	Non-Liq.	0.00
11	128.5	Unsaturated	Unsaturated	22	10	0.0	0	1408.5	1408.5	39.9	0.96	0.441	2.000	Non-Liq.	0.00
12	128.5	Unsaturated	Unsaturated	22	10	0.0	0	1537.0	1537.0	38.8	0.96	0.439	2.000	Non-Liq.	0.00
13	130.9	Unsaturated	Unsaturated	22	10	0.0	0	1667.9	1667.9	37.7	0.95	0.437	2.000	Non-Liq.	0.00
14	130.9	Unsaturated	Unsaturated	22	10	0.0	0	1798.8	1798.8	36.5	0.95	0.434	2.000	Non-Liq.	0.00
15	132.6	Unsaturated	Unsaturated	74	15	0.0	0	1931.4	1931.4	134.5	0.94	0.432	2.000	Non-Liq.	0.00
16	132.6	Unsaturated	Unsaturated	74	15	0.0	0	2064.0	2064.0	132.1	0.94	0.430	2.000	Non-Liq.	0.00
17	132.6	Unsaturated	Unsaturated	74	15	0.0	0	2196.6	2196.6	130.0	0.93	0.427	2.000	Non-Liq.	0.00
18	132.6	Unsaturated	Unsaturated	74	15	0.0	0	2329.2	2329.2	128.0	0.93	0.424	2.000	Non-Liq.	0.00
19	132.6	Unsaturated	Unsaturated	74	15	0.0	0	2461.8	2461.8	126.1	0.92	0.422	2.000	Non-Liq.	0.00
20	132.6	Unsaturated	Unsaturated	36	20	0.0	0	2594.4	2594.4	60.5	0.92	0.419	2.000	Non-Liq.	0.00
21	132.6	Unsaturated	Unsaturated	36	20	0.0	0	2727.0	2727.0	59.7	0.91	0.417	2.000	Non-Liq.	0.00
22	132.6	Unsaturated	Unsaturated	36	20	0.0	0	2859.6	2859.6	59.0	0.90	0.414	2.000	Non-Liq.	0.00
23	105.7	Unsaturated	Unsaturated	38	25	0.0	0	2965.3	2965.3	61.7	0.90	0.411	2.000	Non-Liq.	0.00
24	105.7	Unsaturated	Unsaturated	38	25	0.0	0	3071.0	3071.0	61.1	0.89	0.409	2.000	Non-Liq.	0.00
25	105.7	Unsaturated	Unsaturated	38	25	0.0	0	3176.7	3176.7	60.6	0.89	0.406	2.000	Non-Liq.	0.00
26	105.7	Unsaturated	Unsaturated	38	25	0.0	0	3282.4	3282.4	60.1	0.88	0.403	2.000	Non-Liq.	0.00
27	127.3	Unsaturated	Unsaturated	38	25	0.0	0	3409.7	3409.7	59.5	0.87	0.400	2.000	Non-Liq.	0.00
28	127.3	Unsaturated	Unsaturated	38	25	0.0	0	3537.0	3537.0	62.0	0.87	0.397	2.000	Non-Liq.	0.00
29	127.3	Unsaturated	Unsaturated	38	25	0.0	0	3664.3	3664.3	61.4	0.86	0.394	2.000	Non-Liq.	0.00
30	127.3	Unsaturated	Unsaturated	26	30	0.0	0	3791.6	3791.6	41.5	0.85	0.392	2.000	Non-Liq.	0.00
31	127.3	Unsaturated	Unsaturated	26	30	0.0	0	3918.9	3918.9	41.1	0.85	0.389	2.000	Non-Liq.	0.00
32	127.3	Unsaturated	Unsaturated	26	30	0.0	0	4046.2	4046.2	40.6	0.84	0.386	2.000	Non-Liq.	0.00
33	138.0	Unsaturated	Unsaturated	35	35	47.5	0	4184.2	4184.2	60.2	0.84	0.383	2.000	Non-Liq.	0.00
34	138.0	Unsaturated	Unsaturated	35	35	47.5	0	4322.2	4322.2	59.8	0.83	0.380	1.998	Non-Liq.	0.00
35	138.0	Unsaturated	Unsaturated	35	35	47.5	0	4460.2	4460.2	59.3	0.82	0.377	1.974	Non-Liq.	0.00
36	138.0	Unsaturated	Unsaturated	35	35	47.5	0	4598.2	4598.2	58.9	0.82	0.374	1.952	Non-Liq.	0.00
37	138.0	Unsaturated	Unsaturated	35	35	47.5	0	4736.2	4736.2	58.5	0.81	0.371	1.929	Non-Liq.	0.00
38	139.8	Unsaturated	Unsaturated	35	35	47.5	0	4876.0	4876.0	58.1	0.80	0.368	1.908	Non-Liq.	0.00
39	139.8	Unsaturated	Unsaturated	35	35	47.5	0	5015.8	5015.8	57.7	0.80	0.365	1.887	Non-Liq.	0.00
40	139.8	Unsaturated	Unsaturated	48	40	51.0	20	5155.6	5155.6	76.5	0.79	0.362	1.866	Non-Liq.	0.00
41	139.8	Unsaturated	Saturated	48	40	51.0	20	5295.4	5233.0	76.2	0.78	0.364	1.855	Non-Liq.	0.00
42	139.8	Unsaturated	Saturated	48	40	51.0	20	5435.2	5310.4	76.0	0.78	0.365	1.844	Non-Liq.	0.00
43	117.0	Unsaturated	Saturated	37	45	8.8	0	5552.2	5365.0	54.7	0.77	0.366	1.836	5.0	0.00
44	117.0	Unsaturated	Saturated	37	45	8.8	0	5669.2	5419.6	54.6	0.77	0.367	1.829	5.0	0.00
45	117.0	Unsaturated	Saturated	37	45	8.8	0	5786.2	5474.2	54.4	0.76	0.368	1.821	5.0	0.00
46	117.0	Unsaturated	Saturated	37	45	8.8	0	5903.2	5528.8	54.3	0.75	0.368	1.814	4.9	0.00
47	117.0	Unsaturated	Saturated	37	45	8.8	0	6020.2	5583.4	54.2	0.75	0.369	1.806	4.9	0.00
48	113.0	Unsaturated	Saturated	37	45	8.8	0	6133.2	5634.0	54.0	0.74	0.369	1.800	4.9	0.00
49	113.0	Unsaturated	Saturated	37	45	8.8	0	6246.2	5684.6	53.9	0.73	0.370	1.793	4.9	0.00
50	113.0	Unsaturated	Saturated	60	50	0.0	0	6359.2	5735.2	86.2	0.73	0.370	1.786	4.8	0.00
51	113.0	Unsaturated	Saturated	60	50	0.0	0	6472.2	5785.8	86.0	0.72	0.370	1.780	4.8	0.00
52	113.0	Unsaturated	Saturated	60	50	0.0	0	6585.2	5836.4	85.8	0.72	0.370	1.773	4.8	0.00
53	125.7	Unsaturated	Saturated	60	50	0.0	0	6710.9	5899.7	85.5	0.71	0.370	1.765	4.8	0.00
54	125.7	Unsaturated	Saturated	60	50	0.0	0	6836.6	5963.0	85.3	0.70	0.370	1.757	4.7	0.00
55	125.7	Unsaturated	Saturated	19	55	87.7	42	6962.3	6026.3	27.7	0.70	0.370	0.380	Non-Liq.	0.00
56	125.7	Unsaturated	Saturated	19	55	87.7	42	7088.0	6089.6	27.6	0.69	0.369	0.375	Non-Liq.	0.00
57	122.4	Unsaturated	Saturated	19	55	87.7	42	7210.4	6149.6	27.5	0.69	0.369	0.371	Non-Liq.	0.00
58	122.4	Unsaturated	Saturated	19	55	87.7	42	7332.8	6209.6	27.4	0.68	0.369	0.367	Non-Liq.	0.00
59	122.4	Unsaturated	Saturated	19	55	87.7	42	7455.2	6269.6	27.3	0.68	0.368	0.364	Non-Liq.	0.00
60	122.4	Unsaturated	Saturated	40	60	98.1	19	7577.6	6329.6	61.5	0.67	0.368	1.713	Non-Liq.	0.00
61	122.4	Unsaturated	Saturated	40	60	98.1	19	7700.0	6389.6	61.3	0.66	0.367	1.706	Non-Liq.	0.00
62	122.4	Unsaturated	Saturated	40	60	98.1	19	7822.4	6449.6	61.2	0.66	0.366	1.699	Non-Liq.	0.00
63	127.0	Unsaturated	Saturated	40	60	98.1	19	7949.4	6514.2	61.1	0.65	0.366	1.691	Non-Liq.	0.00
64	127.0	Unsaturated	Saturated	40	60	98.1	19	8076.4	6578.8	60.9	0.65	0.365	1.684	Non-Liq.	0.00
65	127.0	Unsaturated	Saturated	42	65	45.2	0	8203.4	6643.4	63.6	0.64	0.364	1.676	4.6	0.00
66	127.0	Unsaturated	Saturated	42	65	45.2	0	8330.4	6708.0	63.5	0.64	0.363	1.669	4.6	0.00
67	122.4	Unsaturated	Saturated	42	65	45.2	0	8452.8	6768.0	63.4	0.63	0.363	1.663	4.6	0.00
68	122.4	Unsaturated	Saturated	42	65	45.2	0	8575.2	6828.0	63.2	0.63	0.362	1.656	4.6	0.00
69	122.4	Unsaturated	Saturated	42	65	45.2	0	8697.6	6888.0	63.1	0.62	0.361	1.649	4.6	0.00
70	122.4	Unsaturated	Saturated	78	70	0.0	0	8820.0	6948.0	106.5	0.62	0.360	1.643	4.6	0.00

Total Liquefaction Settlement, S = **0.00 inches**


PSH Deaggregation on NEHRP D soil
Reliable_Proper 118.423° W, 34.020 N.
Peak Horiz. Ground Accel.>=0.6709 g
Ann. Exceedance Rate .404E-03. Mean Return Time 2475 years
Mean (R,M,ε0) 11.5 km, 6.77, 1.37
Modal (R,M,ε0) = 6.3 km, 6.59, 1.31 (from peak R,M bin)
Modal (R,M,ε*) = 6.2 km, 6.60, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
0
4
8
12
16
MAGNITUDE (Mw)
5.5
6.0
6.5
Closest Distance, Rcd (km)
0
10
20
30
40
50
60
70
80
90
MAGNITUDE (Mw)
5.0
5.5
6.0
6.5
7.0
7.5
8.0
Prob. SA, PGA
<median(R,M)
>median
▨ ε0 < -2
▨ -2 < ε0 < -1
▨ -1 < ε0 <-0.5
▨ -0.5 < ε0 < 0
▨ 0 < ε0 < 0.5
▨ 0.5 < ε0 < 1
▨ 1 < ε0 < 2
▨ 2 < ε0 < 3
200910 UPDATE
GMT 2016 Mar 18 15:24:51
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 230. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

File No 21671 Balboa Cove Group

Latitude, Longitude: 34.0195, -118.4232



Date	5/21/2019, 9:27:56 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.96	MCE_R ground motion. (for 0.2 second period)
S_1	0.696	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.96	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.307	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	0.838	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.922	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.96	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.166	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.445	Factored deterministic acceleration value. (0.2 second)
S1RT	0.696	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.773	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.821	Factored deterministic acceleration value. (1.0 second)
PGAd	0.987	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.905	Mapped value of the risk coefficient at short periods
C_{R1}	0.901	Mapped value of the risk coefficient at a period of 1 s

FOUNDATION INVESTIGATION

PROPOSED

KINGPIN LANES BOWLING CENTER

3415 So. SEPULVEDA BOULEVARD AND ROSE AVENUE

LOS ANGELES, CALIFORNIA

CONDUCTED FOR

MR. EDWARD KING

IN COOPERATION WITH

ARTHUR FROEHLICH AND ASSOCIATES

ARCHITECTS

and

SAMUEL SCHULTZ

STRUCTURAL ENGINEER

3415 So Sepulveda

PROJECT NO. 60-032-A

FEBRUARY 11, 1960

CONVERSE FOUNDATION ENGINEERING COMPANY

FOUNDATION INVESTIGATION

The object of this investigation was to obtain information concerning the subsurface soils, on which to base recommendations for the safe and economical design of footings, at the site of the proposed Kingpin Lanes Bowling Center located on the west side of Sepulveda Boulevard at Rose Avenue, Los Angeles, California.

It is understood that the proposed structure will be a one story building with high masonry exterior walls. Perimeter wall loads are expected to be in the range of 2000 to 3000 pounds per foot.

Field and Laboratory Investigation

Three borings were drilled vertically into the subsurface soil strata, to depths varying from 25 to 31 feet, by a power driven, rotary bucket-type auger at the locations shown on Drawing No. 1, "Location of Borings." A continuous log of the soil, as encountered in the test borings, was recorded at the time of drilling and is shown on the "Summary of Test Results," Drawing Nos. 2, 3 and 4. Undisturbed samples of the soil were obtained at frequent intervals below the ground surface and were taken to the laboratory for analysis.

The samples were tested in the laboratory to determine shear resistance, density and moisture content of the natural soils. Consolidation tests were run on representative samples to determine the load-settlement characteristics of the soils. The results of the tests are presented on the "Summary of Test Results" and on Drawing No. 5, "Consolidation Curves."

The sampling and testing procedures are described in the "Appendix" at the end of the report.

Soil Strata

The proposed building area is bordered on the west by the Westwood Channel, which formerly was a slightly meandering unregulated natural drainage ditch. Prior to 1956, the general area was used as a dump. In 1956, when the channel was lined, the dumped material in most of the building and parking area was excavated. Unusable portions were removed from the site and were replaced by clean fill, compacted under control of the Converse Foundation Engineering Company. At that time, the controlled fill area was proposed for use as an apartment house development, for which 90 percent compaction after normal processing of the natural ground surface was considered adequate. The controlled fill is bounded on the south and west sides by uncontrolled fill as shown in our report dated September 20, 1956 (Project No. 56-290-D).

From 11 to 12 feet of mixed silt and clay fill with some inorganic rubble was encountered in the three borings. Laboratory tests indicated densities comparable to those determined during grading, ranging from 90 to 95 percent when compared with maximum densities determined in 1956 for the same soil types. Shear and consolidation tests indicated moderate to high shear strength and only slight compressibility. With the addition of water, the samples tested tended to expand.

The controlled fill was underlain by a moderately firm to moderately soft alluvium two to three feet thick, below which moderately firm to firm predominantly silt and clay mixtures were encountered. In Boring No. 2, firm gravelly silty sand was encountered in the bottom 4 feet of the hole. Tests of the alluvium indicated it has moderate to high shear strength but is somewhat compressible. The underlying silt-clay mixtures demonstrated good shear strength and were only slightly compressible.

In Boring No. 3, which was drilled closest to the channel, the soil was saturated at the natural ground line and again below 25 feet. No free water developed during drilling, but it could accumulate in holes left open for any extended period in this area.

Foundation Recommendations

General

It is recommended that structural loads of the proposed building be carried by footings founded below the fill and alluvium and into the firm natural clay mixtures. The compacted fill is strong and can safely carry light loads. However, because of adjacent uncontrolled fills, deep footings are necessary along the south and west sides of the structure at its present location. Also, the alluvium underlying the fill is variable in character and some differential settlement could result under heavy loads. The use of deep footings throughout will provide for low total and differential settlements.

An alternate solution would be to excavate the adjacent uncontrolled fill to a sufficient distance beyond the building lines and replace it with controlled compacted soil. This would require work on the adjacent property. If this is properly done, footings could be placed on the compacted fill using bearing values comparable to those given in our report on the existing compacted fill.

The existing fill is satisfactory for the support of floor slabs, but precautions should be taken against moisture changes since some of the material is expansive.

Cast-in-Place Belled Piers (caissons)

Cast-in-place concrete piers with belled bases founded at least one foot into firm silty clay will provide adequate support for the structure. A bearing value of 4000 pounds per square foot may safely be used. This value is for dead plus live load only and may be increased 1/3 for combined dead, live and seismic loading. The depths at which suitable soils were first encountered are as follows:

Boring No.	Depth to Bearing Stratum	Recommended Minimum Footing Depth
1	16.5'	17.5'
2	12.8'	13.8'
3	14.8'	15.8'

The total settlement of footings up to 6 feet in diameter is expected to be on the order of 1/4 to 3/8 of an inch. The settlement will vary approximately with the diameter of the footing and the actual applied load.

All excavations should be inspected by the foundation engineer to be sure they are bottomed in the recommended stratum, are of the correct shape and dimensions, and are free of loose or disturbed soils. All holes should be filled with concrete as soon as practicable after completion to minimize the possibility of water seeping into bells. Concrete should be poured in the dry .

Conclusion

The recommended soil bearing values given in this report are based on the assumption that footings will be placed on firm, undisturbed natural soils or properly placed controlled compacted fills. It is important that all footing excavations be inspected prior to pouring concrete to insure that they are all into satisfactory soils and are free from loose or disturbed materials. The recommendations are based on the results of the field and laboratory investigations, combined with analysis by modern soil mechanics principles, and represent our best engineering judgment. If conditions are encountered during construction that appear to be different from those shown by the borings, this office should be notified in order that proper modifications may be made.

Respectfully submitted,

Converse Foundation Engineering Company

By Thomas D. Lake

Reviewed and approved.

TDL/rw

APPENDIX

Description of General Sampling and Laboratory Testing Procedures

Sampling

Undisturbed soil samples are obtained by forcing a special sampling tube into the undisturbed soils at the bottom of the boring. The sampling tube consists of a steel barrel 2.50 inches inside diameter with a lining of one inch long thin brass rings. A special cutting tip is placed on one end and a double ball valve on the other. The sampling tube is driven approximately 18 inches into the soil and a 6 inch section of the central portion of the sample is taken for laboratory tests, the soil being still confined in the brass rings, after extraction from the sampling tube. The samples are taken to the laboratory in close fitting waterproof containers in order to retain the field moisture until completion of the tests. The driving energy is calculated as the average energy in foot-kips required to force the sampling tube through a measured distance of soil at the depth at which the sample is obtained.

Shear Tests

Shear tests are made with a direct shear machine of the strain control type in which the rate of deformation is approximately 0.1 inches per minute. The machine is so designed that the tests are made without removing the samples from the brass liner rings in which they are secured. Each sample is sheared under a normal load equivalent to the weight of the soil above the point of sampling or estimated future weight of soil above this point. In some instances, samples are sheared under various normal loads in order to obtain the internal angle of friction and cohesion. Where considered necessary, samples are saturated and drained prior to shearing in order to simulate extreme field moisture conditions.

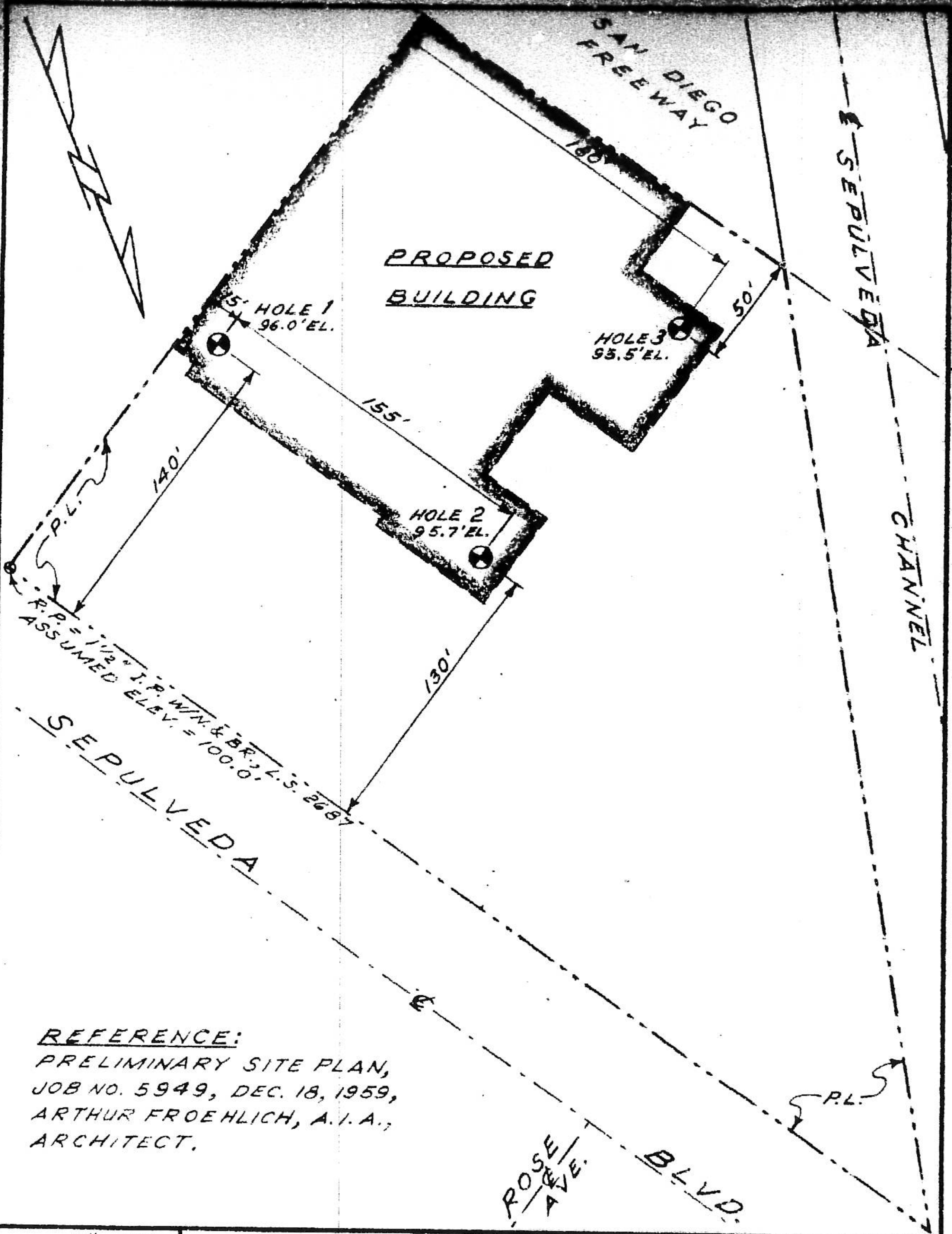
Triaxial shear tests are made on occasion, to check values obtained by the direct shear method.

Consolidation

The apparatus used for the consolidation tests is designed to receive one of the one inch high rings of soil as it comes from the field. Loads are applied in several increments to the upper surface of the test specimen, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit the ready addition or release of water.

Classification

The soils are classified in accordance with the Unified Soil Classification Chart adopted by the U. S. Corps of Engineers and Bureau of Reclamation. Where necessary, classification tests performed in accordance with ASTM procedures are made to substantiate visual classification. These tests might include mechanical analysis, Atterberg limits, and shrinkage tests.



REFERENCE:

PRELIMINARY SITE PLAN,
JOB NO. 5949, DEC. 18, 1959,
ARTHUR FROEHLICH, A.I.A.,
ARCHITECT.

SCALE: 1" = 60'	**LOCATION OF BORINGS**		DRAWING NO.
DWN BY: HRB	KINGPIN LANE BOWLING CENTER LOS ANGELES, CALIFORNIA FOR MR. EDWARD KING		1
DATE: FEB. 10, 1960			
JOB NO. 60-032-A			

CONVERSE FOUNDATION ENGINEERING COMPANY PASADENA, CALIF.

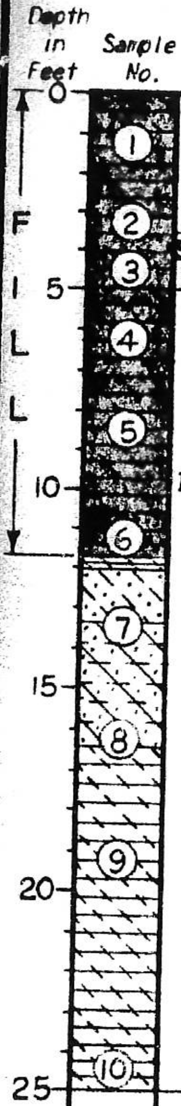
APPROVED FOR PUBLICATION

BY

SUMMARY OF TEST RESULTS

BORING NO. 1

Elevation: 96.0'



Very Moist
Slightly Moist
Moist

Med. s.
Medium Firm
Firm & Medium Firm
Med. s. &Med. f.
Medium Firm
Firm

Gray Brown and Black To Gray
Brown
Dark Gray
Dk. Gr. Brown
Dark Gray Brown
Dk. Gr. Brown
Brown
Red Brown

Mixed: SILTY CLAY & CLAY +15% Gravel to 3" & occasional Brick, Asphalt & Concrete
&CLAYEY SILT-SAND +25% Gravel etc.
← Silty Clay
Fine SILTY SAND (alluvium)
CLAYEY SILT-SAND Porous
SILTY CLAY Slightly Porous
+occasional Gravel to 1"

Sample No.	Code Classification	Drive Energy ft. Kips/ft.	Field Moist. % dry wt.	Dry Density lb/cu.ft.	Shear Res. Kips/sq.ft.
1		1.4	13.2	117.2	1.18
2	CL	1.9	17.1	108.5	0.93
3		7.7	12.5	117.5	1.22
4		6.0	16.8	111.3	1.56
5	& ML/CL	5.0	17.6	111.0	1.53
6		2.8	15.6	110.6	1.80
7	SM				
	ML/CL	1.7	14.8	103.3	1.34
8		5.0	19.4	105.0	2.69
9	CL	6.0	19.5	108.6	3.52
10		6.0	14.2	112.5	2.43

KINGPIN LANE BOWLING CENTER, LOS ANGELES, CALIFORNIA
FOR MR. EDWARD KING

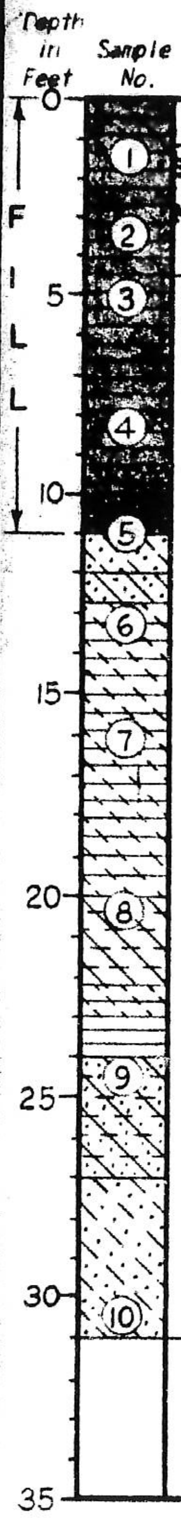
Job No. 60-032-A

CONVERSE FOUNDATION ENGINEERING COMPANY

Drawing No. 2

SUMMARY OF TEST RESULTS
BORING NO. 2

Elevation: 95.7'



Sample No.	Moisture	Consistency	Color	Description		Code Classification	Drive Energy ft. Kips/ft	Field Moist. % dry wt.	Dry Density lb/cu.ft.	Shear Res. Kips/sq.ft.
1	Moist Slightly Moist	Firm	Dark Gray Brown	Mixed: SILTY CLAY + Gravel to 3" &Pieces Concrete, Asphalt in Varying %	+15%	CL	5.7	11.5	105.5	3.79
2							5.5	14.8	112.6	3.18
3	Moist		& Red Brown		+15% & CLAY +30% +5%		8.0	14.0	116.1	2.39
4			Dark Gray Brown &Black to Gray		& SANDY SILT	& ML	7.7	[illegible]	[illegible]	[illegible]
5							6.0	15.0	117.3	2.09
		Medium Soft	Brown&	Fine SILTY SAND	Slightly Porous	SM				
			Dk.Gr.	SANDY SILT (Alluvium)		ML				
6		Medium Firm	Very Dark Gray Brown	SILTY CLAY Slightly Porous		CL	2.8	19.0	101.1	1.61
7		Firm	Dark Gray Brown				6.0	20.0	105.3	2.10
8		Medium Firm		CLAYEY SILT		ML/CL		20.2	97.6	1.38
				SILTY CLAY		CL				
				CLAY +30% Gravel to 3"		CL				
9			Red Brown	CLAYEY SILT-SAND +30% Gravel to 3"		ML/CL	2.4	11.5	114.8	2.05
10		Firm	Dark Gray & Brown	Fine to Coarse SILTY SAND	+40% Gravel to 3" +50% Gravel	SM	10.0	7.6	116.5	3.77

KINGPIN LANE BOWLING CENTER, LOS ANGELES, CALIFORNIA
FOR MR. EDWARD KING

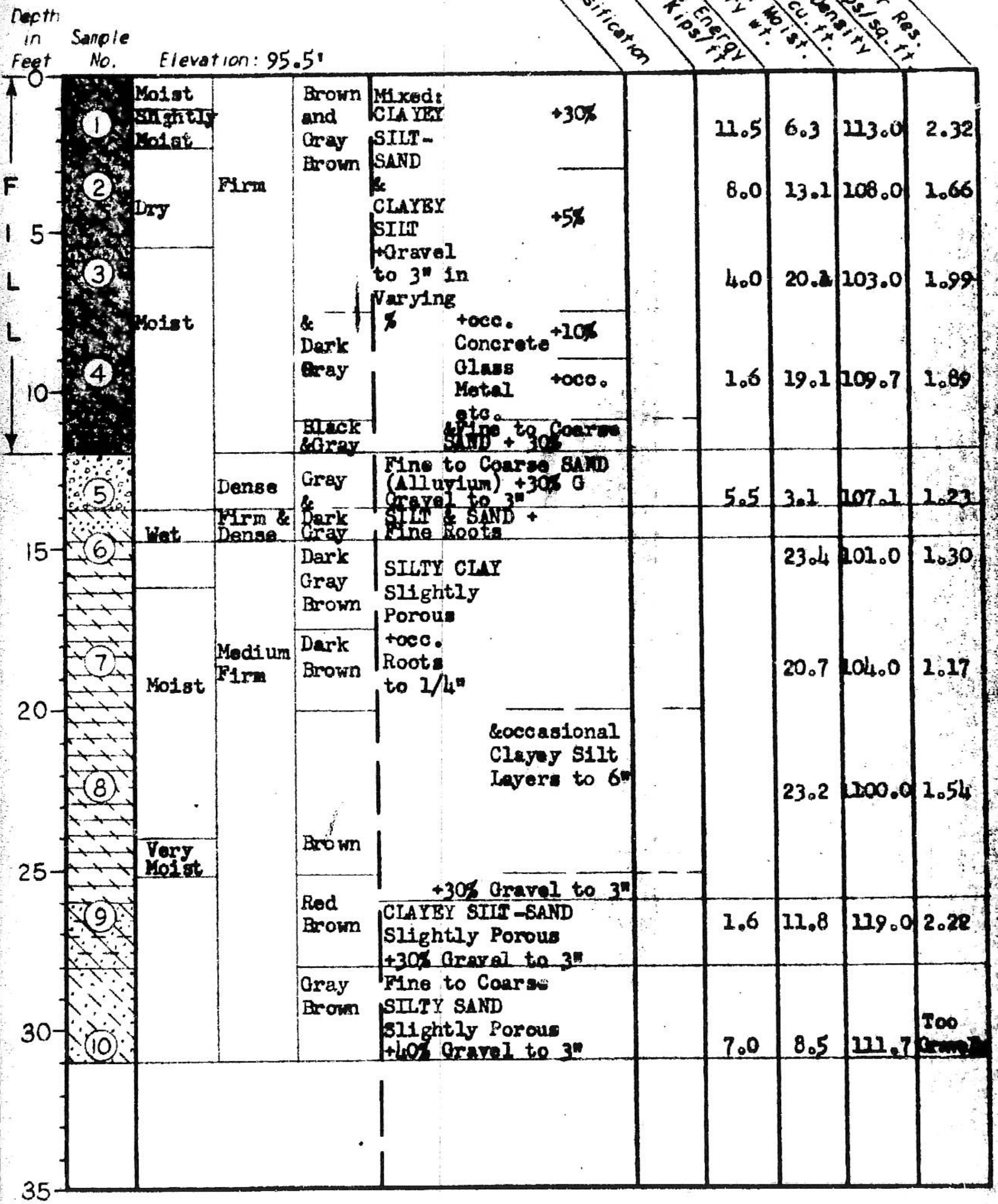
Job No. 60-032-A

CONVERSE FOUNDATION ENGINEERING COMPANY

Drawing No. 3

SUMMARY OF TEST RESULTS

BORING NO. 3



Sample No.					Code Classification	Drive Energy ft. Kips/ft	Field Moist. % dry wt.	Dry Density lb/cu.ft.	Shear Res. Kips/sq.ft
1	Moist Slightly Moist	Firm	Brown and Gray Brown	Mixed: CLAYEY SILT-SAND & CLAYEY SILT +Gravel to 3" in Varying % +30%		11.5	6.3	113.0	2.32
2	Dry			+5%		8.0	13.1	108.0	1.66
3	Moist					4.0	20.1	103.0	1.99
4			& Dark Gray	+occ. Concrete Glass Metal etc. +10% +occ.		1.6	19.1	109.7	1.89
			Black &Gray	&Fine to Coarse SAND + 30%					
5		Dense	Gray & Dark Gray	Fine to Coarse SAND (Alluvium) +30% G Gravel to 3"		5.5	3.1	107.1	1.23
	Wet	Firm & Dense		SILT & SAND + Fine Roots					
6		Medium Firm	Dark Gray Brown	SILTY CLAY Slightly Porous +occ. Roots to 1/4"			23.4	101.0	1.30
7	Moist		Dark Brown				20.7	104.0	1.17
8				&occasional Clayey Silt Layers to 6"			23.2	100.0	1.54
	Very Moist		Brown	+30% Gravel to 3"					
9			Red Brown	CLAYEY SILT-SAND Slightly Porous +30% Gravel to 3"		1.6	11.8	119.0	2.22
10			Gray Brown	Fine to Coarse SILTY SAND Slightly Porous +40% Gravel to 3"		7.0	8.5	111.7	Too Gravelly

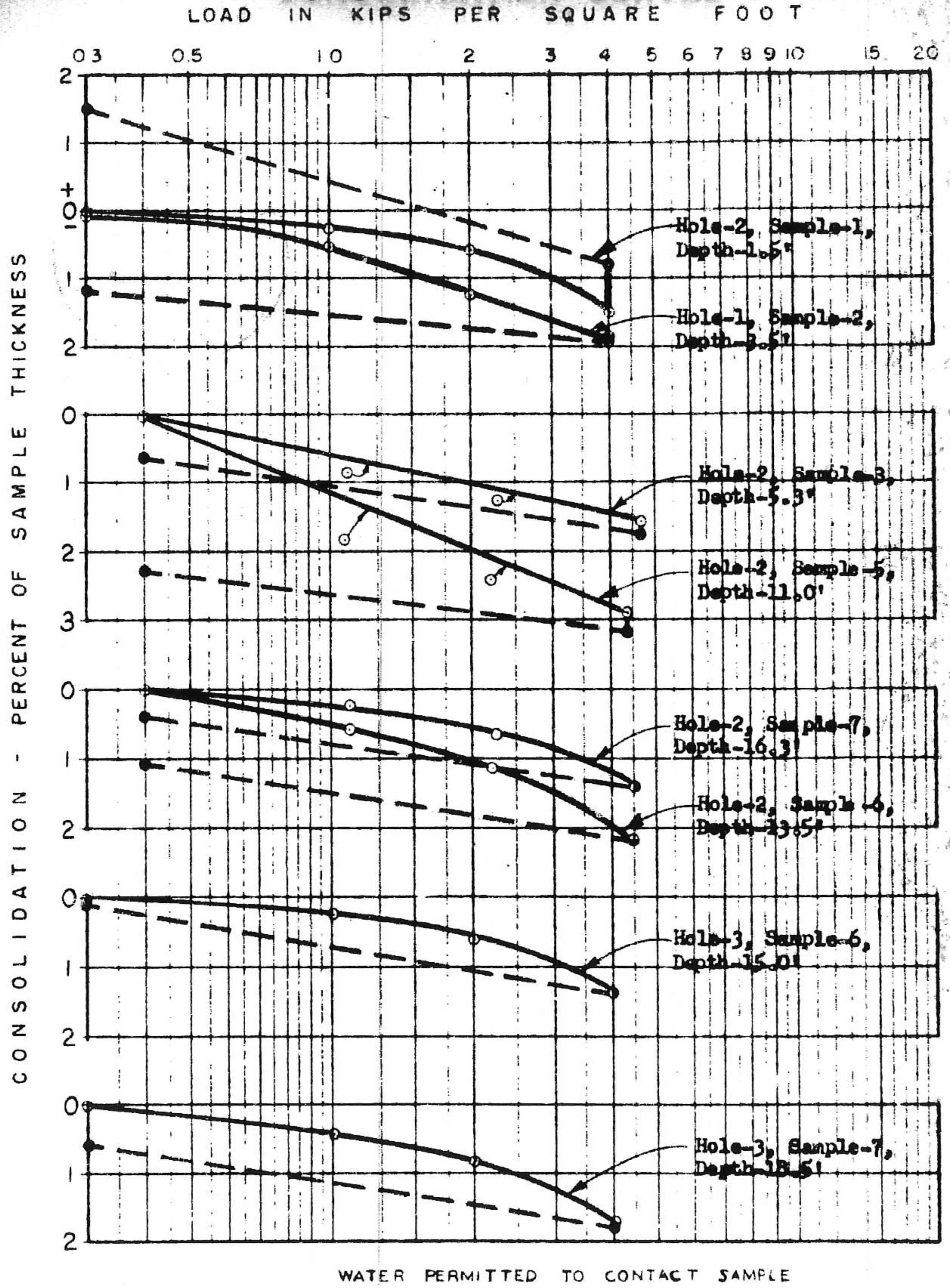
KINGPIN LANE BOWLING CENTER, LOS ANGELES, CALIFORNIA
FOR MR. EDWARD KING

Job No. 60-032-A

CONVERSE FOUNDATION ENGINEERING COMPANY

Drawing No. 4

CONSOLIDATION CURVES



KINGPIN LANE BOWLING CENTER, LOS ANGELES, CALIFORNIA
FOR MR. EDWARD KING

JOB NO.
60-032-A

CONVERSE FOUNDATION ENGINEERING COMPANY

DRAWING NO. 5

SOILS INTERNATIONAL
A CORPORATION
Consulting Foundation Engineers and Geologists

LOS ANGELES
SANTA ANA
VENTURA
HONOLULU
KAHULUI, MAUI

420 S. PINE STREET, SAN GABRIEL, CA 91776 (213) 287-9769 From L. A.: 283-8907

September 8, 1980

L-1176-FG

3415 S. Sepulveda Blvd.

DMG, Inc
435 North Bedford Drive
Beverly Hills, California 90210

Attention: David Bradley

Gentlemen:

This is to report the results of a preliminary soil investigation for the proposed commercial development to be located west of Sepulveda Boulevard at Rose Avenue, West Los Angeles, California.

The details and scope of investigation were set forth in the proposal dated June 26, 1980, following the requirements set forth by the Structural Engineer.

This investigation was made for the purpose of obtaining information on subsurface soils on which to base conclusions for foundation design options, groundwater conditions and other problems for the proposed commercial development planned to be constructed on the west side of Sepulveda Boulevard at Rose Avenue, West Los Angeles. The location of the site relative to surrounding streets and landmarks, is shown on the attached Vicinity Map, Plate 1.

GEOLOGY-SEISMICITY OF THE SITE

A report of the general geology and seismicity of the site has been prepared by David J. Leeds and Associates and forms Part II of this report. Any questions which arise concerning these subjects should be referred to either Mr. Leeds or Dr. Cummings.

PREVIOUS INVESTIGATION

A report of a previous investigation by Converse Foundation Engineering Company has been reviewed and the findings taken into consideration for this investigation. That investigation was made for the present one-story building occupying the southwest portion of the property.

PROPOSED STRUCTURES

It is understood that a 12-story tower with one basement level below grade, 97 by 160, of steel frame construction is planned, with an adjacent 2½ story concrete parking structure, with one basement level below grade.

FIELD INVESTIGATION

Three borings were drilled by means of a power bucket auger to depths ranging from 68 to 69 feet at the locations shown on Plate A. The approximate location of borings was determined by tape measurement from property lines. Approximate elevations of borings were determined by interpolation between survey plan contours. The location and elevation of the borings should be considered accurate only to the degree implied by the method used.

A continuous record of the soils encountered during the drilling was made by the field engineer and is presented on Plates B through J, Logs of Borings.

The lines designating the interface between soil or rock materials on the logs of borings represent approximate boundaries. The transition between materials may be gradual.

Undisturbed samples were secured at frequent intervals from the borings for laboratory testing. The relative sampler penetration resistances exhibited by the soil types encountered are tabulated in the Blows per Foot column of the Logs of Borings.

LABORATORY TESTS

Laboratory testing was programmed following a review of the field investigation, and after considering the probable foundation designs to be evaluated. Laboratory testing included the determination of unit weight, moisture content and shearing resistance of the soil, as well as consolidation characteristics. The results of the tests are plotted or tabulated on the Logs of Borings, Plates B to J, and on Plates K, L, and M, Consolidation Tests.

Details of the sampling and testing procedures are given in the Appendix.

DESCRIPTION OF SITE

The site is presently occupied by a one-story bowling alley which was constructed during the early 1960s. This building occupies much of the area in the southwest portion of the property. The remainder of the property is asphalt paved for parking. The site slopes generally toward the north, with a maximum difference in elevation between borings of approximately 7 feet. Soils encountered consist generally of approximately 10 feet of moderately stiff clayey fill, underlain by approximately 10 feet of recent alluvial moderately stiff clay and moderately compact silty sand. Compact gravelly sand and sandy gravel underlie this stratum, followed by compact sands and, with depth, sandstone and siltstone. A geologic description of these formations is given in the geology-seismicity section of the report.

CONCLUSIONS AND RECOMMENDATIONS

The findings obtained in the borings indicate that support for the proposed structures could be obtained by means of spread footings, drilled, cast-in-place friction piles or drilled cast-in-place belled piers (caissons). The choice will depend upon, or be influenced by, expected loads and whether subterranean basements are planned. These various choices are discussed below.

The allowable bearing value for footings placed in the moderately stiff clay found at depths of 12 to 15 feet below the present surface would be on the order of 3000 to 4000 pounds per square foot, and would be suitable for supporting lighter loads such as might be imposed by a relatively low rise building with one level of subterranean parking.

Higher values could be realized by means of footings founded in the compact gravelly soils found at depths of 18 to 22 feet. It is estimated that bearing values on the order of 6000 to 10,000 pounds per square foot would be available for footings, in the case of deeper excavations, or belled piers (caissons) founded in this strata.

Drilled, cast-in-place friction piles offer another means of support. The following values are based on average conditions found in the three borings and presumed conditions in the portion of the site occupied by the present building. These values assume that the present fills would be left in place.

Length of Pile (ft.) below Present Surface	Allowable Load in Kips for 24-inch dia. Pile
40	150
50	230
60	300

These values are for single piles and would be subject to reduction for efficiency when closely spaced.

Settlements for all three choices is expected to be relatively low. Footings up to 6 feet square founded in the upper natural clay stratum, or up to 10 feet square founded in the gravelly soils is not expected to exceed ½ inch.

Settlement of caissons or piles is expected to be less than one inch, but more likely on the order of ½ inch.

Floor Slabs at Present Grade

Experience with the existing building indicates that the floors, supported on the existing fill, have settled and this settlement is attributed to consolidation of the upper natural soils under the weight of the fill, precipitated by the heavy rains following construction.

Floor slabs supported by the fills would still be subject to subsidence and is inadvisable for this reason. It is recommended that floor slabs on grade obtain support from the natural soils below the fills. Another option would be the removal of the fill and underlying loose or moderately soft alluvial soils and replacement with adequately compacted soil.

Excavations

It is expected that vertical excavations could be made to depths of at least 12 feet, but possibly to as deep as 20 feet where the clay soils extend deeper. In areas such as at Boring No. 1, the cleaner sands would tend to cave and the excavations in these areas would require shoring or to be sloped. In general, excavations below 20 feet would require shoring, or the cuts sloped to from 1:1 to 3/4:1, depending upon local conditions.

Further studies will be required to evaluate stability of cuts to various depths.

Groundwater Conditions

In general, only a slight seepage was encountered, resulting in a very slow flow of water into the borings. Only in Boring No. 2 did water accumulate in the bottom of the hole; at the end of ½ hour, only 6 inches of water had accumulated. A slight seep was encountered at 47 feet in Boring No. 1, but no free water accumulated in the hole during drilling. No water was encountered in Boring No. 3.

Previous borings in 1960 in the area of the present building indicated similar conditions, although seepage was encountered at higher elevations at that time.

Lateral Loads

An allowable lateral bearing value against the sides of footings, piles or caisson shafts of 300 pounds per square foot per foot of depth for the upper natural clays and 400 pounds per square foot per foot of depth for the gravels and gravelly sand to a maximum of 5000 pounds per square foot, may be used provided there is positive contact between the vertical bearing surface and the undisturbed natural soil. Friction between the base of the footing and/or floor slabs and the underlying soil may be assumed as 0.5 times the dead load in the case of the gravels and 750 pounds per square foot of contact area, but in no case more than 50 percent of the dead load for the upper natural clay. Friction and lateral pressure may be combined provided either value is limited to two-thirds of the allowable.

REMARKS

The conclusions and recommendations contained herein are based on the findings and observations made at the three boring locations and data from the previous investigation during 1960.

The conclusions and recommendations should be considered preliminary in nature and subject to revisions influenced by findings in the final phase of investigation.

The investigation was made in accordance with generally accepted engineering procedures, and included such field and laboratory tests considered necessary in the circumstances. In the opinion of the undersigned, this report has been substantiated by mathematical date in conformity with generally accepted engineering principles and presents fairly the information requested. No other warranty, expressed or implied, is made as to the professional advice included in this report.

Respectfully submitted,
SOILS INTERNATIONAL

Robert D. Cousineau, RCE 24269
RDC/mk

APPENDIX

Sampling

Undisturbed samples are obtained by driving a sampling tube by means of a heavy driving weight into undisturbed soils at various intervals below the surface. The number of blows of the weight falling from a specified height required to drive the sample tube through a measured distance, is shown on the Log of Borings. The sampling tube consists of a steel barrel 2.50 inches inside diameter with a lining of one-inch long thin brass rings. A special cutting tip is placed on one end and a ball valve on the other. The sampling tube is driven approximately 18 inches into the soil and a section of the central portion of the sample is taken for laboratory tests, the soil being still confined in the brass rings after extraction from the sampling tube. The samples are taken to the laboratory in close fitting water-proof containers in order to maintain field conditions until completion of tests.

Shearing Resistance

Three types of tests are used: direct shear, triaxial and unconfined compression. For very cohesive soils, unconfined compression tests are applicable. Direct shear tests are most often employed, using a controlled rate of strain. Each sample is sheared under a load approximately equivalent to the expected overburden pressure. By varying the load on a particular sample the internal angle of friction and cohesion may be computed. Where applicable, the moisture content of a sample is raised to simulate expected extreme moisture conditions. Triaxial tests are made on critical soils, as corroboration for other types of shear tests and for studies involving dynamic forces.

Unit Weight and Moisture

One or more one-inch long sections of the sample are cut, trimmed, weighed, oven-dried and reweighed. From these measurements, the equivalent unit weight of the solids in pounds per cubic foot and the per cent moisture are calculated.

Load Consolidation

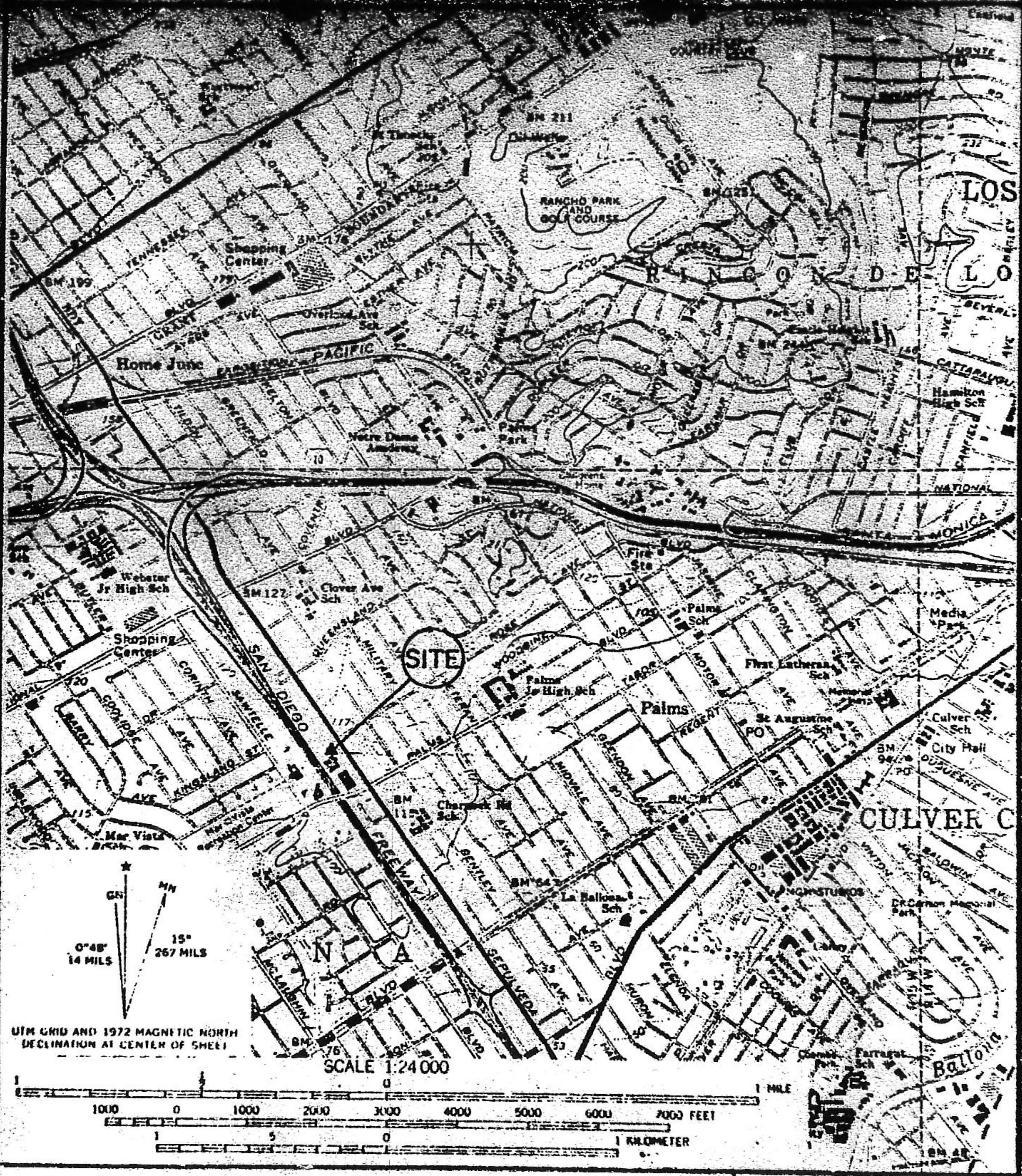
A one-inch high ring of soil is placed in the consolidation apparatus. Loads are applied in increments to the face of the specimen. Deformation, or changes in thickness of the specimens are recorded at selected time intervals. Water is introduced or extracted from the sample through porous disks placed against the top and bottom faces of the specimen.

Expansion

The present method of test is that proposed by the Portland Cement Association and under consideration by such organizations as A. S. C. E., A. S. T. M. and many cities and counties of California. The data so obtained is used to determine an expansion "Index", which in turn forms the basis of categorical classification by public agencies and for design recommendations.

Classification

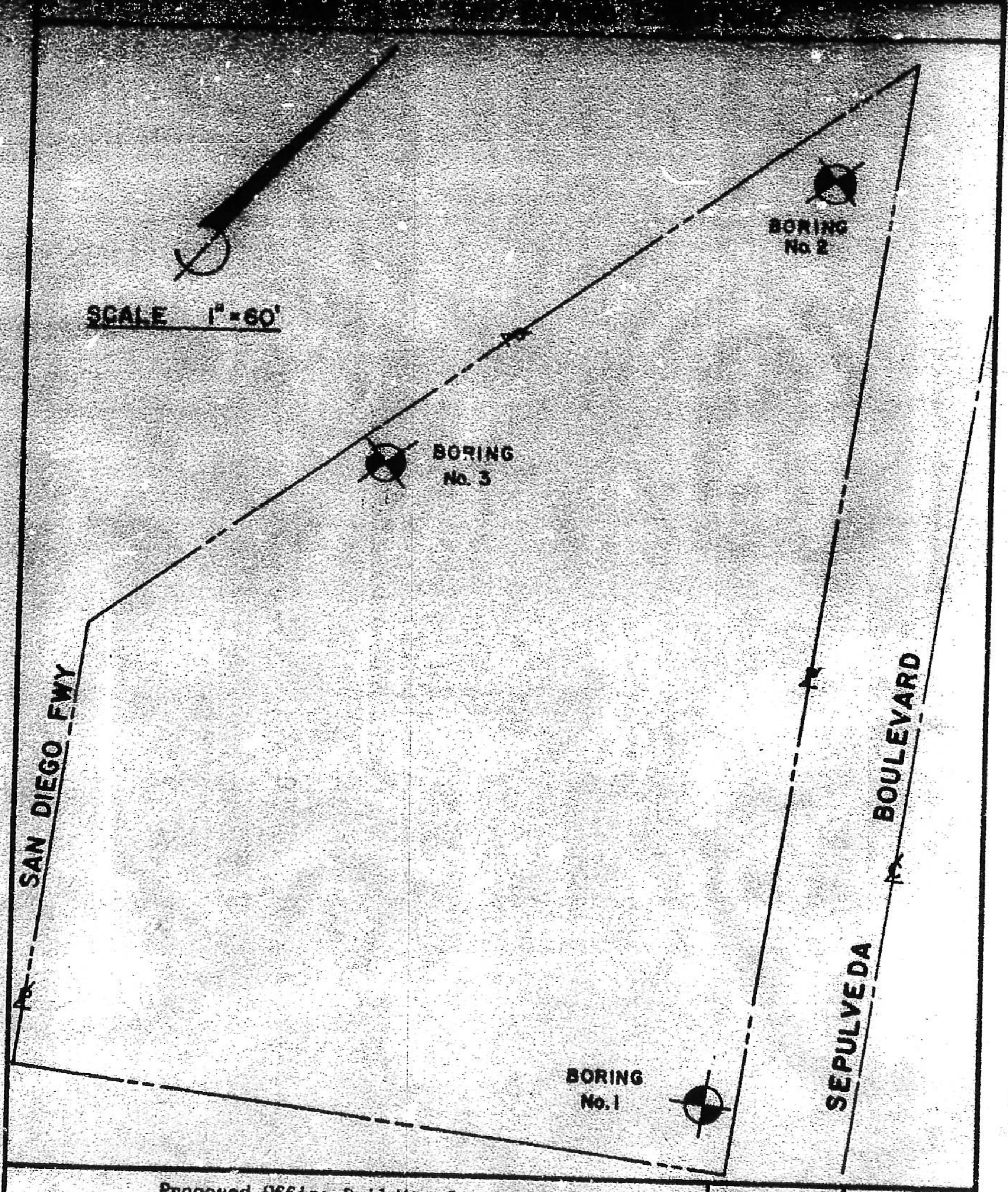
The soils are classified in accordance with the Unified Soil Classification System. Classification tests such as sieve and hydrometer analysis to determine grain size distribution, liquid and plastic limits and shrinkage are used as an aid in substantiating the visual field and laboratory identifications.



Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No. L-1176-FG

PLATE 1



Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	L-1176-FG
	PLATE	A

SOILS INTERNATIONAL

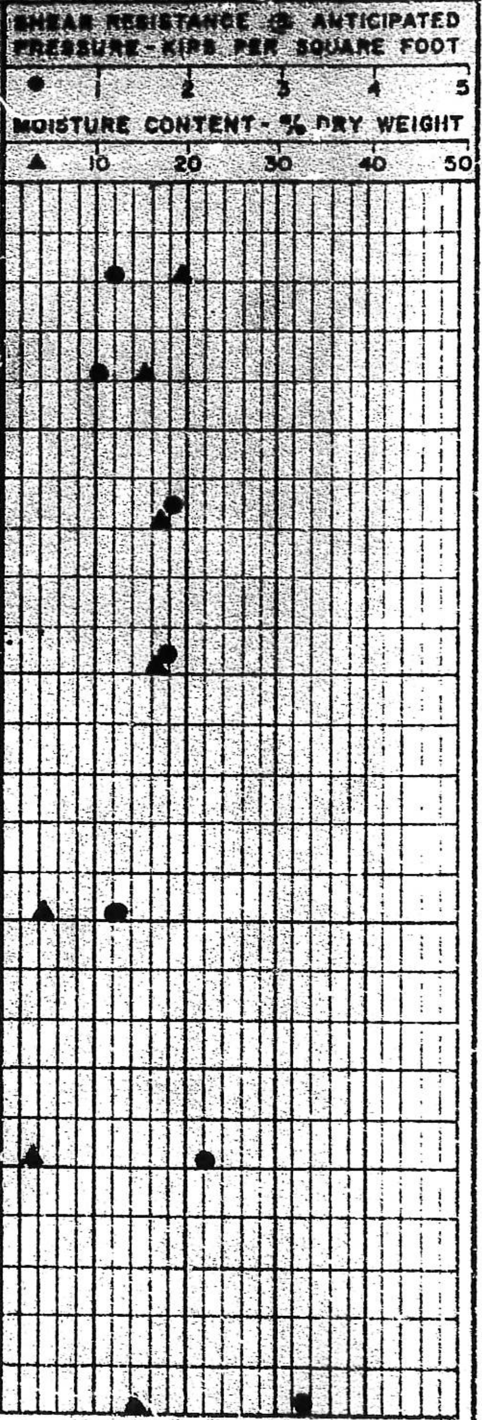
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

201 1-74

DATE DRILLED [illegible] DRILLING EQUIPMENT [illegible] dia. bucket auger

DRIVING WEIGHT [illegible] lbs. at 12" drop | SURFACE ELEVATION 122.0

Depth in Feet	Samples	Blows per Foot	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB. PER CU. FT.
			3" asphalt				
			FILL CLAY very sandy silty w/slate frags	dk. brn.	very moist to moist	stiff	
	■	P					105
	■	P					112
5							
	■	P	concrete frags to 2"				114
			CLAY very silty, porous				
10	■	P					109
			SAND fine, silty	yel. brn.	sl. moist	comp.	
			sl. silty				
15	■	3					100
							15x
			fine-medium clean, with gravel to 1"				
20	■	5					114
							93
			fine, clayey silty	dk.yel brn.	moist		
25	■	3					119



Continued

P = Wt. only - No drop

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

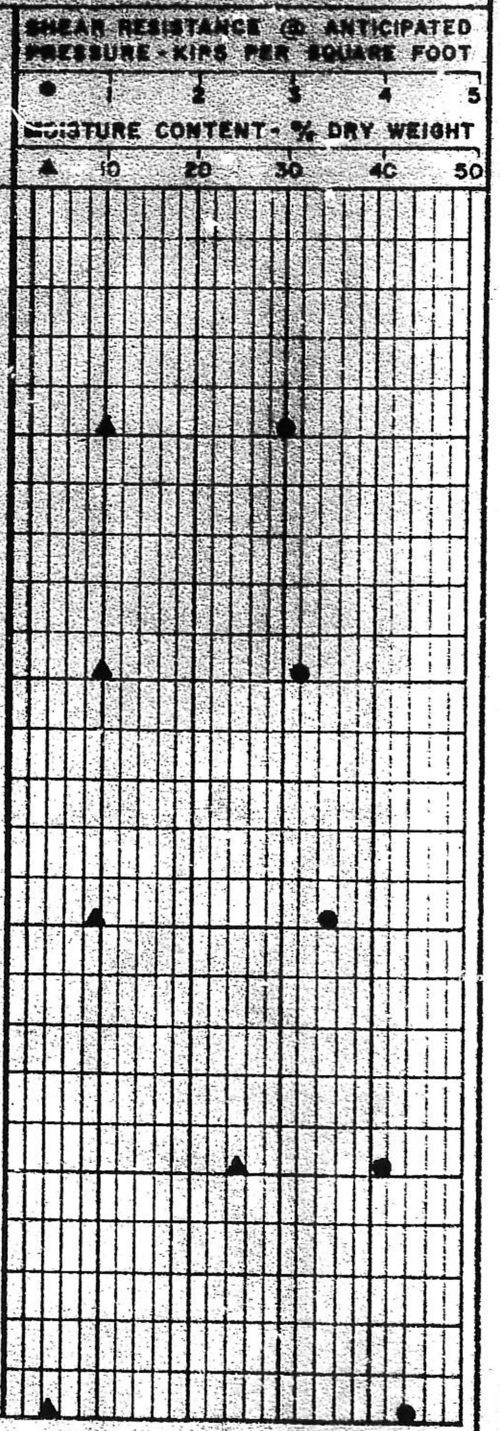
PROJECT No.	L-1176-F
PLATE	B

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

[illegible] EQUIPMENT 18" dia. bucket auger

DRIVING WEIGHT [illegible] @ 12" drop SURFACE ELEVATION

[illegible]	[illegible]	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB. PER CU.FT.
		SAND fine, very clayey silty	dk. yel. brn.	moist	comp.	
		GRAVEL to 2", silty clayey with sand	gray brn.			
30	8					119
		SAND fine, sl. silty	tan			
35	17					98
40	19					98
		fine, v.silty	olive gray	very moist*		
45	7					98
			dk. gray			
				seepage**		
		med. to coarse clean	lt. yel. brn.			
50	40*					108



Continued *Driving Weight - 850 lbs. changed at 50 ft.

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	C

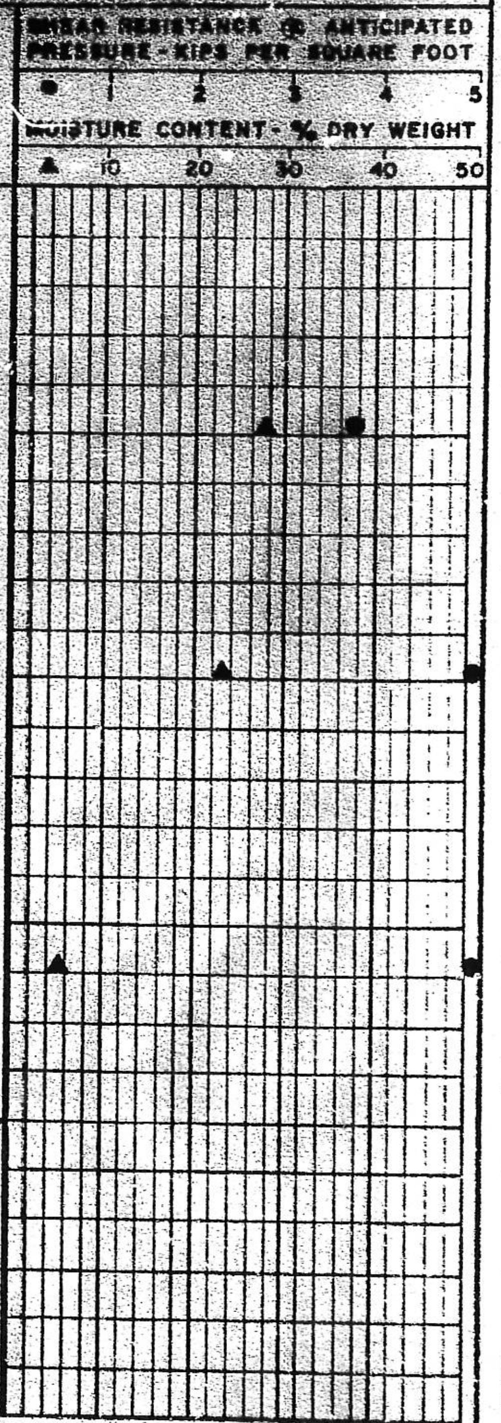
SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

11 206 1-71

[illegible] 18" dia. Bucket Auger

DRIVING WEIGHT [illegible] lb at 12" drop | SURFACE ELEVATION

[illegible]	[illegible]	[illegible]	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB PER CU FT
			SAND med. to coarse clean	lt. yel. brn.	sl. moist	comp.	
			SANDSTONE thinly bedded with sandy and clayey siltstone	olive gray	very moist to		
55	■	20		olive gray & rust	moist		96
60	■	25		blk. & olive gray			104
			med. to coarse clean with gravel	tan	sl. moist		
65							
	■	60					117
70			End of boring at 69.0 ft. **slight seepage - no water collected in hole during drilling				



Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	D

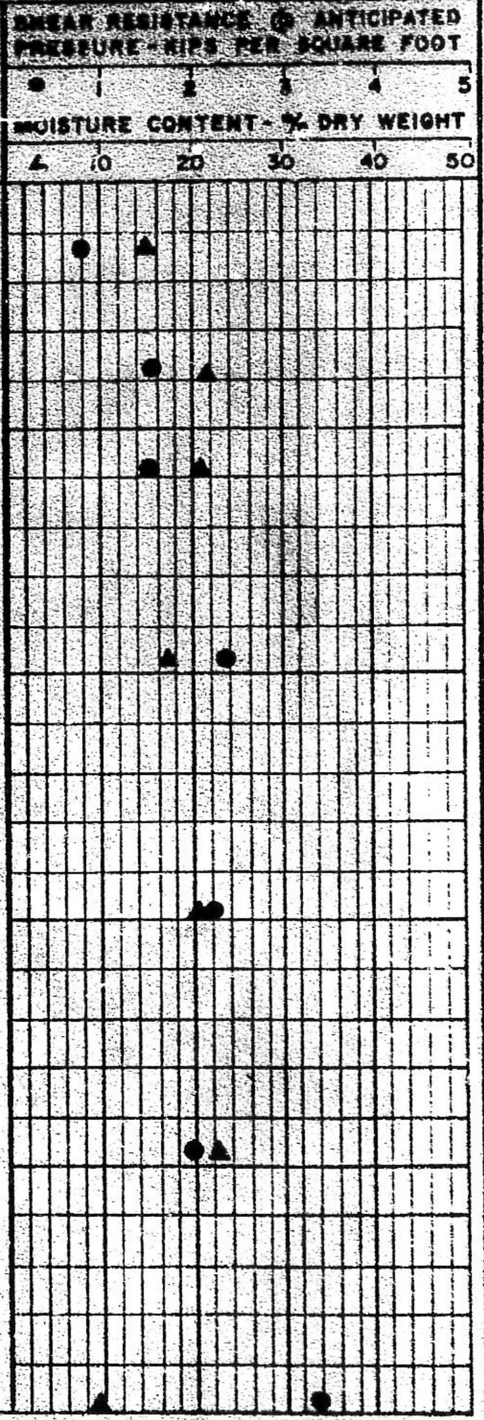
SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

SI 206 1-74

DATE DRILLED [illegible]

DRIVING WEIGHT [illegible] lbs. at 12" drop | SURFACE ELEVATION 113.5

Depth in Feet	Sample	Blows per foot	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB. PER CU.FT.
			2.5" asphalt				
	■	2	FILL CLAY very sandy silty, some slate fragments, mottled	dk. brn.	moist	firm	119
	■	P	sl. sandy		very moist		104
5	■	P	silty				107
10	■	2					110
			SAND fine to medium silty, clayey concrete frags. to 4"			mod. comp.	
			CLAY silty sl. porous	dk. yel. brn.		stiff	
15	■	P					105
				brn.		mod. stiff	
20	■	24					97
			SAND fine to medium clayey, w/gravel			comp.	
25	■	3	fine-coarse, silty w/slate, gravel	yel.br. gray	sl. moist		125



Continued

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	E

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

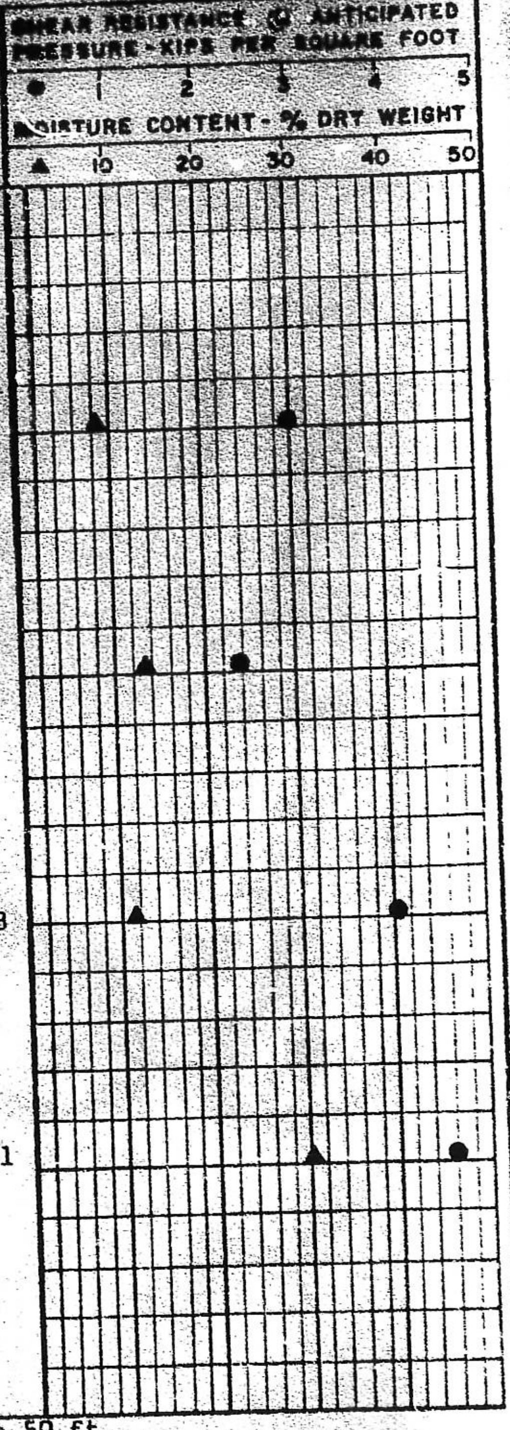
SI 206 1-71

[illegible]

DRIVING WEIGHT 1400 lbs. at 12" drop

SURFACE ELEVATION

Depth in Feet	[illegible]	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB. PER CU. FT.
		SAND fine-coarse, silty w/40% slate gravel	yel. brn. gray	sl. moist to moist	comp.	
		SLATE GRAVEL to 2" sandy, silty				
30	7					122
		with sand streaks				
35	8					122
		SAND fine-coarse sl. clayey w/45% gravel to 2"		seepage**		
		fine, sl. silty	yel. brn.			
40	16					108
			lt. olive gray			
		SILTSTONE clayey			firm	
45	8					91
		very sandy	blk.			
		CLAY silty	br.gry.		stiff	
50	12*					



Contd. * driving weight 850 lbs. - changed at 50 ft.

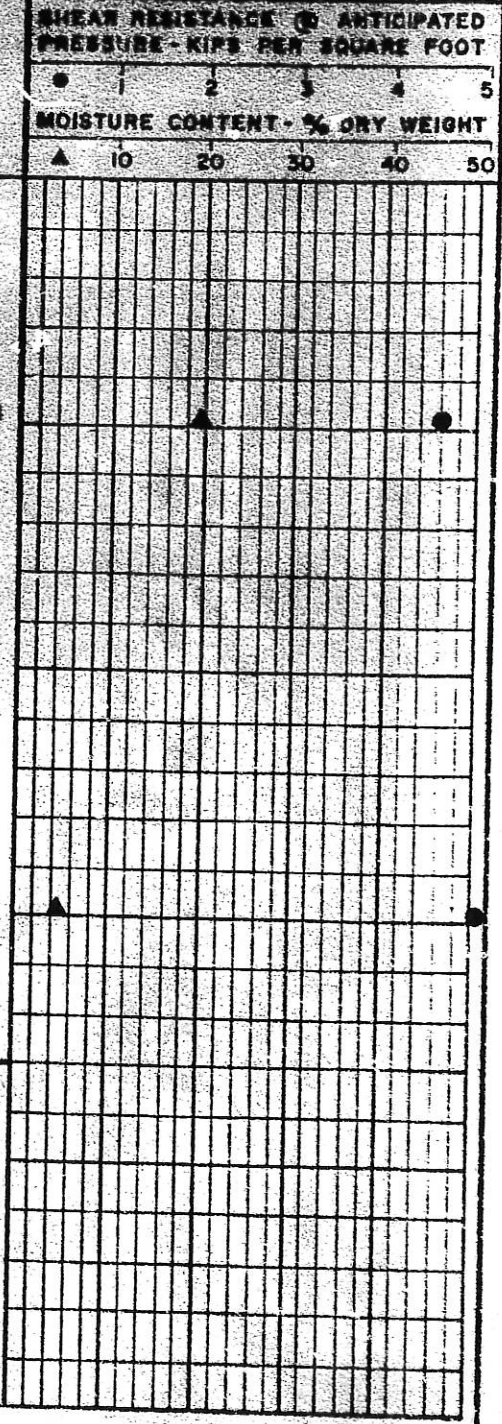
Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	F

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

[illegible]	DRILLING EQUIPMENT 18" dia. bucket auger
DRIVING WEIGHT [illegible] lbs. at 12" drop	SURFACE ELEVATION

Depth in Feet	Sample	Blows per Foot	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB. PER CU. FT.
			SANDSTONE w/alternate layers of SILTSTONE	tan & rust	very moist	comp.	
			fine to medium	pale olive gray			
55	■	33	fine, silty				104
60	■			dk. yel. brn.			
65	■		medium to coarse clean, 30% pea gravel to 40% cobbles to 4"	tan	sl. moist		116
70			End of boring at 68.0 ft. **Water seeping into hole from 38 ft. Only few inches of water visible at bottom of hole during drilling. After 30 min., 4" of water had collected in bottom of hole.				



Proposed Office Building for DMG, Inc
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	G

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

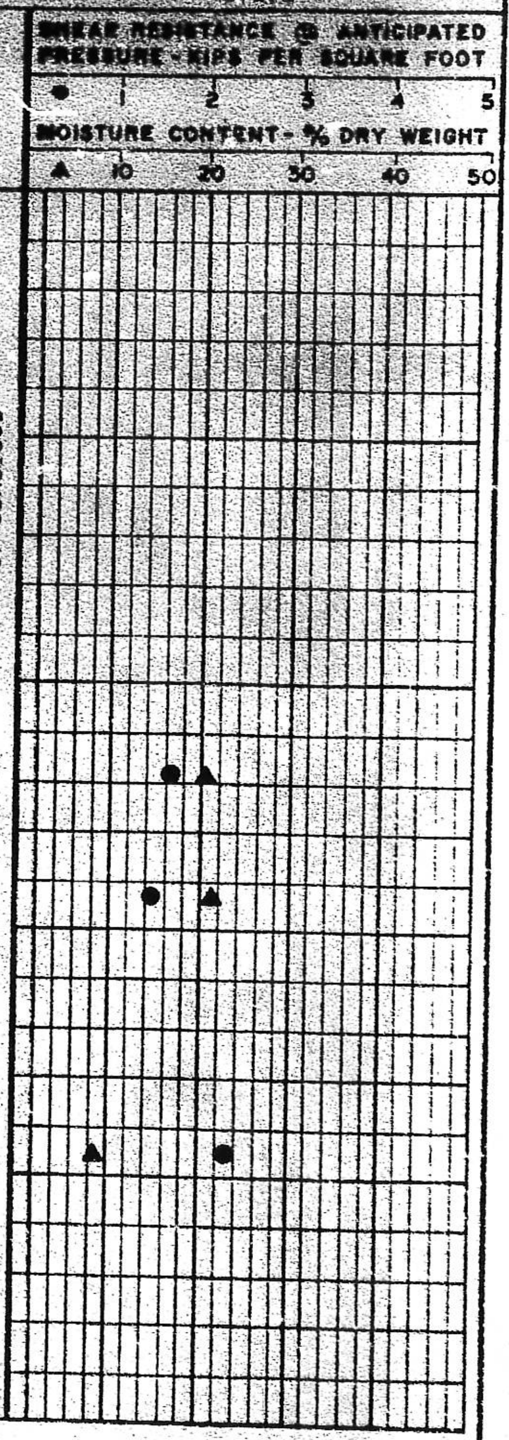
1 204 1-74

[illegible] EQUIPMENT 18" dia. bucket auger

DRIVING WEIGHT [illegible] lbs. at 12" drop | SURFACE ELEVATION 116.3

Depth in Feet	Sample	Blows per foot	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB. PER CU. FT.
			2.5" asphalt				
	■	P+	FILL CLAY very sandy silty some small gravel	dk. brn.	moist	stiff	
	■	P+					
5	■	3	SAND fine-coarse silty, with cobbles & gravels, broken brick	dk. gray to blk.		mod. loose	caving lge. pcs. of concrete
10			CLAY silty	dk. brn.	very moist	stiff	
	■	P+					106
15	■	P+				mod. stiff	102
						very stiff	
			SAND slate fragments		sl. moist		
20	■	2					120
			SLATE GRAVEL sandy clayey	dk. brn.-blk.		comp.	
25	■	3					

Continued



Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	H

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

[illegible]

DRIVING WEIGHT [illegible] 12" drop | SURFACE ELEVATION

SHEAR RESISTANCE @ ANTICIPATED PRESSURE-KIPS PER SQUARE FOOT
● 0 1 2 3 4 5
MOISTURE CONTENT - % DRY WEIGHT
▲ 10 20 30 40 50

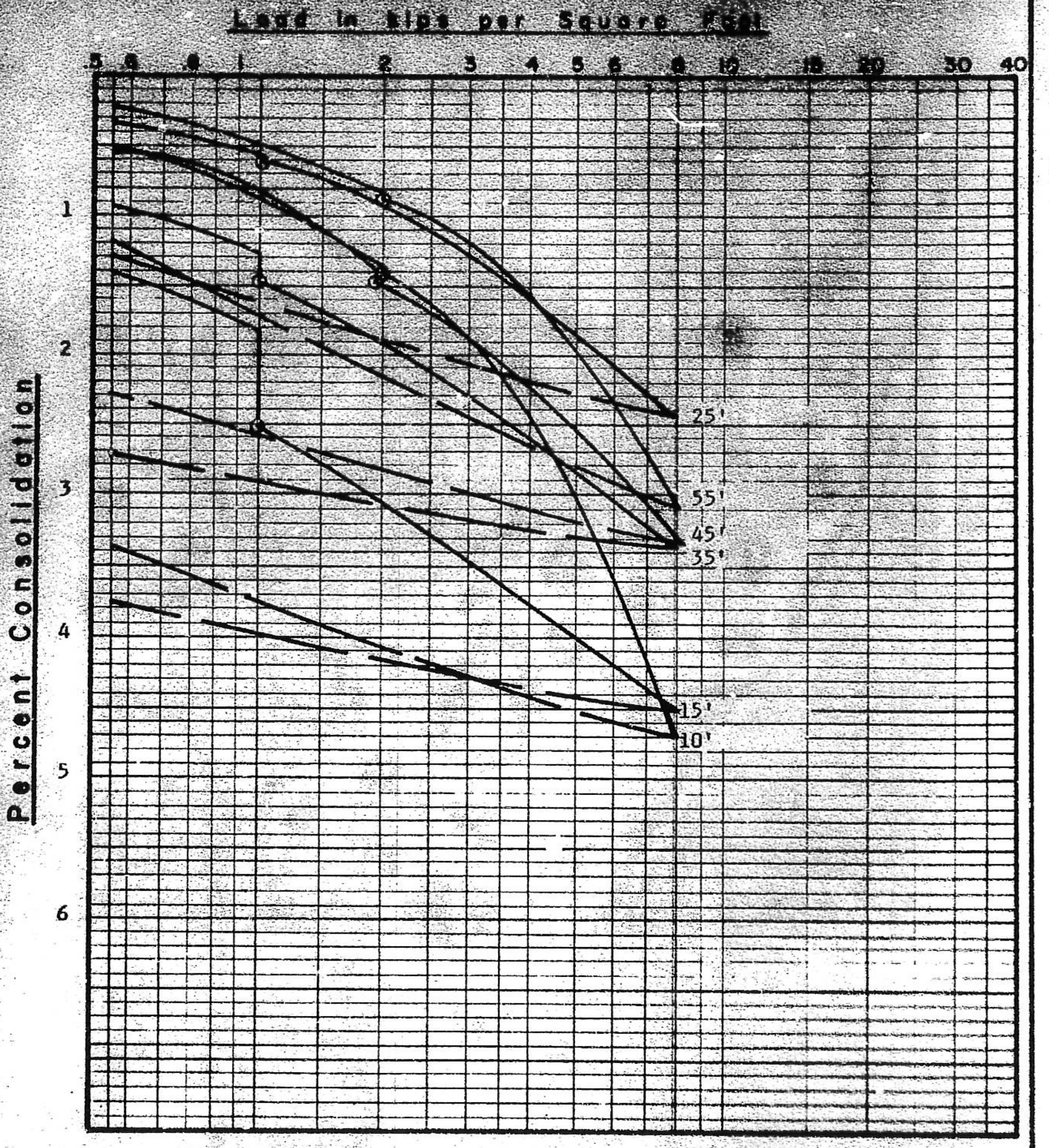
[illegible]	[illegible]	[illegible]	SOILS CLASSIFICATION	COLOR	MOISTURE	CONSISTENCY	DRY UNIT WEIGHT LB. PER CU. FT.
			SANDSTONE silty	lt. yel.	moist	comp.	
55	■	30	medium to coarse clean				
60	■	47	SAND fine, sl. silty to clean	pale gray	sl. moist		
			with gravel, clay streaks		moist		
65	■	39	coarse, clean with thin clayey silty layers				
			End of boring at 68.0 ft. No water encountered.				
70							

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	J

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

SI 206 1-74



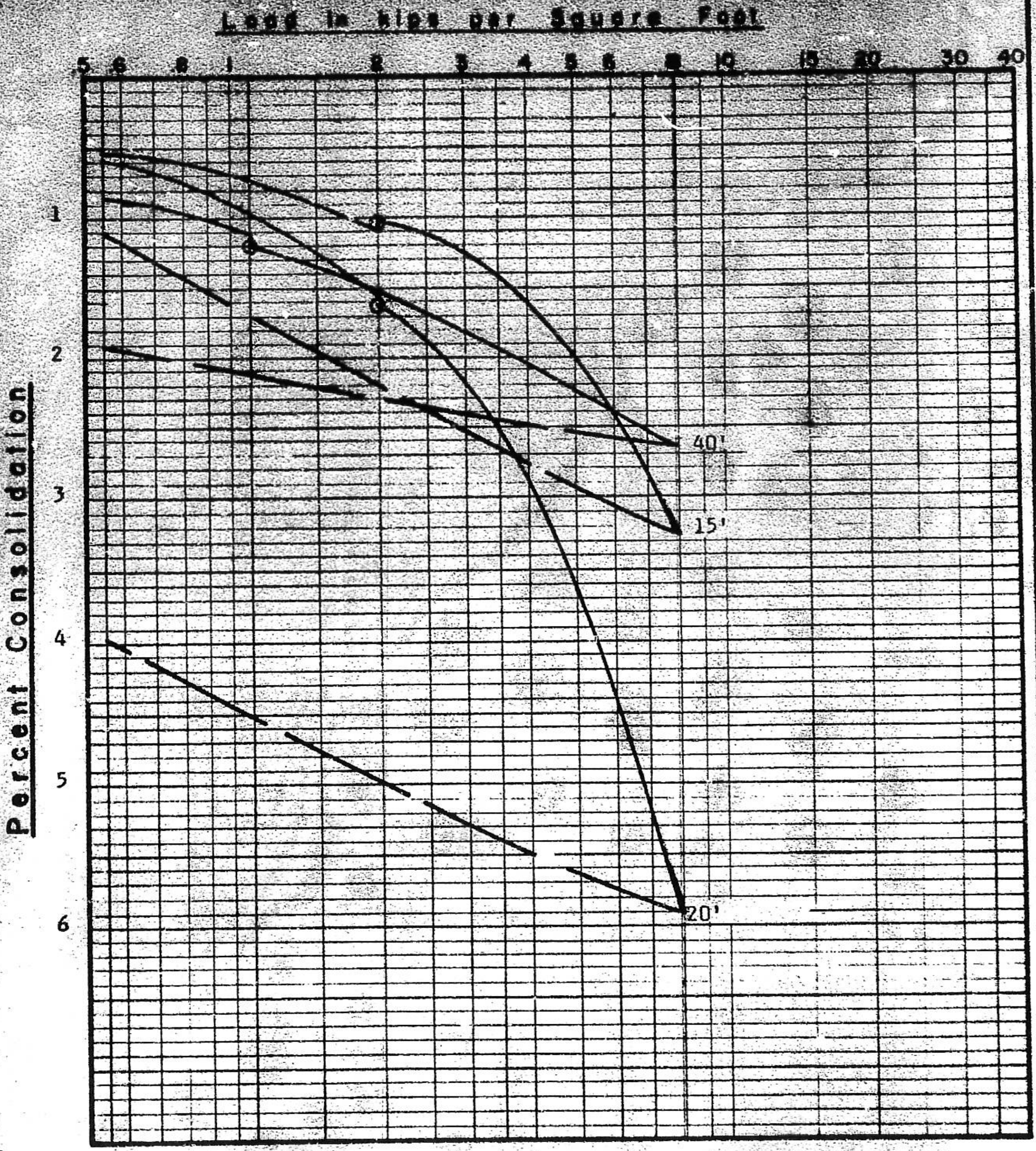
○ WATER PERMITTED TO CONTACT SAMPLE BORING NO.1

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	L-1176-F
	PLATE	K

SOILS INTERNATIONAL

CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

SI 204 1-74

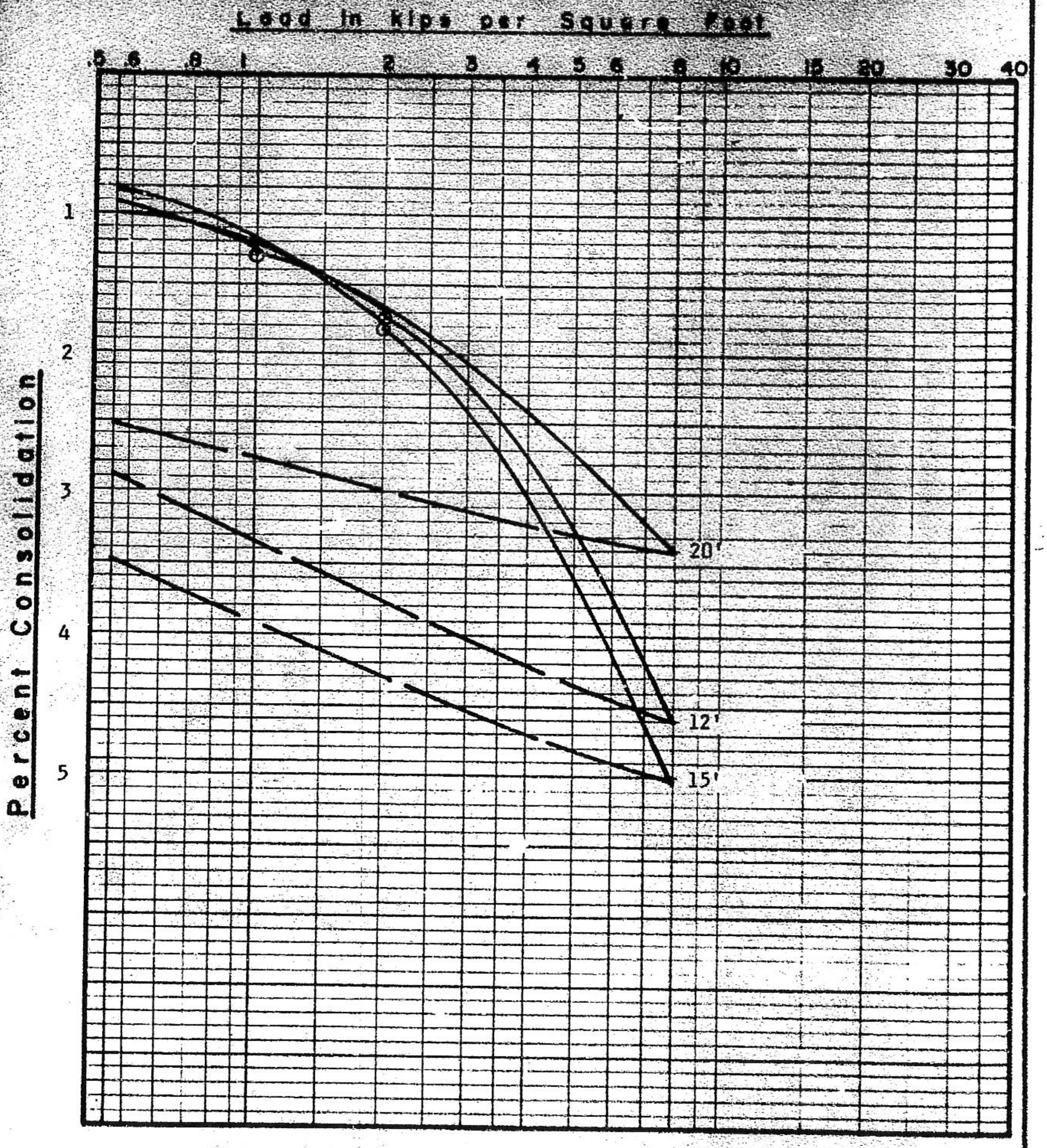


○ WATER PERMITTED TO CONTACT SAMPLE BORING NO. 2

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	L-1176-F
	PLATE	L

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

CONSOLIDATION TEST



○ WATER PERMITTED TO CONTACT SAMPLE

BORING NO.3

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	**PROJECT No.**	L-1176-F
	PLATE	M

SOILS INTERNATIONAL

CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

DAVID J. LEEDS AND ASSOCIATES

Consultants in Engineering Seismology/Geology/Geophysics

11972 Chalon Road
Los Angeles, CA 90049
(213) 472-0282

PART II

GEOLOGY AND SEISMICITY STUDY

GEOLOGY AND SEISMICITY STUDY

The site of the proposed project lies in the coastal plain in the Palms area of Los Angeles County, at the intersection of Sepulveda Boulevard and Rose Street (Fig.1).

SCOPE OF THIS REPORT

This report develops recommended seismic ground motion levels for design. It follows the format of the Structural Engineers' Association of California, "Suggested Procedures for Developing Seismic Ground Motions", Second Edition, 1979. The report contains a geological and seismological study, including a discussion of faults; a discussion of the recurrence rates of earthquakes; and the ground motion characteristics.

1-A Geologic and Seismologic Information

Available geologic and seismologic information have been reviewed. Pertinent publications of direct interest to the site are given in the accompanying figures or listed in the References.

1-B Geology

The site is located in the western part of the coastal plain in the Los Angeles basin, two miles northwest of the Baldwin Hills. Several geologic maps (Figures 2 through 7) show regional and local geology; Figures 8 through 11 show faults and earthquake epicenters.

The Los Angeles basin is bounded by the Peninsular Range Province on the east and southeast and by the Transverse Range on the north and northwest. Parts of the basin extend southward and are submerged by the Pacific Ocean (Figures 2,3).

The Peninsular Range Province is a northwest-trending topographic and structural high that is transected by the east-trending Transverse Range Province. The rocks of the Peninsular Range are mainly igneous, meta-sedimentary and meta-volcanic of Paleozoic and Mesozoic age with some local occurrences of Cenozoic sedimenatry and volcanic rocks. Rocks of the Transverse Range are igneous, meta-igneous, and meta-sedimentary of Precambrian to Tertiary age. These rocks are extensively folded and faulted. The Los Angeles basin is a down-warped area filled with detritus from these mountain ranges. The sedimentary rocks in the basin range in age from Upper Cretaceous to Holocene. A continuous record of deposition is not present, either because of erosion or non-deposition throughout the history of the basins development. The rocks of the basin have been folded and faulted.

The sedimentary rocks of the basin have been faulted and folded; the trends of these structures are northwesterly. The deformation has been sporadically active since the Cenozoic and has continued into the present. Oil and gas has accumulated along some of the structural trends.

The types of faults include the complete range of faults; strike-slip (both left-lateral and right-lateral), reverse or thrust, normal, and combined strike-slip and reverse or strike-slip and normal. The major faults in the region that are of interest to the site include the San Andreas, the San Fernando, the Newport-Inglewood and the Santa Monica-Hollywood. These will be discussed below.

1-C Geologic Site Reconnaissance

The site is located along and below a west-facing flank of a Pleistocene Terrace composed of the Upper Pleistocene Palos Verdes sand (Figure 5). The Palos Verdes sand is exposed in a cut in the shopping center on the northwest corner of the intersection of Sepulveda Boulevard and Rose Street. The rock is composed of friable, tan to reddish-brown, interbedded medium to coarse sand with lenses of conglomerate. Figure 6 is a northeast-southwest cross section, about 2 miles southwest of the site.

In the area of the site, the upper surface of this terrace has been eroded by a southeast flowing tributary to Ballona Creek. Locally, this tributary has deposited recent alluvium. Artificial fill has been placed on the site.

Three holes were drilled on the site as part of the soil investigation. The logs of these borings appear in the Soil Report in the front part of this combined report. Examination of the samples indicate that the artificial fill ranges in thickness from about 8½ feet to 13 feet. The fill is underlain by a dark gray clay that ranges in thickness from about 3½ feet to 9½ feet. Underlying this clay is a zone of sandstones and conglomerates, 13 to 19½ feet thick. The pebble-sized angular fragments in the conglomerate are predominantly slate and were probably derived from the Santa Monica slate in the Santa Monica Mountains to the northwest. The clay is interpreted as a residual, weathered soil of the sandstone and conglomerate sequence; the lithology is similar to the Holocene "50-foot gravel" described by Poland, et al (1959). The "50-foot gravel" has been interpreted by them to be alluvial deposits formed by the ancestral Los Angeles River. Underlying the "50-foot gravel" is the Pleistocene San Pedro Formation. The minimum thickness of this unit at the site is 38 feet, based on the boreholes.

1-D Potential for Site Ground Failure

Because of the cohesive nature of the near-surface soils and the absence of groundwater in the upper 38 to 47 feet, the likelihood of liquefaction is considered remote. Because no faults are known to exist on or beneath the site, the probability of surface rupture from fault activity is considered remote.

1-E Local and Regional Faults

Approximately 75 faults are known within a radius of 60 miles (150 km) of the site that can be considered active or potentially active. The overwhelming evidence of the activity and threat of several of these faults precludes consideration of the entire suite. The characteristics of the faults significant to the project are discussed briefly in section 1-G. Two faults occur close to the site: The Charnock Fault is 0.5 miles to the southwest and the Overland Fault is 0.75 miles to the northeast (Figures 4,5). These faults bound a northwest trending graben. Neither of these faults show evidence of surface displacement (Poland, et al, 1959, p. 78; Ziony, et al, 1974; Figure 7 this report) and the "50-foot gravel" is not known to be cut by either of these faults (ibid, p.78). However, these faults do offset the Pleistocene San Pedro Formation. The vertical displacement on the Overland Fault is about 30 feet, down on the west; vertical displacement on the Charnock Fault is about 140 feet, down on the east (Figure 6). Ziony, et al, 1974 (Figure 7) indicate that Holocene rocks have not been faulted or disturbed by these faults, but that late Quaternary rocks older than Holocene have been faulted. Minor earthquakes (Richter Magnitude <4) have been detected in the general area (Real, et al, 1978; Buika and Teng, 1978; Figures 10 and 11, this report). None is known to have occurred on either the Charnock or Overland faults. Based on the available evidence--absence of surface displacement, no offset of the "50-foot gravel", and lack of evidence of earthquake occurrence on either fault--the Charnock and Overland faults are not considered to be active faults; however, they are considered to be potentially active.

The locations of most of the active and potentially active faults in the area are shown on Figures 2, 3, 4 and 7. Figure 8 indicates the maximum credible rock acceleration that might be expected from the active faults. Figure 9 associates historical earthquakes in the area with these known active faults.

1-F Active Faults Traversing Site

No known active fault is believed to traverse the site or is known to be within 1.5 miles of the site.

1-G Capable Faults

The seismogenic faults of most significance to the project are:

San Andreas Fault. The closest approach of this fault to the site is 40 miles. Activity on this fault is considered "imminent" by many seismologists. A magnitude 8+ event in the sector from Gorman to San Bernardino is considered a distinct possibility, and in the early part of the project's lifetime. Since the maximum event has such a high probability of occurring, consideration of smaller events on this fault is moot. This is a strike-slip fault whose motion, with proper scaling, can be modeled by the 1952 Kern County (Taft N21E) accelerogram. The San Andreas Fault is considered the control event for purposes of design for the maximum probable earthquake.

<u>San Fernando Valley Faults</u>. This fault system is 20 miles from the site and has been historically active with the most recent event occurring in 1971. At that time, the site was exposed to shaking of about 0.1g.

<u>Newport-Inglewood Fault Zone</u>. The closest approach of this fault zone to the site is 2.4 miles to the northeast. However, this segment of the fault zone is near its northern terminus. Consequently, the likelihood of the maximum credible earthquake occurring in this area is considered to be very low. The maximum probable earthquake is expected to occur about 20 miles from site, and such an event is considered highly probable. This fault zone has been historically active with the 1933 Long Beach earthquake (Richter Magnitude, M=6.3) having occurred in this fault zone, about 30 miles southeast of the site. In June, 1920 the Inglewood earthquake (M 5) occurred in this fault zone, about 5 miles southeast of the site.

<u>Santa Monica-Hollywood Fault System</u>. The closest approach of this fault system is 1.5 miles to the northwest. Even though the likelihood of a moderate earthquake (M=5.5) is considered low, the system is seismically active (Figures 10 and 11). Because of the seismic activity and the closeness of the fault to the site, the Santa Monica-Hollywood Fault System is considered to provide the control earthquake for purposes of design for the maximum credible earthquake.

Charnock and Overland Faults. Although these faults are considered potentially active and are closer to the site than the Santa Monica-Hollywood Fault System, they are not considered to present a serious threat to the site for reasons given above. Based on the evidence to date, seismic design of the project must be based on other, more obvious threats.

1-H Subsoil Conditions

Refer to the soils investigation for a detailed description of subsurface soil conditions.

RECURRENCE

2-A Average Recurrence Rates

Average recurrence rates are derived from the historical record of earthquakes. Figure 10 plots earthquake epicenters in the area for a 75-year period having magnitudes of 4.0 or greater. Figure 11 shows the local area in more detail, with all instrumental events plotted, but for a shorter timespan (1973-1976). Southern California seismicity may also be represented by listings of earthquakes, such as Tables I and II. The data are expressed in terms of recurrence in Figure 12, in which comparison of local seismicity with regional seismicity is possible. The scarcity of data because of the short timespan does not warrant further breakdown of specific recurrence rates for each separate fault. Extension of the timeframe, both backward and to date, would very slightly raise the recurrence rate for all of Southern California and raise the Los Angeles Basin line to where it would practically overlap the Long Beach Harbor rate. The slightly higher levels for both the 25-mile radius and the 100-mile radius areas reflect inclusion of events with recurrences greater than the timespan of data used.

2-B <u>Probability Models</u>

While the average recurrences on the basis of the data presented are shown in Figure 12, little quantitative information can be derived from the data. On that basis, plus a knowledge of the geologic and seismologic regime, the best statement that can be made is that there is a high probability of the site experiencing significant ground motion in the project lifetime. Reference to the short-term data above yields a probability of only a few percent in a given year. The effects of the threats mentioned would be mitigated by a design accepting the Santa Monica-Hollywood ground motion levels at the site.

A general statement for ground motion in Southern California has been published (Figure 13). Although not specifically applicable to the project, this generalization is reasonable and reflects exposure of the project to seismic ground motion.

GROUND MOTION CHARACTERISTICS

3-A General Development

Site ground motion is developed by a multistep method:

a. Definition of seismic zones, faults, or source areas
b. Development or selection of site or region specific attenuation characteristics
c. Calculation of site ground motions (acceleration, velocity, displacement, and duration)
d. Selection of analogous time-histories and response spectra, along with their scale factors

The capable seismogenic faults have been defined in Section 1-G.

Attenuation is taken from the date presented in Figures 1A through 16.

Representative ground motions are given in Table III.

3-B <u>Empirical Procedure - Recommended Ground Motions</u>
It is the opinion of the writer that an appropriate model of ground motion for the site can be based on Newmark-Hall procedures. A convenient source for this purpose is available in ATC-2 materials. The response spectrum for the site (Figures 17a, b) has been adjusted to 0.4g for the maximum credible earthquake and 0.2g for the maximum probable earthquake, using the control earthquakes on the Santa Monica-Hollywood Fault System and the San Andreas Fault, respectively. The response spectra are felt to encompass a ground motion with 50% probability of occurrence in a 50-year life.

While we do not normally accept the Newmark-Hall response spectrum envelope for all projects, it is felt that the model adequately encompasses both near-field and far-field spectral characteristics of motion, and that the site itself is somewhat better than the "average" conditions used in the input to the Newmark-Hall model.

As a "real time" alternative to the response spectrum, two time-history accelerograms are suggested:

a. Kern County Earthquake of 1952, at Taft, N21E component, scaled to 0.4g and 0.2b for the maximum credible and maximum probable earthquake, respectively. This is the only available time-history of an actual large earthquake. See Figures 18 and 19 for the time-history and response spectrum.

REFERENCES

Barrows, A. G., 1974, A review of the geology and earthquake history of the Newport-Inglewood structural zone, southern California: California Division of Mines and Geology Special Report 114, 115p

Buika, J. A. and Teng, T., 1978, Recent seismicity and fault-plane solutions of the Santa Monica Mountains, Santa Monica Bay, and northern Los Angeles <u>in</u> Geol. Guide and Engineering Geology Case Histories, Los Angeles Metropolitan Area, Assoc. Engineering Geologists, 1st Annual California Section Conference, May 12-14,1978, p. 64-72.

Castle, R. O., 1959, Geologic Map of the Beverly Hills and Venice Quadrangles, Surficial Geology: U. S. Geological Survey Open File Report

City of Los Angeles Department of Building and Safety (1977), "Rule of General Application RGA-1-76" Los Angeles

Duke, C. M. and D. J. Leeds (1971), "Site Characteristics of Southern California Strong Motion Earthquake Stations," California Division of Mines Special Publication 38

Earthquake Engineering Research Laboratory (1972-1976), "Analyses of Strong Motion Earthquake Accelerograms," California Institute of Technology, Pasadena

Hill, Robert L. et al (1979-1980), "Earthquake Hazards Associated with Faults in the Greater Los Angeles Metropolitan Area...," California Division of Mines & Geology Open File Report 79-16 LA and 80-1 LA

Jahns, R. H., 1954, Investigations and problems of southern California geology, in Geology f Southern California, California Division of Mines and Geology, Bulletin 170, v.1, p. 5-29

National Bureau of Standards (1973), "Building Practices for Disaster Mitigation," Building Science Series 46. Includes papers by Algermissen and Donovan

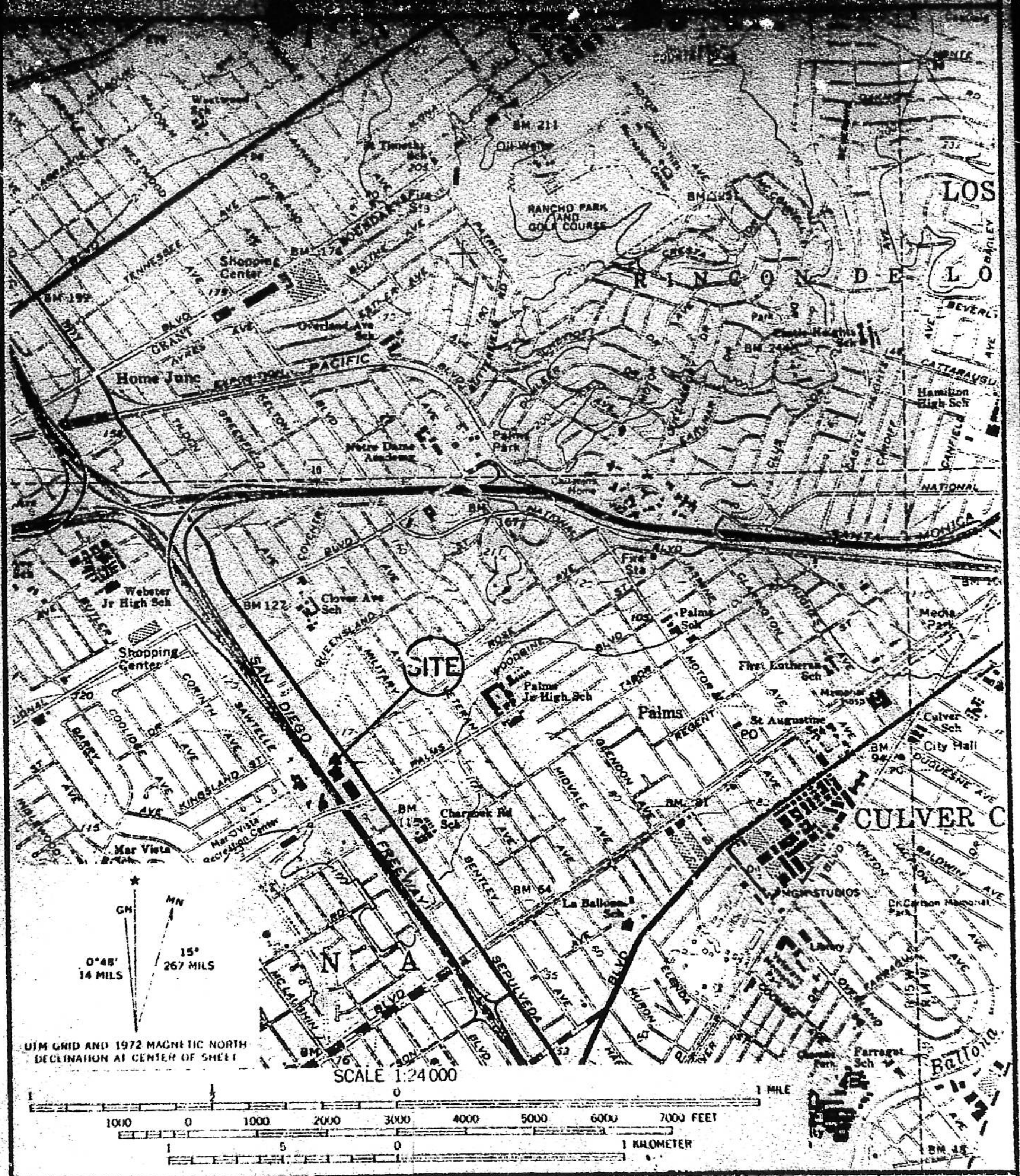
Poland, J. F., et al, 1959, Geology, hydrology, and chemical character of groundwaters in the Torrance-Santa Monica area, California: U. S. Geological Survey Water Supply Paper 1461, 425p

References-Contd.

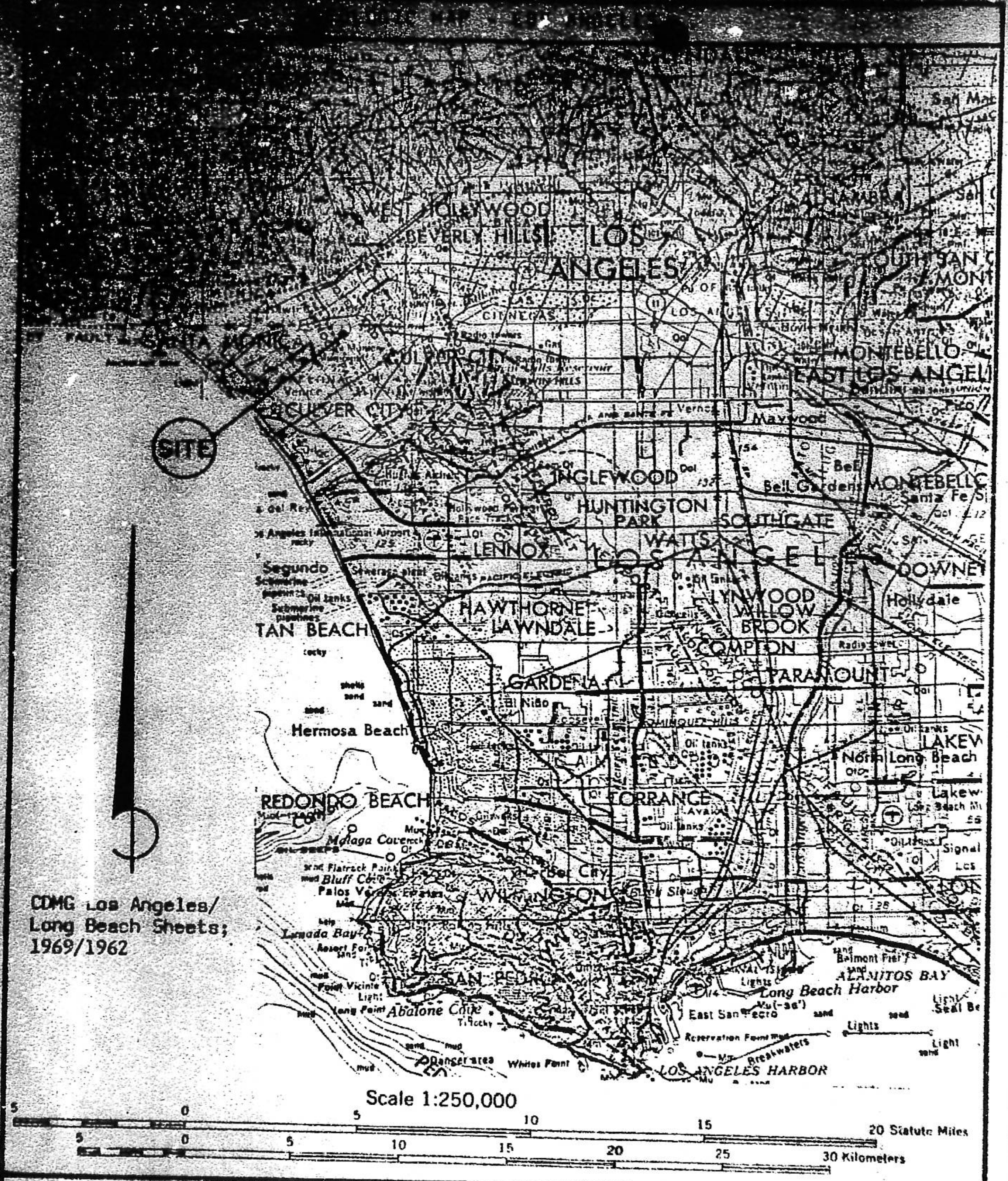
Structural Engineers of California (1979), "Suggested Procedures for Developing Seismic Ground Motions, San Francisco. (Including all references contained therein.)

Yerkes, R. F., et al, 1965, Geology of the Los Angeles Basin, California - an introduction: U. S. Geological Survey Professional Paper 420-A, 57p

Ziony, J. I., et al, 1974, Preliminary map showing recency of faulting in coastal Southern California: U. S. Geological Survey Map MF-585

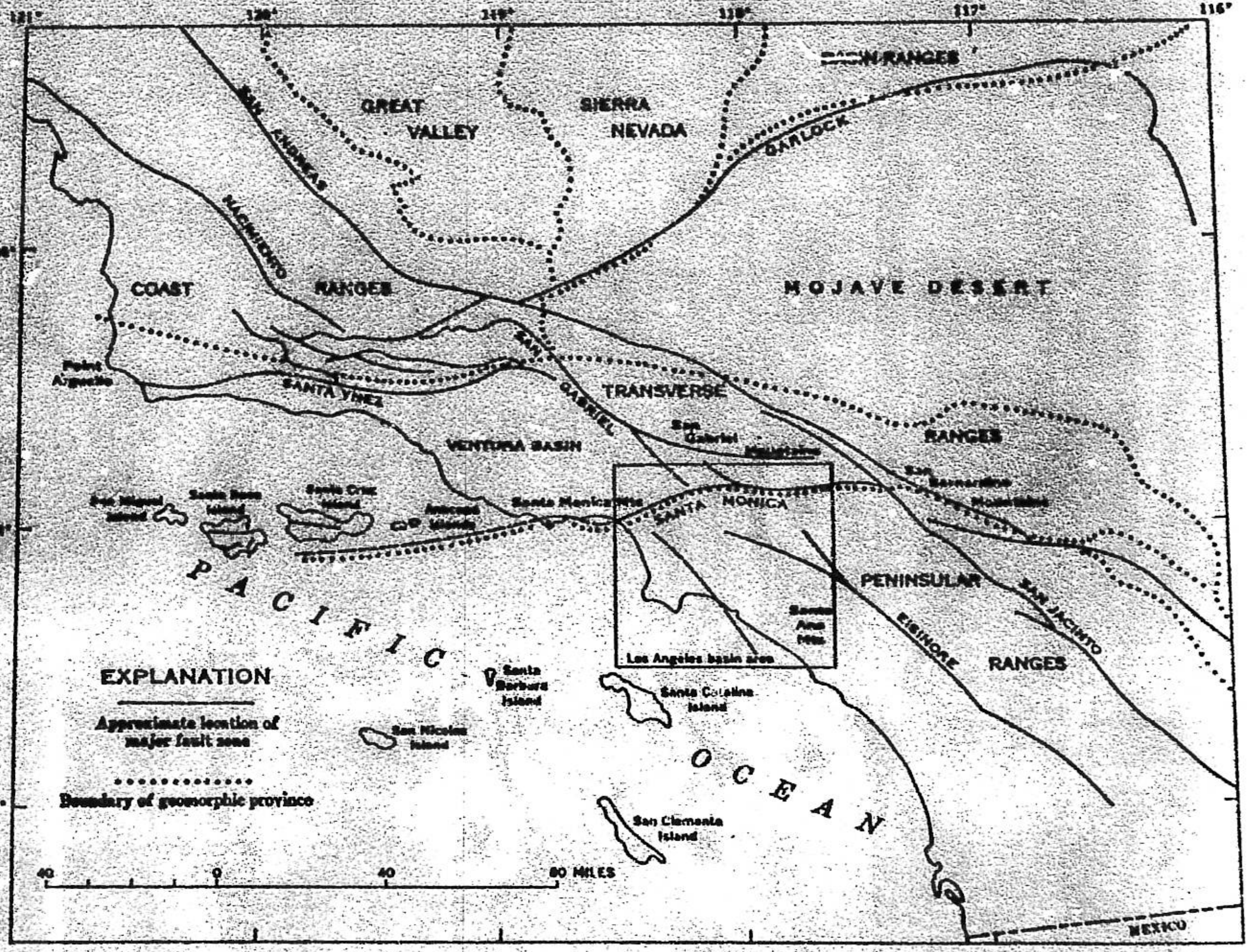


Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		1



CDMG Los Angeles/
Long Beach Sheets;
1969/1962

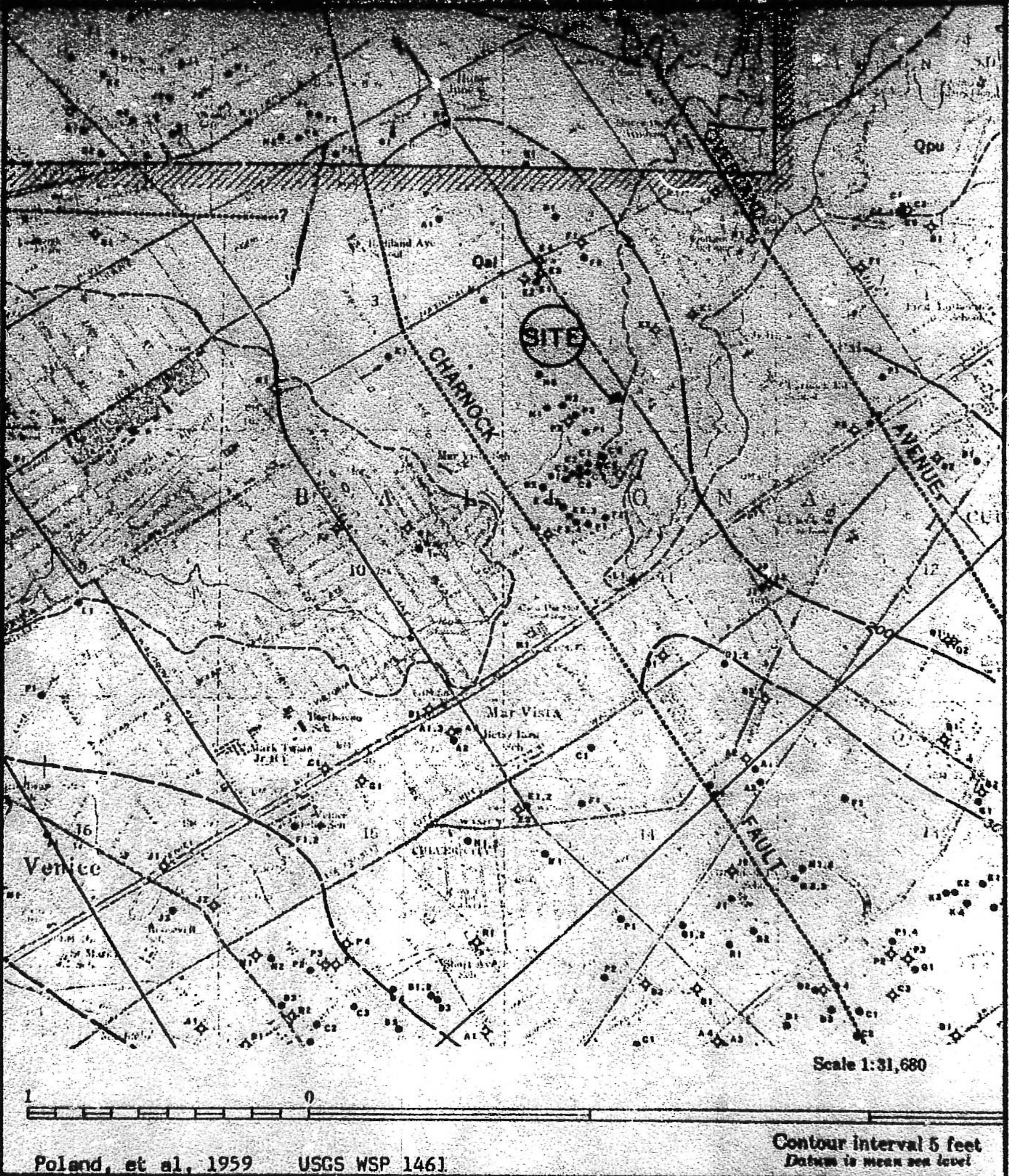
Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		2



Yerkes, 1965

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	
	3



Poland, et al, 1959 USGS WSP 1461

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	
	5a

EXPLANATION

Recent

Qal

Alluvial and coastal deposits

Sand, gravel, and silt in stream channels and beneath flood plains; silt, clay, sand, and gravel along and near the coast. Coarser materials highly permeable to water

Qs

Dune sand

In places underlain by Pleistocene beach deposits

UNCONFORMITY

Upper Pleistocene

Qpu

Terrace cover

Reddish-brown sand, nonfossiliferous and probably largely continental

Palos Verdes sand

Sand and some pebble gravel; fossiliferous and marine; moderately permeable but largely above water table; crops out only locally at edges of mesas, in road cuts, and in gravel pits

Unnamed deposits

Silt, sand, and gravel; moderately permeable to water; inferred to crop out only locally at edge of mesas

LOCAL UNCONFORMITY

Lower Pleistocene

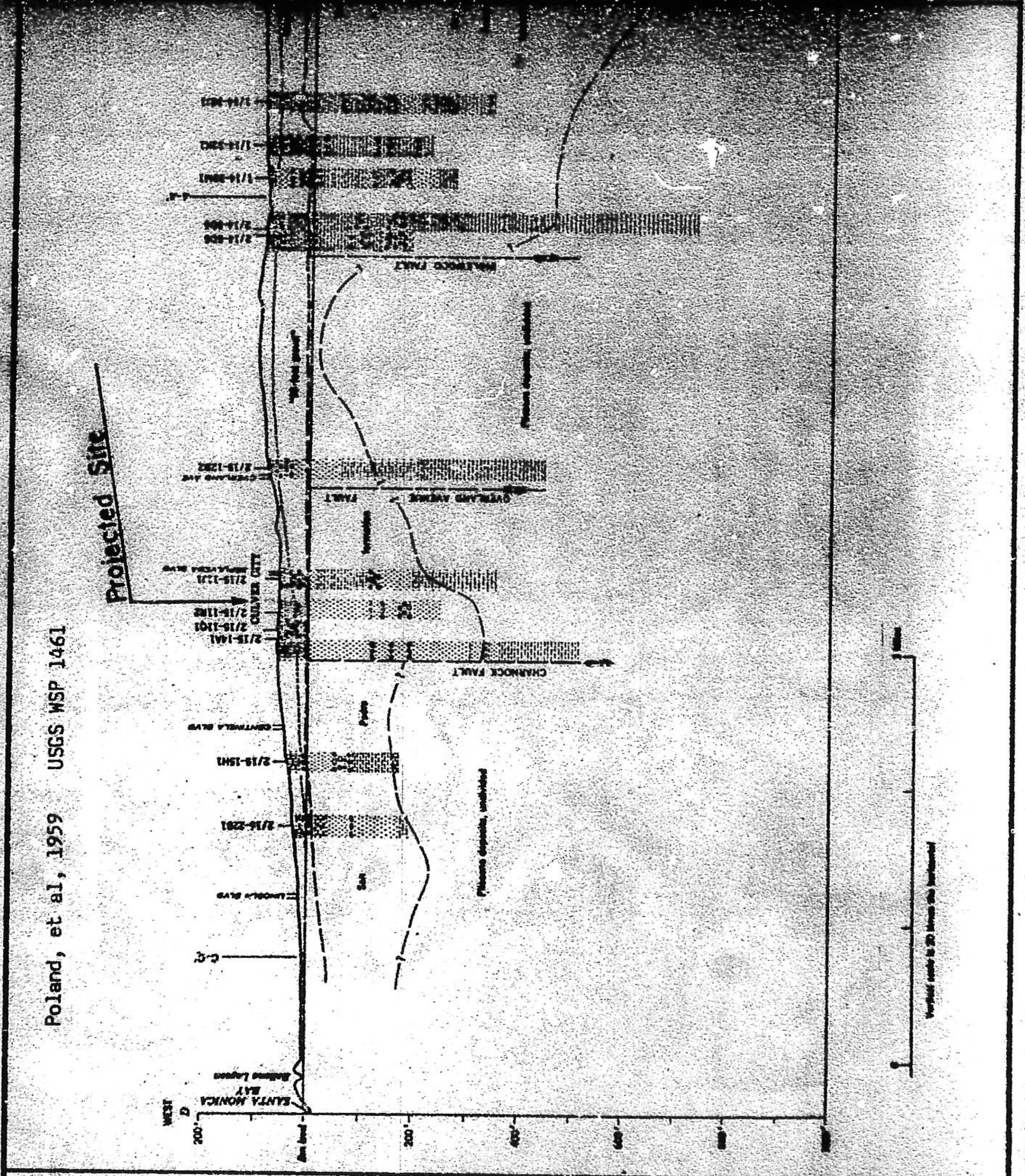
Qsp

San Pedro formation

Sand, gravel, silt, and clay; includes Timms Point silt and Lomita marl members at base. Thick bodies of sand and gravel yield water copiously to many wells

QUATERNARY

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		5b

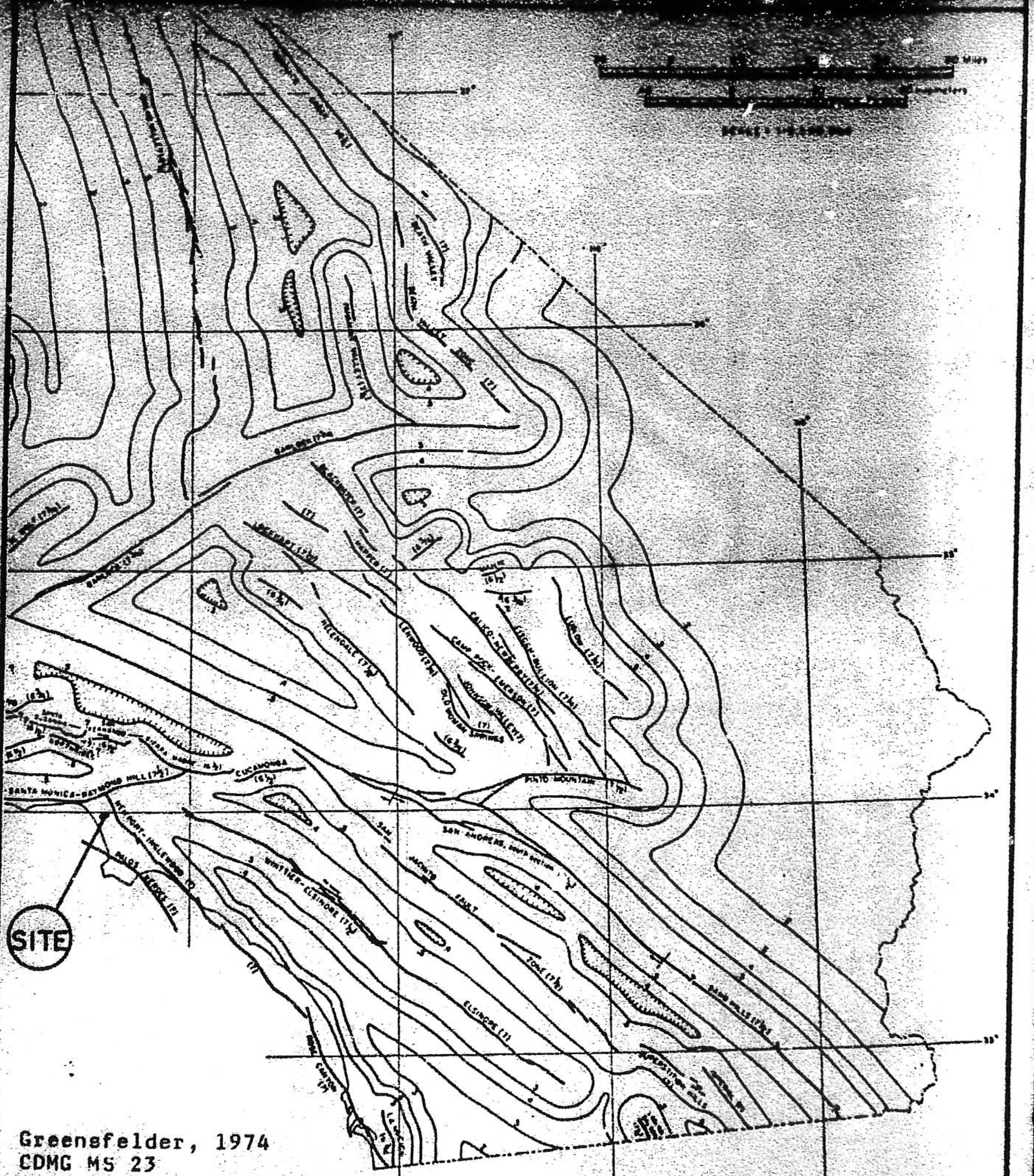


0 8 9 0 1 2 0 0 1 0 3

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.	
	6

S.F. 208 1-74

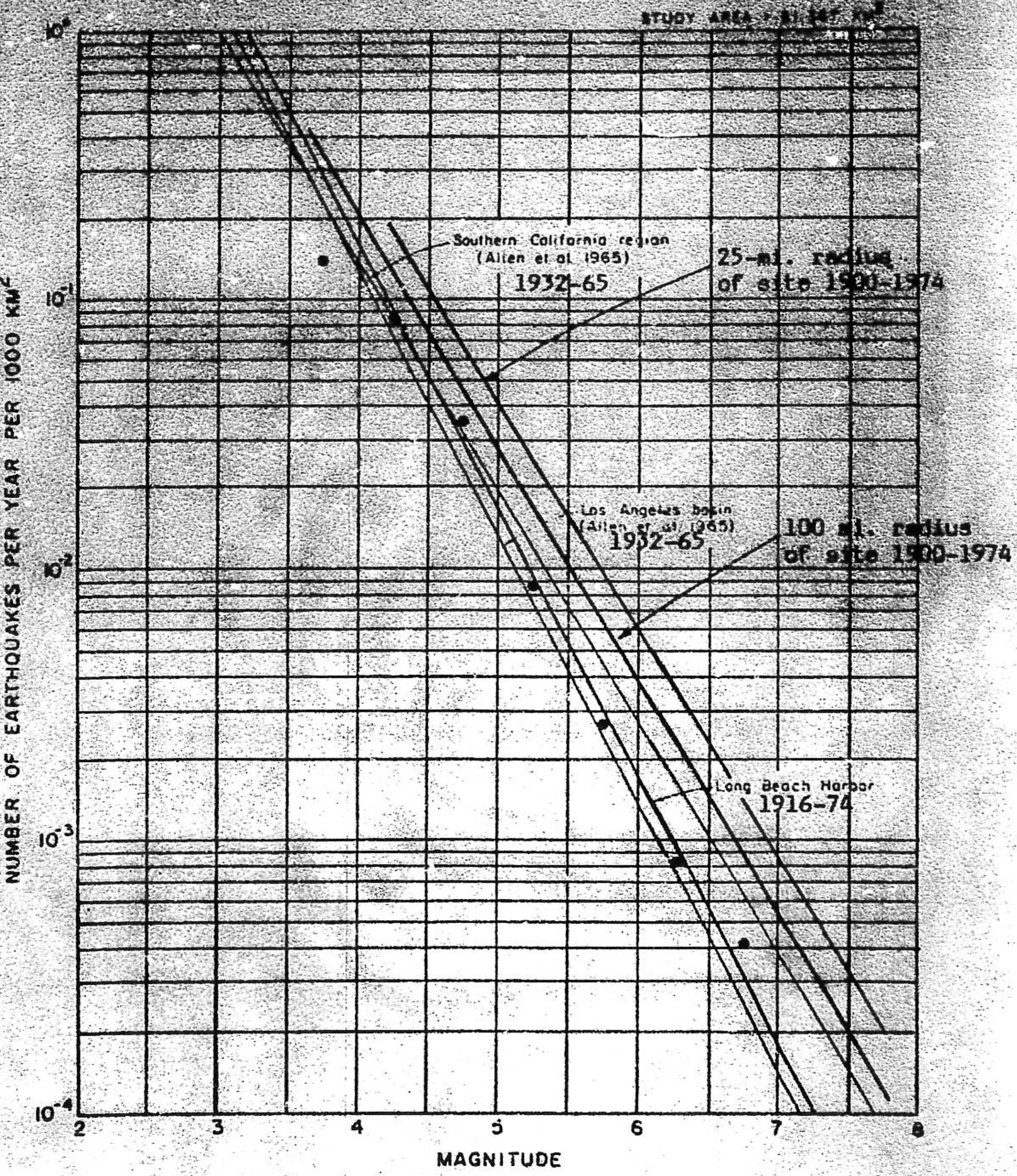


Greensfelder, 1974
CDMG MS 23

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

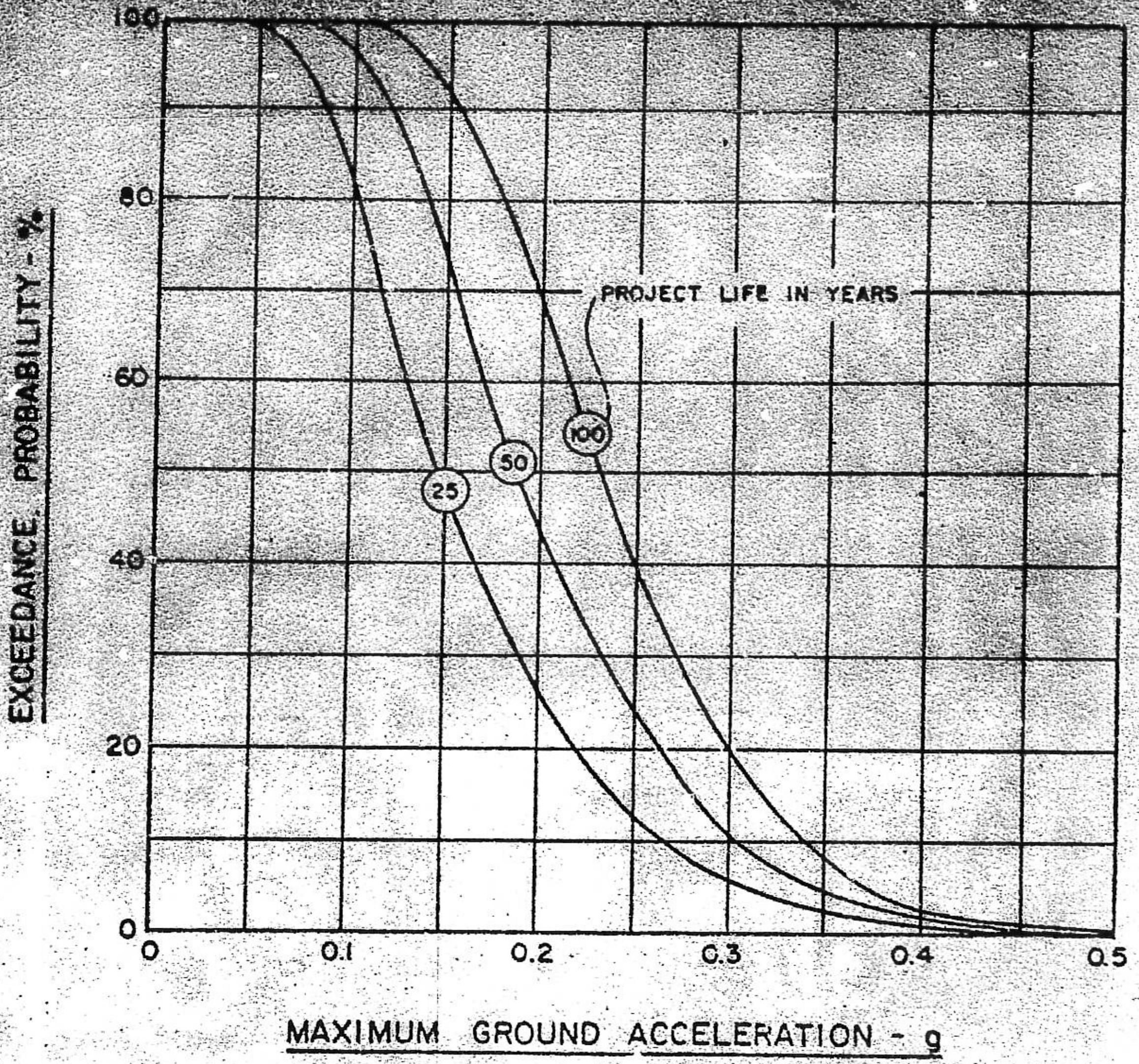
PROJECT No.	
	8

I. 208 1-74



Reference: Port of Long Beach
Master Environmental Setting, I, 1976

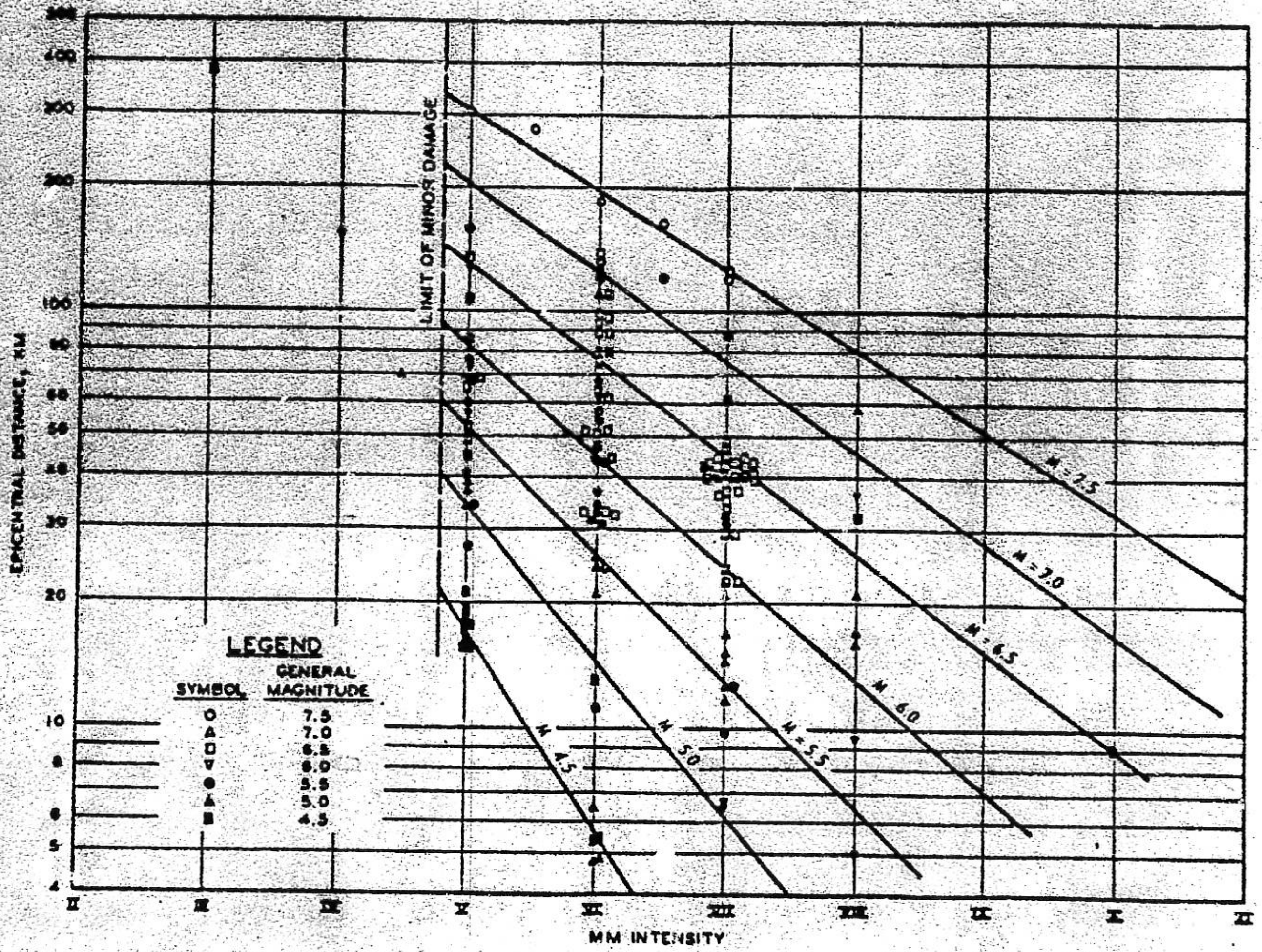
Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		12



EXCEEDANCE PROBABILITY OF GROUND SURFACE ACCELERATIONS

Port of Long Beach, Master Environmental Setting I, 1976

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		13



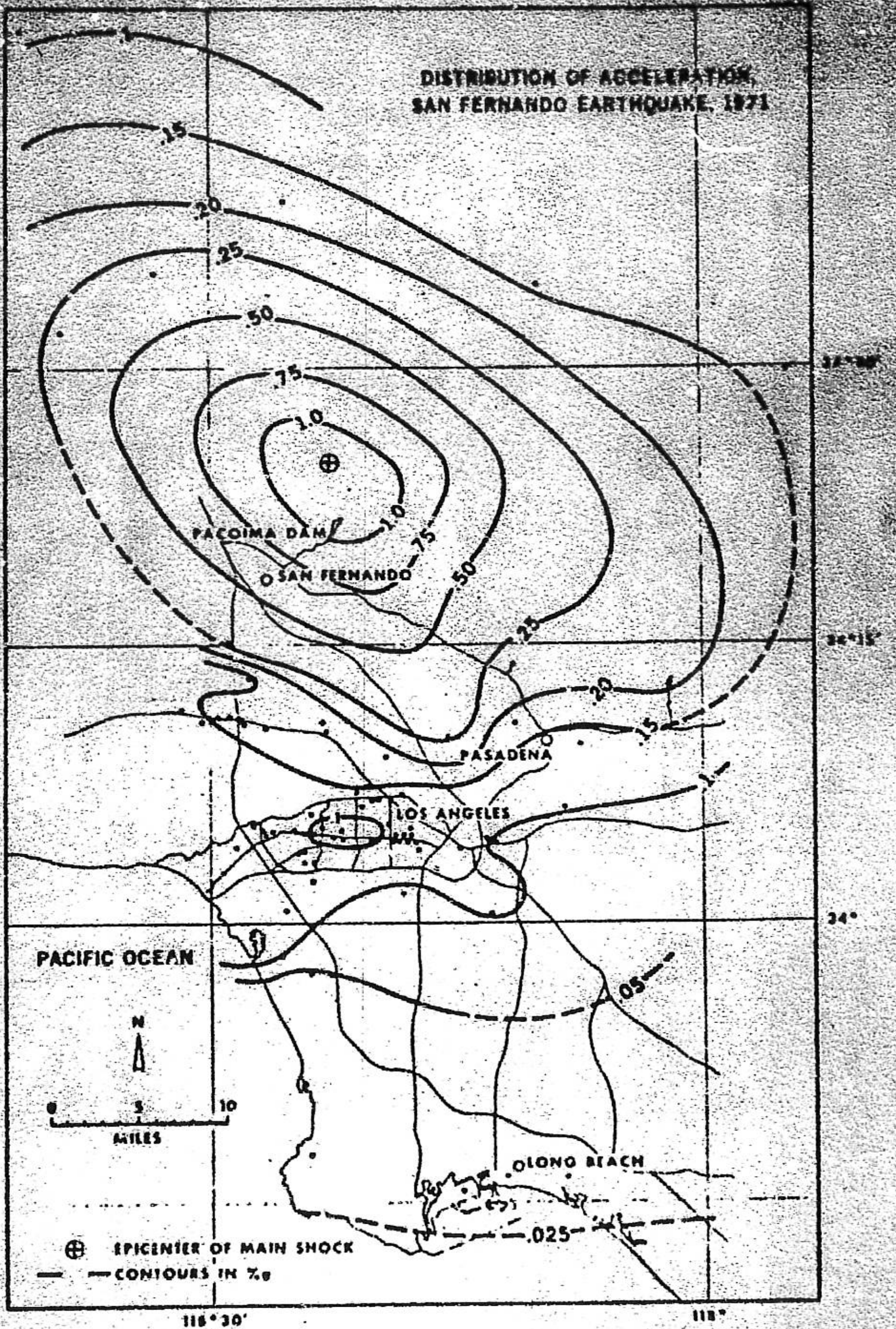
Intensity versus magnitude and epicentral distance, western United States

Krinitzsky & Chang, 1975
WES: MP S-73-1, Rpt 4

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.

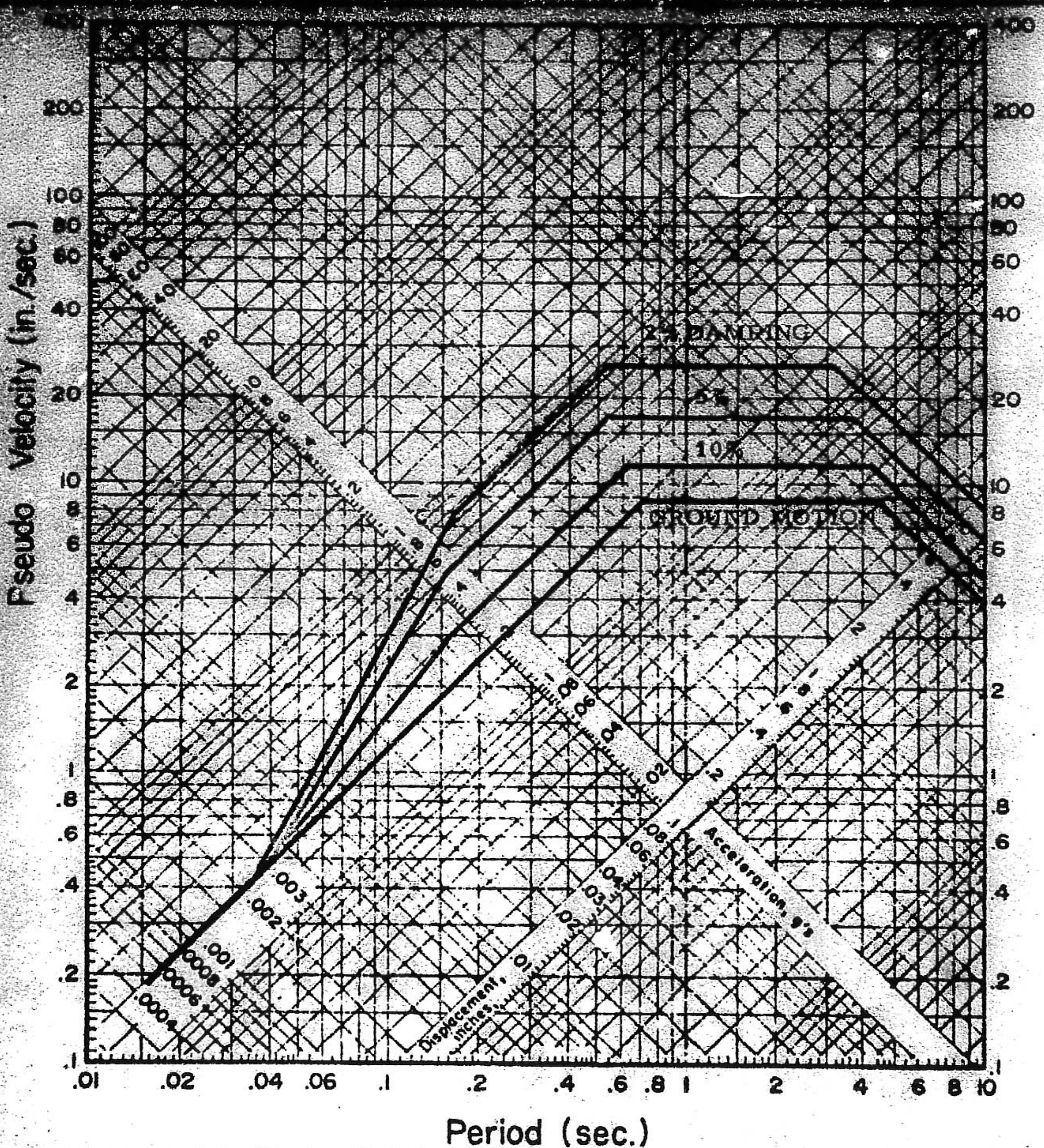
14



APPROXIMATE DISTRIBUTION OF MAXIMUM HORIZONTAL ACCELERATION DURING THE 1971 SAN FERNANDO EARTHQUAKE

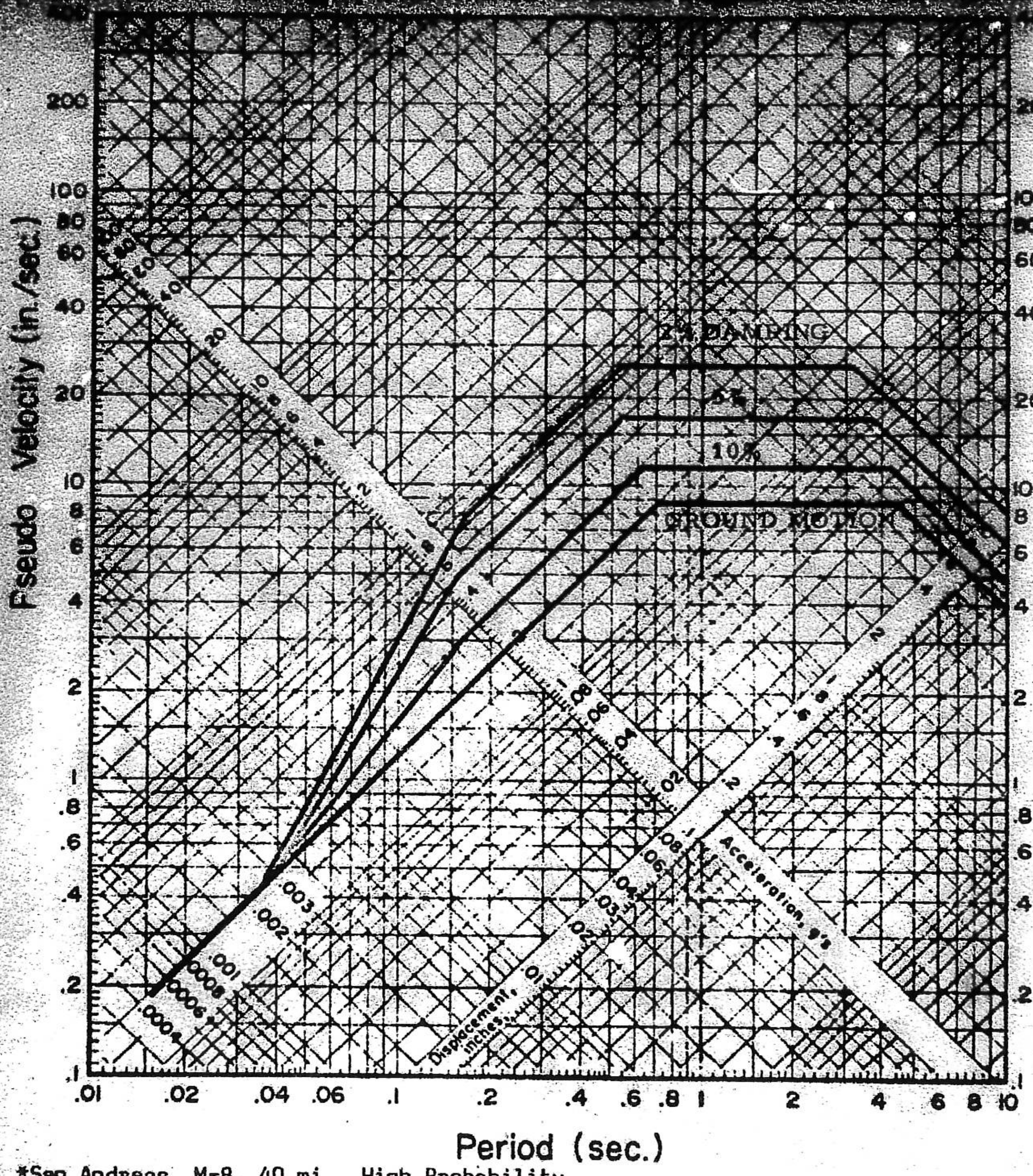
Algermusseen, 1973, BSS 46

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		16



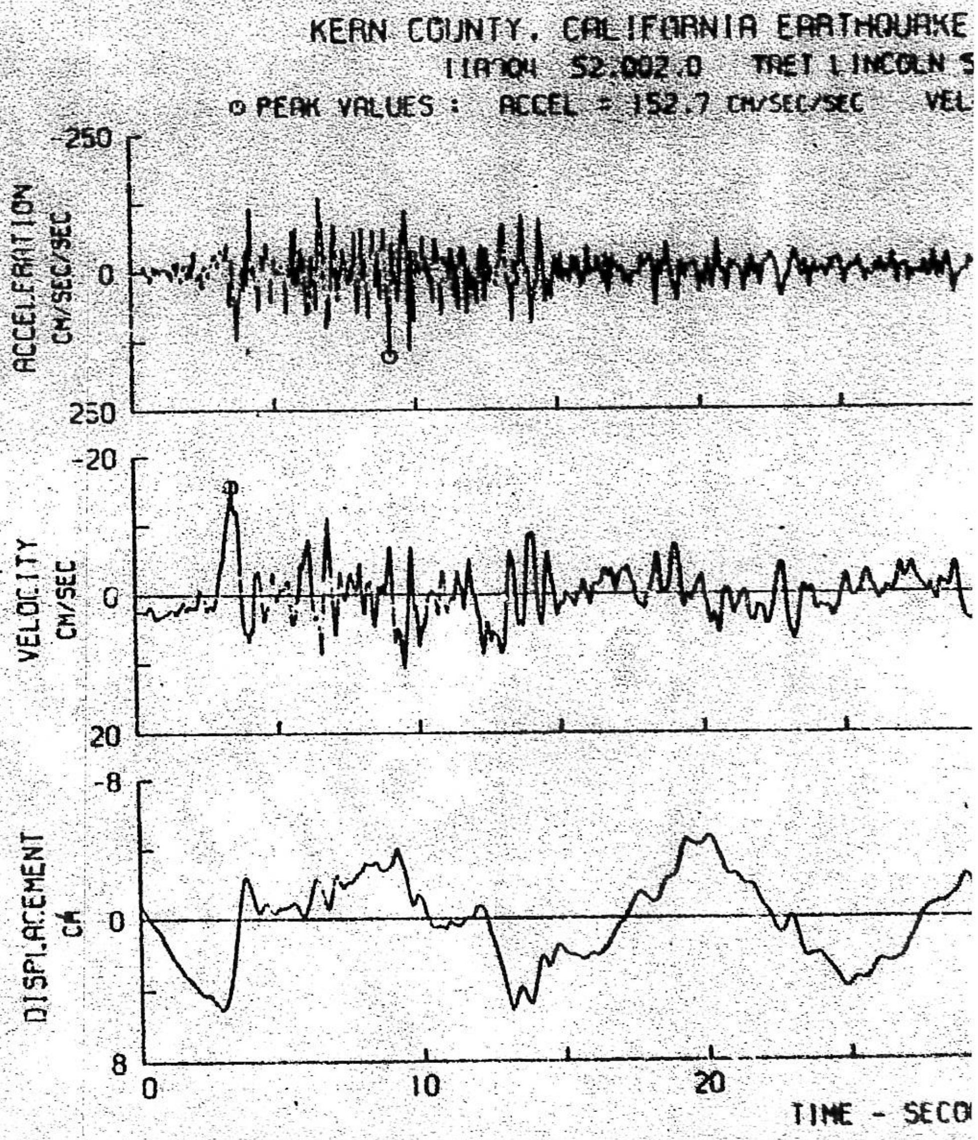
*San Andreas, M=8, 40 mi., High Probability
San Fernando, M=6.5, 20 mi. Moderate Probability
Newport-Inglewood, M=5.5, 20 mi., High Probability
Santa Monica-Hollywood, M=5.5, 1.5 mi., Low Probability
*Control Event

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.
	17b



*San Andreas, M=8, 40 mi., High Probability
San Fernando, M=6.5, 20 mi. Moderate Probability
Newport-Inglewood, M=5.5, 20 mi., High Probability
Santa Monica-Hollywood, M=5.5, 1.5 mi., Low Probability
*Control Event

Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		17b

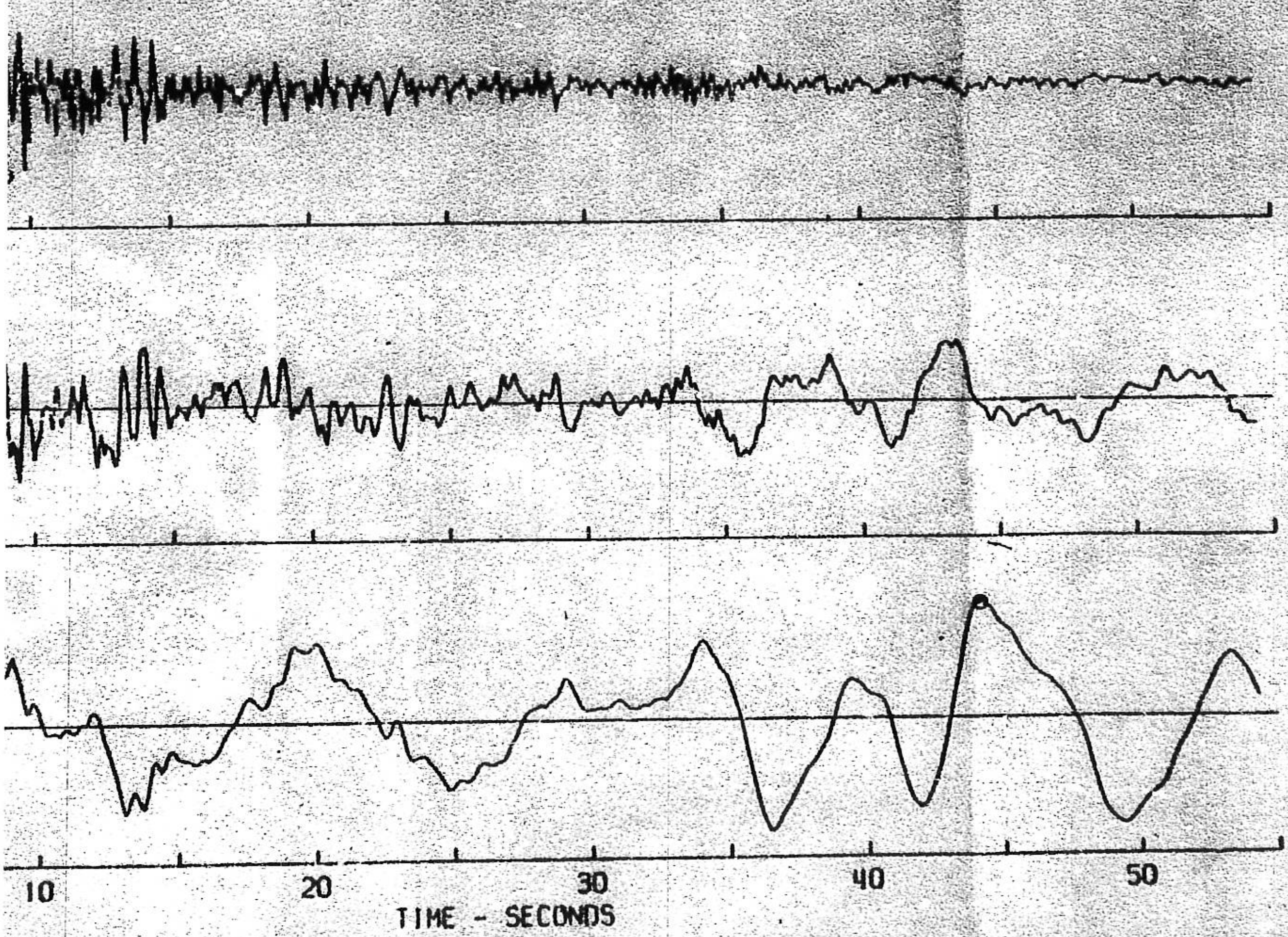


Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.

18

CIT EERL 71-50



COUNTY, CALIFORNIA EARTHQUAKE JULY 21, 1952 - 0453 PDT

IIA004 52.002.0 TAFT LINCOLN SCHOOL TUNNEL COMP N21E

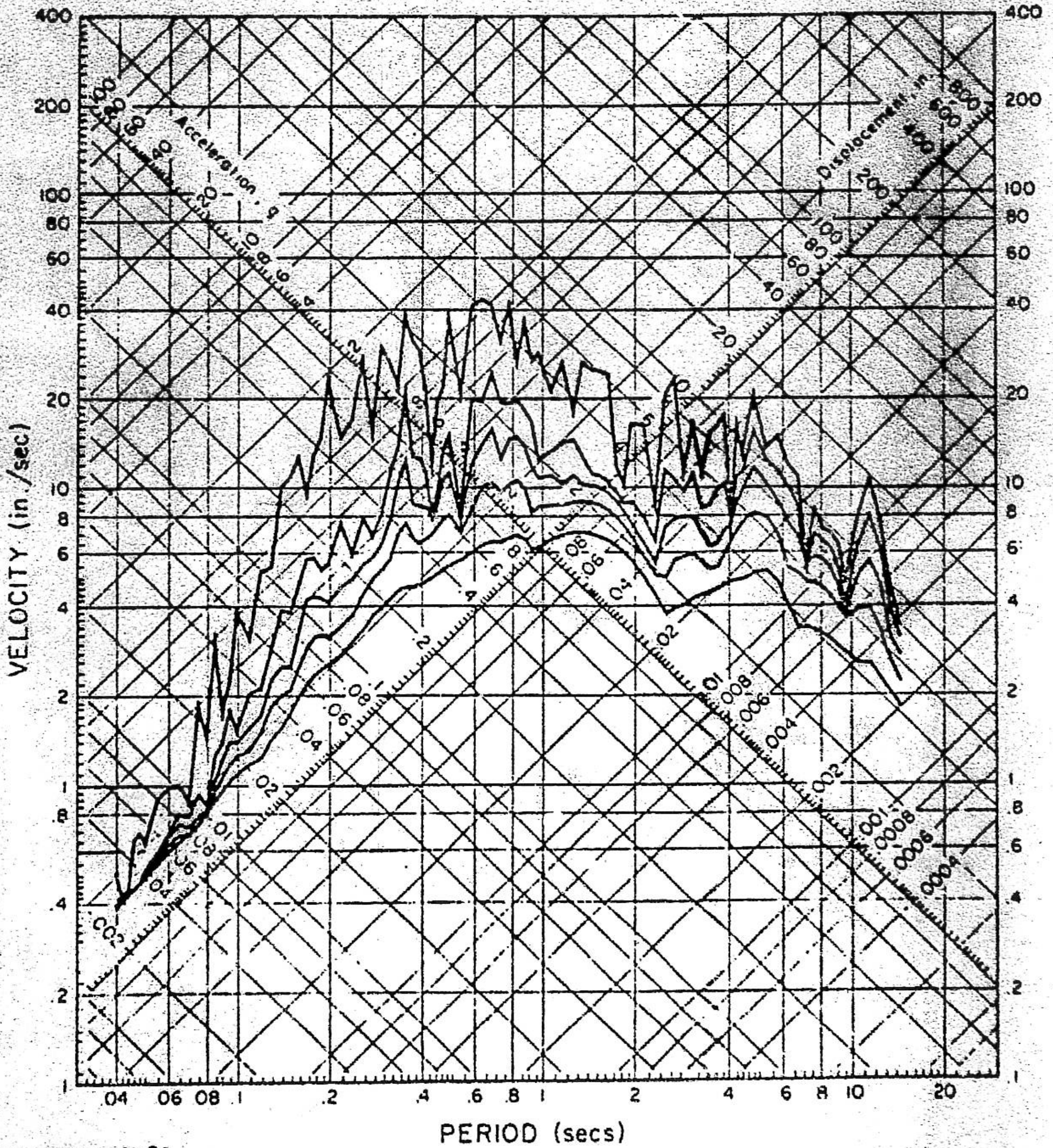
ES : ACCEL = 152.7 CM/SEC/SEC VELOCITY = -15.7 CM/SEC DISPL = -6.7 CM

RESPONSE SPECTRUM

KERN COUNTY, CALIFORNIA EARTHQUAKE JULY 21, 1952 - 0453 PDT

IIIA004 52.002.0 TAFT LINCOLN SCHOOL TUNNEL COMP N21E

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL



CIT EERL 72-80

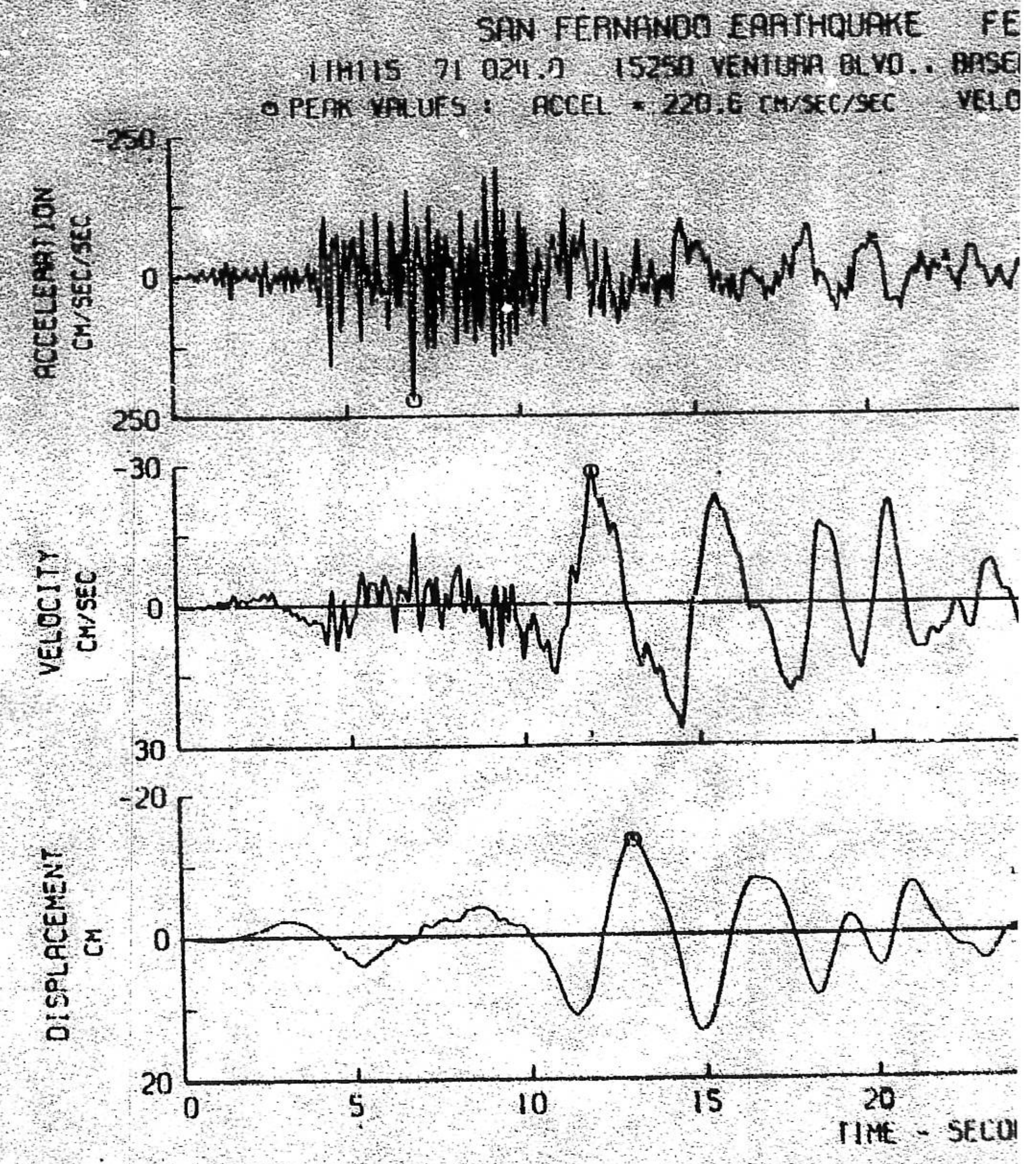
Proposed Office Building for DMG, Inc. Sepulveda Boulevard at Rose Avenue West Los Angeles	PROJECT No.	
		19

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.

20

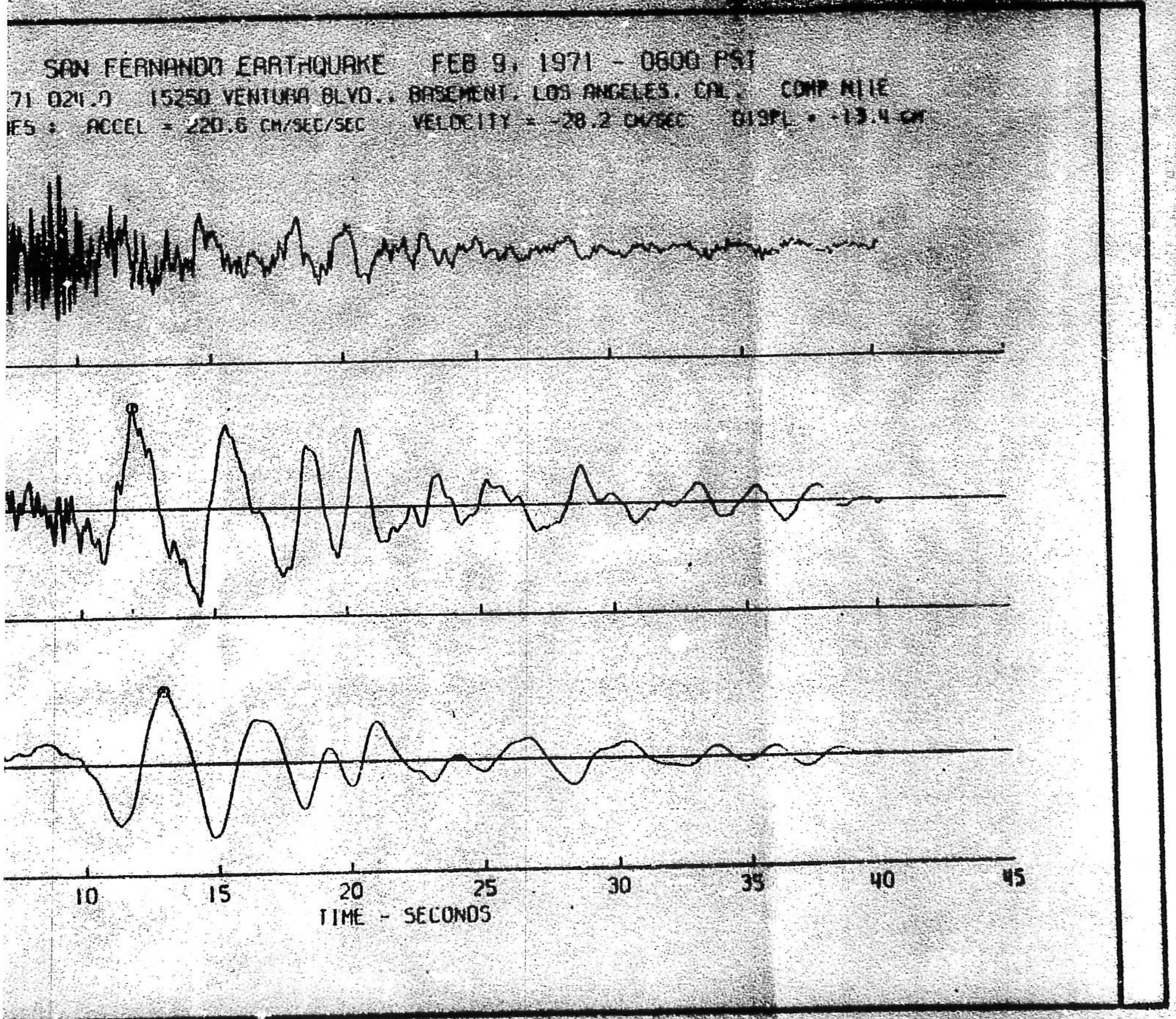
CIT EERL 74-50



SAN FERNANDO EARTHQUAKE FE

IIH115 71 024.0 15250 VENTURA BLVD.. BASE

⊙ PEAK VALUES : ACCEL = 220.6 CM/SEC/SEC VELO



SAN FERNANDO EARTHQUAKE FEB 9, 1971 - 0600 PST

71 024.0 15250 VENTURA BLVD., BASEMENT, LOS ANGELES, CAL. COMP N11E

ES : ACCEL = 220.6 CM/SEC/SEC VELOCITY = -28.2 CM/SEC DISPL = -13.4 CM

TABLE II

EARTHQUAKES OF SOUTHERN CALIFORNIA - 1912-1980

MAGNITUDES 6.0 AND GREATER

DATE	TIME (PST)	LAT.	LONG.	MAG.	INT.	FELT AREA	LOCATION
1915 Jun 22	19:59	32.8	115.5	6.25	VIII	50,000-100,000	Imperial Valley
1915 Jun 22	20:56	32.8	115.5	6.25	VIII	50,000-100,000	Imperial Valley
1915 Nov 20	16:14	32	115	7.1	VII	120,000	Colorado Delta
1916 Oct 22	18:44	34.9	118.9	6	VII	25,000-50,000	Tejon Pass
1918 Apr 21	14:32	33.7	117	6.8	IX	130,000	San Jacinto
1923 Jul 22	23:30	34	117.2	6.25	VII	70,000	San Bernardino
1925 Jun 29	06:42	34.3	119.8	6.3	VIII-IX	--	Santa Barbara
1927 Sep 17	18:07	37.5	118.7	6	VII	75,000	Bishop
1933 Mar 10	17:54	33.6	118.0	6.3	IX	100,000	Long Beach
1934 Dec 30	05:52	32.2	115.5	6.5	IX	60,000	Colorado Delta
1934 Dec 31	10:45	32	114.7	7.1*	X	80,000	Colorado Delta
1935 Feb 24	--	32.0	115.2	6.0	--	--	Colorado Delta
1937 Mar 25	08:49	33.5	116.6	6.0	VII	30,000	Terwilliger Valley
1940 May 18	20:36	32.7	115.5	7.1*	X	60,000+	Imperial Valley
1940 Dec 8	--	31.7	115.1	6.0	--	--	Colorado Delta
1941 Jun 30	23:51	34.4	119.6	6.0	VIII	20,000	Santa Barbara
1942 Oct 21	08:22	33.0	116.0	6.5	VII	35,000	Borrego Valley
1946 Mar 15	05:49	35.7	118.1	6.3	VIII	65,000	Walker Pass
1947 Apr 10	07:58	35.0	116.6	6.2*	VII	75,000	Manix
1948 Dec 4	15:43	33.9	116.4	6.5	VII	65,000	Desert Hot Springs
1952 Jul 21	03:52	35.0	119.0	7.7*	XI	160,000	Kern County
1952 Jul 22	23:53	35.0	118.8	6.4	VII	--	Kern County
1952 Jul 23	05:17	35.2	118.8	6.1	VII	--	Kern County
1952 Jul 28	23:03	35.4	118.9	6.1	VII	--	Kern County
1954 Mar 19	01:54	33.3	116.2	6.2	VI	40,000	Santa Rosa Mountains
1954 Oct 24	--	31.5	116.0	6.0	--	--	Agua Blanca
1954 Nov 12	04:26	31.5	116	6.3	V+	--	Agua Blanca
1956 Feb 9	06:32	31.8	115.9	6.8	VI+	30,000+	San Miguel
1956 Feb 9	--	31.7	115.9	6.1	--	--	San Miguel
1956 Feb 14	10:33	31.5	115.9	6.3	V+	--	San Miguel
1956 Feb 14	17:20	31.5	115.9	6.4	V+	--	San Miguel
1966 Aug 7	09:36	31.8	114.5	6.3	VI	--	Gulf, California
1968 Apr 8	18:29	33.1	116.1	6.5*	--	--	Borrego Mountains
1971 Feb 9	06:01	34.4	118.4	6.6*	[illegible]	80,000	San Fernando
1979 Oct 15	16:17	32.6	115.3	6.6*	X	60,000+	Imperial County

---

*Surface faulting

Proposed Office Building for DMG, Inc.
Sepulveda Boulevard at Rose Avenue
West Los Angeles

PROJECT No.

CITY OF LOS ANGELES
CALIFORNIA

COMMISSIONERS
MAURICE F. MARTINEZ
PRESIDENT
MARCIA MERCUS
VICE-PRESIDENT
RACHEL GULLIVER DUNNE
TOSHIKAZU TERASAWA

TOM BRADLEY
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
[illegible] CITY HALL
LOS ANGELES, CALIF. 90012
JACK M. FRATT
GENERAL MANAGER

December 12, 1980

THE LANDAU PARTNERSHIP
RECEIVED
DEC 24 1980

OWNER: D.M.G. Limited
435 North Bedford Drive
Beverly Hills, CA 90210

APPLICANT: LANDAU PARTNERSHIP
10850 WILSHIRE BOULEVARD
LOS ANGELES, CA 90024

PROJECT ADDRESS: 3415 S. SEPULVEDA BOULEVARD

TRACT: 14879, LOTS 18 and 19

RE: Soil-Geology-Seismology report (Job No. L-1176-FG) dated September 8, 1980, prepared by David J. Leeds and Associates.'

ABSTRACT:

Factored response spectra based on two maximum "credible" earthquakes (distant and local) and one maximum "probable" earthquake (local) are presented for structural damping values of 2%, 5% and 10%. Each response spectra represents the maximum horizontal response of a single degree of freedom system to the predicted ground motion, at the ground surface, resulting from the postulated design earthquakes.

In the Development of the response spectra, procedures were used which consider the effects of local soil and geologic conditions. These site-dependent procedures reflect the current state-of-the art and are presented in literature referenced in the report.

The predicted response of the deposits underlying the site and the influence of local soil and geology conditions during earthquakes were based on statistical results of several comprehensive studies of site-dependent spectra developed from actual time-histories recorded by strong motion instruments located in various parts of the world.

Postulated design earthquakes maximum "credible" and maximum "probable" were selected based on the characteristics of causative faults and statistical probabilities of those earthquakes occurring during the life of the development. The peak ground motion values were based on the attenuation equations of M.D. Trifunac. These values were then used to develop site-dependent response spectra reduced to reflect sustained levels of ground motion rather than absolute peaks. Damping-dependent amplification factors were finally applied to obtain spectral bounds.

Dynamic characteristics of the deposits underlying the site, which were used as a basis for classifying the site into one of the several groups identified in the statistical studies, were estimated from the results of a down hole seismic survey, log borings and static and dynamic test data.

DEPARTMENT RECOMMENDATION:

The subject report is acceptable provided the structural design utilizing the maximum "credible" and maximum "probable" (design) earthquake response spectra are verified by engineering plan check as complying with the current accepted practice for damping, ductility demand and deflection controls permitted for a structure of the type and configuration proposed.

This letter covers only the soil-geology-seismology investigation portion of the report. Any information applicable to the foundation investigation is covered under a separate application for approval.

JOHN O. ROBB
Chief of Grading Division

Horace E. Lumpkin
Horace E. Lumpkin
Structural Engineering Associate

HEL: ua
485-3435

SOILS INTERNATIONAL
A CORPORATION
Consulting Foundation Engineers and Geologists

LOS ANGELES
SANTA ANA
VENTURA
HONOLULU
KAHULUI, MAUI

420 S. PINE STREET, SAN GABRIEL, CA 91776 (213) 287-9769 From L. A.: 283-8907

January 8, 1981
L-1176-F

D.M.G. Limited
435 North Bedford Drive
Beverly Hills, California 90210

Attention: Mr. David Bradley

Gentlemen:

At the request of the Architects, Landau Partnership, and in response to a letter from the Department of Building and Safety, dated October 30, 1980, a review of the foundation plans and an estimate of expected settlement of the various footings within the building have been made.

It should be recognized that these conclusions have been reached based upon the limited investigation performed to date and may be modified subject to findings in the final phase of investigation.

Again, subject to later findings, specific recommendations for footing support are as follows:

1. For footings founded in the clay or silty sand found above elevations of 104 in Boring No.1, 96 in Boring No.2, and 98 in Boring No.3, an allowable bearing value of 3000 pounds per square foot may be used for footings founded at least 2 feet below the lowest adjacent surface. This value may be increased by 10 percent for each additional foot of embedment or width over 2 feet to a maximum of 4000 pounds per square foot.

2. For footings founded in the compact gravelly sand found generally below the elevations given in 1., above, an allowable bearing value of 6000 pounds per square foot is recommended. This value may be increased by 10 percent for each additional foot of width over 2 feet to a maximum of 10,000 pounds per square foot.

The above values are for dead plus live loads and may be increased by one-third when combined with short duration wind or seismic forces.

Settlements of footings as shown on the foundation plans have been estimated and are shown on Plate 1, attached. Also shown are the settlements assuming the same sized footings resting on the deeper gravelly sands. Settlement of other sized footings under different loads would, of course, have to be calculated.

Differential settlements can be calculated by comparing the settlements shown on Plate 1 for various sizes.

Since piles are not being considered, downdrag forces are not applicable. Furthermore, the proposed basement is expected to remove all or most of the fill.

A final report will be submitted confirming or modifying the conclusions and recommendations given above, after the final phase of investigation has been completed.

Respectfully submitted,

SOILS INTERNATIONAL

Robert D. Cousineau

Robert D. Cousineau, RCE 24269

RDC/mk

Dist: (2) D.M.G. Ltd.
(3) Landau Partnership
(3) Ruthroff & Englekirk

SOILS INTERNATIONAL
A CORPORATION
Consulting Foundation Engineers and Geologists

LOS ANGELES
SANTA ANA
VENTURA
HONOLULU
KAHULUI, MAUI

420 S. PINE STREET, SAN GABRIEL, CA 91776 (213) 287-9769 From L. A.: 283-8907

L-1176-F

March 4, 1981

DMG, Ltd.
435 North Bedford Drive
Beverly Hills, California 90210

Re: Sepulveda Center
3415 South Sepulveda Boulevard

Gentlemen:

The following is submitted in response to applicable items of the Department of Building & Safety letter dated February 17, 1981.

Item No. 6

Plans for the foundation of the buildings have been reviewed and are in conformance with the recommendations of the report dated September 8, 1980. Refer also to letter of January 8, 1-81.

Item No. 12

Calculations indicate that vertical cuts up to 14 feet in height (expected maximum cuts) will have an adequate factor of safety. Therefore, no shoring or support for excavations up to this height will be necessary, based on the findings to date, subject, of course, to modification after additional investigation and to observations made during the actual excavation. These calculations assume no surcharge loads, traffic or otherwise, adjacent to and within a distance from the top of slope equal to the depth of excavation.

Plate 2, attached gives the results of calculations for the stability of the proposed cuts.

Respectfully submitted,

SOILS INTERNATIONAL

Robert D. Cousineau, RCE 24269
RDC/mk

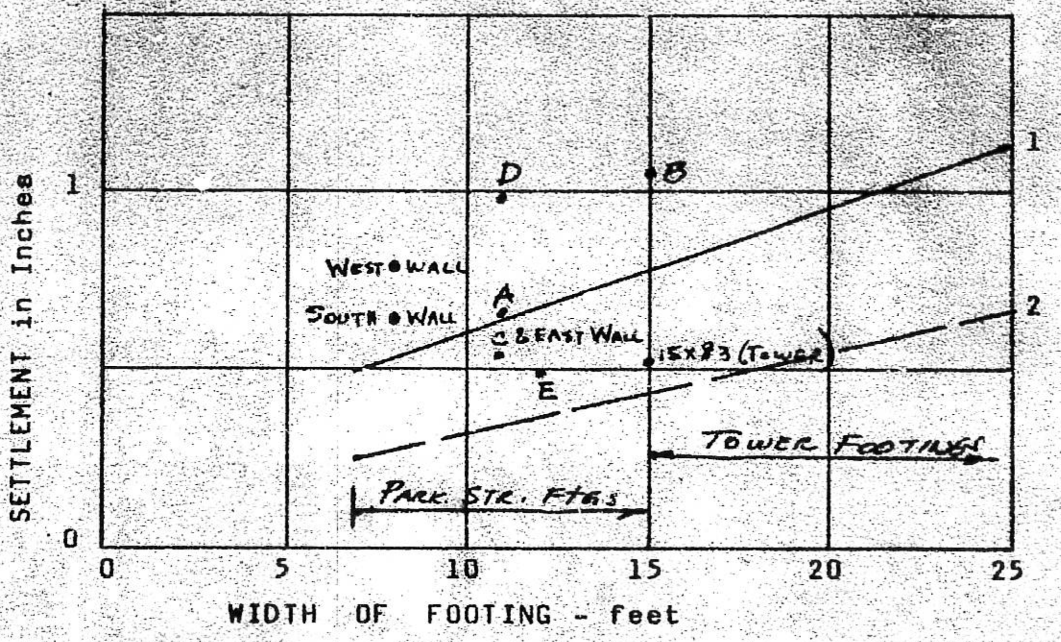
Dist: (1) DMG, Ltd.
(4) Landau Partnership
(Ed Rollerson)
(1) Ruthroff & Englekirk

LEGEND: 1. Estimated settlement of square footings under design loads at depths of footings shown on Plan.

2. Estimated settlement of square footings under same design loads, but founded on compact gravelly sands (at approx. Elevation 100).

LOADING CONDITIONS

TOWER	D+L	PARKING STRUCTURE	D+L
1 - 7'x7'	$3.57^{k}/ft^{2}$	A - 11'x11'	$3.6^{k}/ft^{2}$
2 - 16.5'x16.5'	3.21	B - 15'x15'	3.7
3 - 18'x18'	3.3	C - 12'x96'	2.6
4 - 18'x18'	3.3	D - 10'9"x10'9"	5.5
5 - 22'x22'	3.25-3.44	E - 14'x62'	1.75
6 - 23.5'x23.5'	3.25		
7 - 15'x83'	1.76		



These values are for average conditions reflected by tests. A variation from these values by 20% might be expected due to differences in characteristics both vertically and horizontally. Further, some modification can be expected when the information is available from additional borings in final phase of investigation.

Proposed Office Building for DMG, Ltd. Sepulveda Boulevard at Rose Avenue West Los Angeles		**PROJECT No**	L-1176-F
		PLATE	1

SOILS INTERNATIONAL

CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

S.I. 208 1-74

Reference: "Design Manual - Soil Mechanics, Foundations, and Earth Structures," NAVFAC DM-7, March 1971, Pg. 7-7-8.

Parameters

Slope Height (H) = 14 feet
Slope Angle (β) = 90 degrees
Cohesion (C) = 1000 pounds per square foot
Angle of Friction (φ) = 30 degrees
Wet Unit Weight (γ) = 120 pounds per cubic foot

Calculations

$$\lambda_{C\phi} = \frac{\gamma\ H \tan\phi}{C} = \frac{120 \times 14 \times \tan 30^\circ}{650} = 1.5$$

$N_{cf} = 5.3$ (Fig. 7-4, Pg.7-7-8) → NAVFAC DM 7.02

$$\text{Factor-of-Safety} = \frac{Ncf\ C}{\gamma\ H} = \frac{5.3 \times 1000}{120 \times 14} = 3.15$$

Proposed Office Building for DMG, Ltd.
Sepulved Bouleverd at Rose Avenue
West Los Angeles

PROJECT No.	L-1176-F
PLATE	2

SOILS INTERNATIONAL
CONSULTING FOUNDATION ENGINEERS & ENGINEERING GEOLOGISTS

SOILS INTERNATIONAL
A CORPORATION
Consulting Foundation Engineers and Geologists

LOS ANGELES
SANTA ANA
VENTURA
HONOLULU
KAHULUI, MAUI

420 S. PINE STREET, SAN GABRIEL, CA 91776 (213) 287-9769 From L. A.: 283-8807

File copy

L-1176-F

June 17, 1981

TRACT: Subdivision of Stephens
LOT:
LOCATION: 3415 S. Sepulveda BL

D.M.G., Ltd.
435 North Bedford Drive
Beverly Hills, California 90210

Re: Temporary Excavation Cuts
Sepulveda Center
3415 South Sepulveda Boulevard
Supersedes letter of May 11, 1981

Gentlemen:

At the request of Mr. Ed Rollerson, consideration has been given to the temporary excavation cuts to be made along the south property line.

The character of near-surface materials in this area has not been investigated. Therefore, it is recommended that such cuts provide an inclination from top to toe which does not exceed 3/4 horizontal to 1 vertical. Surcharge or traffic should be kept away from the top of slope a distance equal to the height of cut.

Observations should be made during the initial phase of excavation in order to verify assumed conditions, with the possibilities of modifying the inclination if the material dictates or justifies.

Respectfully submitted,
SOILS INTERNATIONAL

Robert D. Cousineau

Robert D. Cousineau, RCE 24269

RDC/mk

Dist: (1) DMG, Ltd. (2) Landau Partnership (Ed Rollerson)
(1) Robert Englekirk, Inc. (2) Grading Division, City of Los Angeles
(Mr. Niknam

8437

SOILS INTERNATIONAL
A CORPORATION
Consulting Foundation Engineers and Geologists

LOS ANGELES
SANTA ANA
VENTURA
HONOLULU
KAHULUI, MAUI

420 S. PINE STREET SAN GABRIEL, CA 91776 (213) 287-9769 From L. A.: 283-8907

GRADING JAN 26 1983

L-1176-I

January 24, 1983

Murdock Development Co.
10900 Wilshire Boulevard
Suite 16
Los Angeles, California 90024

Attention: Ron Douglas
Vice President

Project Reference: Sepulveda Center
3415 South Sepulveda Boulevard
Los Angeles, California

COMPACTION REPORT

Gentlemen:

This is to report the results of observations and tests performed during the placing and compaction of fill at the referenced project. These observations and tests were required in order to comply with project specifications, recommendations of this office and City ordinances.

Site Conditions

Prior to the commencement of the construction for the tower building and parking structure, the site was essentially flat with a gentle slope toward the north. Asphalt pavement covered the north and east portions of the site. An existing structure was demolished and debris removed from the site.

Grading Procedures

In general, grading and compaction within the specified areas were performed as follows:

1. The area within the proposed tower and garage structures was excavated to an approximate elevation of 108.8.

2. Further excavations for the elevator pit, water storage tank, sump pits and transformer vault were made within the excavated building area.

3. Within the building areas, the upper 12 inches of subgrade was compacted prior to placement of the basement slab.

4. Within the exterior open parking areas to the east and north of the buildings, the fill was placed over the existing asphalt pavement.

5. Backfills were placed and compacted behind basement (retaining) walls.

6. All fill was spread, moistened and/or allowed to dry and compacted by vibratory equipment and/or wheel-rolling with construction equipment in lifts not exceeding 6 inches in thickness.

7. The specified minimum degree of compaction was 90 percent of the maximum dry density, as determined by the ASTM D-1557-78 standard. Maximum dry density and optimum moisture content of the various fill soils used are tabulated in Table One. In-place soil densities were determined in accordance with ASTM D-1556, standard method. The results of tests are tabulated in Table Two. The test locations are shown on Plate A, attached.

8. Backfill was placed and compacted within utility trenches shown on Plate A, attached.

CONCLUSIONS AND RECOMMENDATIONS

The observations and test results indicate that compaction was accomplished in accordance with the Soils International report dated September 8, 1980, earthwork specifications prepared by The Landau Partnership, Architects and Planners, and City of Los Angeles grading regulations.

The soils used in fills on this project are generally classified as clayey sands and sandy clays. The completed subgrade condition is consistent with design recommendations presented in the Foundation Investigation Report dated September 8, 1980.

This report pertains only to the grading and compaction described herein and is subject to review by the controlling authorities for the project.

Respectfully submitted,
SOILS INTERNATIONAL

Andre M. Minassian, RCE 33813

AMM/RDC/mk

Dist:
(2) Murdock Development Co.
(1) Robert Englekirk, Inc., Structural Engineers
(1) The Landau Partnership, Architects and Planners
(1) Jones Brothers, c/o Jobsite
(2) City of Los Angeles

CITY OF LOS ANGELES

VERIFICATION OF COMPLIANCE

Job Address: 3415 S. Sepulveda Boulevard
Los Angeles, California 90034

Owner: Murdock Development Co.
10900 Wilshire Boulevard, Suite 16
Los Angeles, CA 90024

Date Work Started on Project: 10/29/81
Date Work Covered by this Report: 8/12/82
Date of this Verification: 1/24/83

To the Superintendent of Building:

I verify that I have personally supervised others who have observed and performed tests during the placing of compacted earth fill on the above described property, and that such observation and tests have indicated that the fills were placed in accordance with the requirements of the Building Code of the City of Los Angeles.

A. M. Minassian

Andre M. Minassian, RCE 33813

CITY OF LOS ANGELES

VERIFICATION OF COMPLIANCE

Job Address: 3415 S. Sepulveda Boulevard
Los Angeles, California 90034

Owner: Murdock Development Co.
10900 Wilshire Boulevard, Suite 16
Los Angeles, CA 90024

Date Work Started on Project: 10/29/81

Date Work Covered by this Report: 8/12/82

Date of this Verification: 1/24/83

To the Superintendent of Building:

I verify that I have personally supervised others who have observed and performed tests during the placing of compacted earth fill on the above described property, and that such observation and tests have indicated that the fills were placed in accordance with the requirements of the Building Code of the City of Los Angeles.

A. M. Minassian

Andre M. Minassian, RCE 33813

TABLE ONE
LABORATORY TEST SUMMARY

Soil Type	Classification	Maximum Dry Density (pcf)	Optimum Moisture Content (% Dry Wt.)
I	CLAY, sandy with some gravel	126.0	11.0
II	SAND, clayey	128.0	11.0
III	SAND, fine to medium silty with some gravel	133.0	8.0
IV	CLAY, silty, sandy with some gravel	125.0	12.0

TABLE TWO
RELATIVE COMPACTION SUMMARY

Test No.	Date of Test (1981)	Depth below finish grade (feet)	Soil Type	Moisture Content (%)	Dry Unit Weight (pcf)	Relative Compaction (%)
1	10/28	@ subgrade	I	14.7	123.0	98
2	10/28	"	I	14.4	125.1	99
3	10/28	"	I	15.3	115.9	92
4	11/3	"	I	18.8	116.8	93
5	11/3	"	I	18.3	116.4	92
6	11/3	"	I	17.9	121.2	96
7	11/3	"	I	10.1	124.0	98
8	11/6	"	I	19.0	111.3	88*
9	11/6	"	I	13.9	119.0	94
10	11/25	4.5	I	15.2	108.7	86*
11	11/25	4.0	I	11.7	102.9	82
12(11)	11/25	4.0	I	11.3	120.1	95
13	12/3	4.5	II	12.3	114.9	90
14	12/3	4.5	II	15.7	109.9	86
15(14)	12/3	4.5	II	10.9	115.0	90
16	12/3	3.5	II	14.4	120.8	94
17	12/3	3.5	II	15.5	116.8	91
18	12/4	3.5	II	18.8	117.7	92
19	12/4	3.5	II	17.1	117.4	92
20	12/4	3.0	II	16.0	119.1	93
21	12/7	2.0	III	11.2	127.1	96
22	12/7	1.5	III	4.4	137.4	103
23	12/7	0.5	III	7.2	134.3	100+
	(1982)					
24	2/15	3.5	II	15.6	115.8	90
25	2/16	3.0	II	17.6	116.7	91
26	2/16	2.5	II	16.4	118.6	93
27	2/22	3.5	II	15.0	119.9	94
28	2/22	4.0	II	13.3	119.0	93
29	2/22	4.5	II	13.7	119.8	94
30	2/24	1.0	II	12.9	121.3	95
31	2/24	2.0	II	13.5	116.8	91
32	2/25	5.5	II	12.0	116.4	91
33	2/25	5.0	II	15.2	120.6	94
34	2/25	0.5	II	13.6	117.4	92
35	2/25	0.5	II	14.2	118.9	93
36	2/25	1.5	II	13.0	117.0	91
37	3/1	4.0	II	12.9	117.5	92
38	3/1	3.0	II	12.2	115.9	91
39	3/2	1.5	II	11.2	115.7	90
40	3/2	0.5	II	12.0	117.2	92
41	3/2	0	II	11.5	116.3	91
42	3/29	9.5	II	18.2	111.6	87*

* Materal too wet, removed.
12(11) Retest of test in parenthesis

Sepulveda Center

L-1176-I

SOILS INTERNATIONAL

TABLE TWO
RELATIVE COMPACTION SUMMARY

Test No.	Date of Test (1982)	Depth below finish grade (feet)	Soil Type	Moisture Content (%)	Dry Unit Weight (pcf)	Relative Compaction (%)
43	3/29	11.0	II	17.6	107.7	84*
44	4/9	8.8	II	15.5	122.7	96
45	4/9	8.0	II	14.0	123.8	97
46	4/9	8.0	II	16.0	116.5	91
47	4/20	10.0	II	18.3	126.0	98
48	4/20	10.5	II	17.6	119.3	93
49	4/20	8.0	II	16.0	118.2	92
50	4/20	8.5	II	18.1	118.0	92
51	4/20	10.5	II	16.0	121.8	95
52	4/20	8.0	II	17.0	121.1	95
53	4/20	6.0	II	16.0	117.8	92
54	4/20	6.0	II	15.6	115.2	90
55	4/21	5.0	II	14.6	116.0	91
56	4/21	5.0	I	13.5	115.4	92
57	4/21	4.0	I	14.4	115.8	92
58	4/21	4.0	II	14.9	115.9	91
59	4/21	4.0	II	14.3	118.5	92
60	4/21	10.0	II	15.2	118.7	93
61	4/22	9.5	II	14.2	114.7	90
62	4/22	8.0	II	15.8	116.6	91
63	4/22	8.5	II	18.6	113.0	88
64(63)	4/26	8.5	II	13.3	117.0	91
65	4/26	7.5	II	12.0	119.7	94
66	4/26	7.5	II	12.8	118.1	92
67	4/26	6.0	II	13.3	119.2	93
68	4/26	5.0	II	14.7	118.3	92
69	4/26	5.0	II	15.0	119.7	94
70	4/26	6.0	II	13.6	116.5	91
71	4/27	6.0	II	11.8	119.2	93
72	4/27	6.5	II	13.1	117.5	92
73	4/27	0.5	II	12.0	117.7	92
74	4/27	0.5	II	14.4	119.6	93
75	4/27	0.5	II	13.5	118.8	93
76	4/28	5.0	II	14.5	116.9	91
77	4/28	5.0	II	13.3	118.9	93
78	4/28	4.0	II	14.1	118.4	93
79	4/23	4.0	II	15.0	116.2	91
80	4/28	4.0	II	15.5	117.1	92
81	4/28	3.0	II	14.5	117.8	92
82	4/29	3.0	II	14.8	118.1	92
83	4/29	5.0	II	14.4	117.0	91
84	4/29	2.0	II	14.1	117.1	92
85	4/29	2.0	II	15.5	118.1	92
86	4/29	2.0	II	15.8	117.2	92

TABLE TWO
RELATIVE COMPACTION SUMMARY

Test No.	Date of Test (1982)	Depth below finish grade (feet)	Soil Type	Moisture Content (%)	Dry Unit Weight (pcf)	Relative Compaction (%)
87	4/29	2.5	II	14.8	117.3	92
88	4/29	1.5	II	14.0	118.4	93
89	4/30	1.5	II	14.6	116.6	91
90	4/30	1.5	II	13.3	117.1	92
91	4/30	0.5	II	13.0	117.2	92
92	4/30	0.5	II	13.5	119.6	93
93	4/30	10.0	II	12.1	119.7	94
94	5/3	9.0	II	13.5	119.3	93
95	5/3	0.5	II	12.0	118.9	93
96	5/3	8.0	II	14.1	117.4	92
97	5/3	10.0	II	14.7	118.5	93
98	5/3	6.0	II	12.8	119.4	93
99	5/3	8.0	II	13.8	122.6	96
100	5/3	4.0	II	13.6	118.0	92
101	5/4	4.0	II	12.2	117.6	92
102	5/4	6.0	II	13.5	117.1	92
103	5/4	9.0	II	13.3	117.0	91
104	5/4	5.0	II	12.6	119.1	93
105	5/4	5.0	II	13.3	117.8	92
106	5/4	8.0	II	14.2	118.2	92
107	5/5	8.0	II	12.7	118.3	92
108	5/5	7.5	II	14.4	115.7	90
109	5/5	7.5	II	14.0	117.5	92
110	5/5	3.5	II	14.2	119.9	94
111	5/5	3.5	II	13.4	118.5	93
112	5/5	5.5	II	13.8	118.8	93
113	5/5	3.5	II	14.1	118.3	92
114	6/8	9.0	II	14.1	116.0	91
115	6/8	9.0	II	12.4	116.9	91
116	6/9	10.0	II	12.4	117.7	92
117	6/9	10.0	II	12.9	118.6	93
118	6/9	9.0	II	14.4	118.6	93
119	6/9	7.5	II	14.5	117.7	92
120	6/9	10.0	II	13.4	116.9	91
121	6/9	6.5	II	13.1	116.8	91
122	6/9	8.0	II	12.6	116.5	91
123	6/9	7.0	II	13.5	118.7	93
124	6/9	7.0	II	13.4	116.8	91
125	6/10	8.0	II	15.0	116.0	91
126	6/10	6.0	II	14.3	118.8	93
127	6/10	7.0	II	13.6	116.1	91
128	6/10	6.0	II	14.9	117.2	92
129	6/10	5.5	II	13.0	115.1	90
130	6/10	6.0	II	14.3	117.9	92
131	6/10	6.0	II	15.0	116.2	91
132	6/11	5.0	II	13.7	117.5	92

TABLE TWO
RELATIVE COMPACTION SUMMARY

Test No.	Date of Test (1982)	Depth below finish grade (feet)	Soil Type	Moisture Content (%)	Dry Unit Weight (pcf)	Relative Compaction (%)
133	6/11	5.0	II	11.3	119.6	93
134	6/11	5.0	II	14.4	118.4	93
135	6/11	5.0	II	13.4	118.5	93
136	6/11	5.0	II	13.9	120.0	94
137	6/11	8.0	II	15.0	118.1	92
138	6/11	5.0	II	14.1	120.6	94
139	6/11	2.0	II	12.0	118.3	92
140	6/11	3.0	II	14.4	120.6	94
141	6/14	5.5	II	13.1	115.0	90
142	6/14	9.0	II	15.9	110.6	86
143(142)	6/14	9.0	II	14.1	117.2	92
144	6/14	4.5	II	14.8	117.9	92
145	6/14	4.0	II	14.4	116.6	91
146	6/14	3.0	II	14.4	116.0	91
147	6/14	4.0	II	14.7	118.3	92
148	6/14	4.0	II	14.1	117.7	92
149	6/16	3.0	II	12.9	117.7	92
150	6/16	2.0	II	14.3	120.6	94
151	6/16	2.0	II	14.4	120.8	94
152	6/16	1.0	II	12.9	118.8	93
153	6/16	1.0	II	13.9	118.8	93
154	6/16	0.5	II	14.5	119.4	93
155	6/16	2.0	II	13.3	120.0	94
156	6/17	1.5	II	9.6	119.0	93
157	6/17	1.0	II	13.0	119.5	93
158	6/17	0.5	II	12.5	120.0	94
159	6/17	0.5	II	12.1	121.6	95
160	6/17	2.0	II	13.6	119.5	93
161	6/17	2.0	II	12.8	117.6	92
162	6/17	2.0	II	14.1	117.3	92
163	6/18	0.5	II	13.2	117.6	92
164	6/18	2.0	II	12.3	120.1	94
165	6/18	1.0	II	13.7	117.2	92
166	6/18	1.0	II	14.2	118.7	93
167	6/18	0.5	II	14.5	118.6	93
168	6/18	1.0	II	13.7	118.4	93
169	6/18	1.0	II	12.9	119.6	93
170	6/21	0.5	II	14.0	121.9	95
171	6/21	0.5	II	13.8	119.5	93
172	6/21	2.0	II	14.4	116.9	91
173	6/21	2.0	II	14.4	116.3	91
174	6.21	0.5	II	14.7	119.3	93
175	6/21	0.5	II	14.4	120.1	94
176	6/21	1.0	II	13.6	118.0	92

TABLE TWO
RELATIVE COMPACTION SUMMARY

Test No.	Date of Test (1982)	Depth below finish grade (feet)	Soil Type	Moisture Content (%)	Dry Unit Weight (pcf)	Relative Compaction (%)
177	6/23	4.0	II	17.2	120.8	94
178	6/23	4.0	II	19.3	116.2	91
179	6/23	4.0	II	17.6	119.3	93
180	6/24	3.0	I	12.6	114.8	91
181	6/24	3.5	I	13.0	115.8	92
182	6/28	0	I	13.9	116.8	93
183	6/28	0	I	14.0	115.8	92
184	7/20	0.5	IV	12.1	122.7	98
185	7/20	0.5	IV	13.0	119.5	96
186	7/20	0.5	IV	13.5	120.9	97
187	7/20	0.5	IV	12.5	122.0	98
188	7/21	0.5	IV	13.4	123.5	99
189	7/21	0.5	IV	11.7	120.0	96
190	7/21	0.5	IV	13.6	123.5	99
191	7/21	0.5	IV	11.9	121.6	97
192	7/21	0.5	IV	13.9	119.8	96
193	7/21	0.5	IV	14.1	121.4	97
194	7/21	0.5	IV	14.3	122.0	98
195	7/21	0.5	IV	14.7	119.5	96
196	7/23	0.5	IV	15.1	121.9	98
197	7/23	10.0	IV	15.8	120.6	97
198	7/23	9.5	IV	16.4	118.3	95
199	7/23	8.5	IV	18.3	112.7	90
200	7/29	4.5	I	15.9	122.0	97
201	7/29	1.5	I	14.6	121.1	96
202	7/29	0.5	I	15.0	122.9	98
203	7/29	0.5	I	15.4	121.7	97
204	7/30	9.0	I	15.2	122.5	97
205	7/30	9.0	I	15.6	120.2	95
206	7/30	8.0	I	15.1	122.9	98
207	7/30	7.0	I	16.4	119.2	95
208	7/30	4.0	I	14.8	123.7	98
209	7/30	6.0	I	14.5	120.2	95
210	7/30	5.5	I	14.6	120.6	96
211	7/30	4.5	I	15.7	121.2	96
212	8/2	10.0	II	12.2	118.5	93
213	8/2	9.0	II	12.4	120.5	94
214	8/2	8.0	II	14.4	121.4	95
215	8/2	6.0	II	15.7	117.5	92
216	8/2	5.5	I	13.6	122.3	97
217	8/2	5.5	I	14.1	120.5	96
218	8/3	4.5	I	11.2	120.6	96
219	8/3	5.0	II	12.1	123.5	97
220	8/3	5.0	II	12.1	122.9	96
221	8/3	4.5	II	12.6	125.3	98
222	8/4	4.5	II	13.0	124.2	97

TABLE TWO
RELATIVE COMPACTION SUMMARY

Test No.	Date of Test (1982)	Depth below finish grade (feet)	Soil Type	Moisture Content (%)	Dry Unit Weight (pcf)	Relative Compaction (%)
223	8/4	2.5	II	12.6	122.6	96
224	8/4	2.5	II	13.4	124.3	97
225	8/4	1.5	II	13.0	123.5	97
226	8/5	0.5	I	13.3	124.8	99
227	8/5	2.0	I	12.4	123.6	98
228	8/5	0.5	I	12.7	122.1	97
229	8/5	0.5	I	11.9	124.0	98
230	8/6	0.5	I	12.9	122.5	97
231	8/6	0.5	I	13.5	122.3	97
232	8/6	0.5	I	12.9	121.9	97
233	8/6	0.5	I	11.3	114.5	91
234	8/11	1.0	I	13.1	116.1	92
235	8/11	2.0	I	12.8	117.9	94
236	8/11	4.0	I	13.4	116.8	93
237	8/11	1.0	I	11.3	119.4	95
238	8/11	3.0	I	11.6	116.4	92
239	8/12	0.5	III	8.9	126.5	95
240	8/12	1.5	III	10.0	123.4	93

CALIFORNIA

MAURICE E. MARTINEZ
PRESIDENT
MARCIA MARCUS
VICE-PRESIDENT

RACHEL GULLNER DUNNE
TOM [illegible] TERASAWA
HONORA D. WELLS

TOM BRADLEY
MAYOR

[illegible] SAFETY
[illegible] 90012

THE LANDAU PARTNERSHIP
RECEIVED
NOV [illegible] 1980

October 30, 1980

DMG Limited
435 North Bedford Drive
Beverly Hills, CA 90210

JOB ADDRESS: 3415 SOUTH SEPULVEDA BOULEVARD

The Department of Building and Safety has reviewed the foundation investigation report No. L-1176-FG, dated September 8, 1980, prepared by Soils International, and the Geological Report, dated September 8, 1980, prepared by David J. Leeds and Associates, concerning a proposed 12-story tower and adjacent parking structure.

Approval of the proposed development will be considered after a foundation investigation report is submitted to the Grading Division containing specific foundation design recommendation. The report shall be based on review of development plans for the proposed structures and shall include special consideration of the existing fill soil and its effect on the proposed structure, including downdrag effect on the proposed piles. An estimate of anticipated differential settlement of the proposed structure shall also be provided.

JOHN O. ROBB
Chief of Grading Division

MICHAEL R. WOOD
Engineering Associate

Ivan O. Tkatch
IVAN O. TKATCH
Engineering Geologist

MRW/IOT:ua
485-3435

cc: The Landau Partnership
Soils International
David J. Leeds & Assoc.
WLA District Office
WLA Plan Check
LA Plan Check

CITY OF LOS ANGELES
CALIFORNIA

COMMISSIONERS
MAURICE E. MARTINEZ
PRESIDENT
MARCIA MARCUS
VICE-PRESIDENT
RACHEL GULLIVER DUNNE
TOSHIKAZU TERASAWA

TOM BRADLEY
MAYOR

DEPARTMENT OF BUILDING AND SAFETY
[illegible] CITY HALL
LOS ANGELES, CALIF. 90012
[illegible]
GENERAL MANAGER

February 17, 1981

DHG, Limited
435 N. Bedford Drive
Beverly Hills, CA 90210

3415 SOUTH SEPULVEDA BOULEVARD

REF: Department letter dated October 30, 1980.

The Department of Building and Safety approves the Foundation Investigation Reports No. L-1176-F, dated September 8, 1980 and January 8, 1981, prepared by Soils International and Geology report, dated September 8, 1980, prepared by David J. Leeds and Associates, concerning a proposed 12-story tower and adjacent parking structure.

The plans shall comply with the recommendations contained in the foundation engineer's report and the additional conditions listed below:

1. A grading permit shall be obtained, for all structural fill, and retaining wall backfill.
2. Existing fill soil shall be removed from the site.
3. If the actual foundation design loads do not conform to the foundation loads assumed in the report, the Foundation Engineer shall submit a supplementary report containing specific design recommendations for the heavier loads to the Department for review and approval prior to issuance of a permit.
4. The building design shall incorporate provisions for anticipated differential settlements in excess of one-fourth inch.
5. Frictional and lateral resistance of soils may be combined, provided the lateral bearing resistance does not exceed 2/3 of allowable lateral bearing.
6. The geologist and soils engineer shall review and approve the detailed plans, prior to issuance of any permits.

7. The consulting geologist and/or soils engineer shall inspect the excavations for the footings to determine that they are founded in the recommended strata before calling the Department for footing inspection.

8. Suitable arrangements shall be made with the Department of Public Works for the proposed removal of support and/or retaining of slopes adjoining the public way.

9. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety.

10. A supplemental report shall be submitted to the Grading Division containing recommendations for shoring, underpinning and sequence of construction if any excavation would remove the lateral support of the public way or adjacent structures. A plot plan showing the type, number of stories, and location of any structures (or absence of any structures) adjacent to the excavation shall be provided with the excavation plans.

11. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans. Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

12. Prior to issuance of permits, a supplemental report shall be submitted to the Grading Division containing detailed recommendations, including design criteria and sequence of construction for both permanent and temporary support of proposed excavations. Show proposed depth and location of excavations on plot plan.

13. All of the recommendations of the report which are in addition to or more restrictive than those contained herein shall be incorporated into the plans.

JOHN O. ROBB
Chief of Grading Division

Ivan O. Tkatch
Engineering Geologist
Ivan O. Tkatch

Michael R. Wood
Engineering Associate

MRW/IOT:mcg
485-3435

cc: The Landau Partnership, Inc.
Soils International
David J. Leeds & Assoc.
WLA District Office
LA Plan Check
WLA Plan Check

GL0211A:95