Western Laboratories
Geotechnical Engineering

GEOTECHNICAL ENGINEERING REPORT

PROPOSED CARWASH & RETAIL BUILDINGS
4201 E. WILLOW STREET
LONG BEACH, CALIFORNIA

FEBRUARY 26, 2010

WORK ORDER 09-4329

PREPARED FOR:

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Western Laboratories
Geotechnical Engineering

February 26, 2010

Work Order 09-4329

LONG BEACH GATEWAY LLC
c/o Joon Kim
3055 Wilshire Boulevard, Suite 810
Los Angeles, California 90010

Subject: Geotechnical Exploration
Proposed Carwash & Retail Buildings
4201 E. Willow Street
Long Beach, California

Dear Sir:

Pursuant to your authorization to provide geotechnical engineering consulting services for the above referenced project, the accompanying Geotechnical Engineering Report has been prepared.

Based upon the subsurface conditions that were encountered during our exploration, the results of laboratory tests, and the findings from our liquefaction potential and earthquake-induced settlement analysis, it is our conclusion that the proposed development is feasible from a geotechnical engineering standpoint, provided the recommendations contained herein are incorporated into project planning, design, and construction. Structural support for the proposed buildings should be provided by conventional spread footing foundations that are constructed into compacted/engineered fill soils.

The contents in this report should be reviewed in detail and be made a portion of the project design package. Please contact this office if any questions arise regarding the contents of this report.

Respectfully submitted,

WESTERN LABORATORIES

Edward Castellanos
GE 191

PROFESSIONAL ENGINEER EDWARD CASTELLANOS 3/10 No. 191 GEOTECHNICAL STATE OF CALIFORNIA

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Introduction

This report presents the results of our geotechnical exploration performed for the proposed carwash and retail buildings to be constructed at 4201 E. Willow Street, in the City of Long Beach, California. Our exploration was limited due to the presence of existing site improvements.

It is proposed to demolish the existing auto dealership and associated improvements and construct a carwash facility and two (2) one story retail buildings on the subject property. It is anticipated that masonry block and/or timber and stucco will be used in construction of the exterior walls of the proposed structures with concrete slabs-on-grade as the flooring systems.

Purpose and Scope of Work

The purpose of our exploration was to evaluate the subsurface soil conditions at the site to provide geotechnical engineering recommendations for design and construction of the proposed structures. Our work was conducted in accordance with generally accepted geotechnical engineering principles and practice at this time and location. The scope of our services included: field examination of the site, subsurface exploration by hollow-stem auger and rotary wash borings, and a cone penetration test (CPT), soil classification, laboratory testing on selected soil samples encountered, and analyzing the results of the field and laboratory work developed for this site to provide the geotechnical design information contained herein.

Site Description

At the time of our exploration, the property was occupied by a auto dealership, which is comprised of a sales building located at the southeast corner of the property and three service buildings. The surface of the site is paved with asphaltic concrete. The property is bordered on the east by Lakewood Boulevard, on the south by Willow Street, on the west by a Residence Inn, and on the north by Caltrans property.

Field Exploration

The subsurface conditions were explored by drilling four (4) 8" diameter hollow-stem auger borings, one (1), 4" diameter rotary wash boring and performing one (1), CPT sounding at the locations depicted on the attached Plot Plan. The borings were logged by our field representative and disturbed and relatively undisturbed samples were obtained for laboratory testing and analysis.

Descriptions of the materials encountered during our exploration are presented on the boring logs and the CPT-01.txt data sheets contained in Appendix A. The logs and data sheets only depict subsurface conditions on the dates shown on the logs and the data sheets at the approximate locations shown on the Plot Plan. Subsurface conditions may differ across the site from the conditions encountered in our borings and the CPT probe sounding.

The hollow-stem auger borings were performed by 2R Drilling, Inc., the rotary wash boring was performed by Socal Drilling, under the supervision of Western Laboratories (WL). Standard Penetration Tests (SPT) were performed by driving a 2-inch outside diameter, 1⅜-inch constant inside diameter split-barrel sampler into in-situ soil to obtain a measure of the resistance of the soil to the penetration of the sampler. This test developed by Terzaghi & Peck using the above sampler determines the number of consecutive blows required to drive the sampler from 6 to 18 inches. The reporting standard is the number of blows (the "N" value) used to drive the sampler 1.0 foot utilizing a 140-pound hammer with a 30-inch drop. These blow count numbers are presented on the boring logs.

Bulk and relatively undisturbed soil samples were obtained at depths appropriate to the exploration. Relatively undisturbed ring samples were obtained from the borings using a Drive Sampler provided by 2R Drilling, Inc., with an outside diameter of 3.25" and an inside diameter of 2.42", driven with a 140-lb. hammer and a drop of 30 inches. The drive barrel of the sampler is lined with numerous 1-inch brass rings. The bottom portion of the ring samples were retained for testing. All samples were carefully sealed in moisture-resistant containers, labeled, logged and transported to our laboratory. Bulk, SPT and relatively undisturbed ring samples served as the basis for the laboratory testing and engineering conclusions contained in this report.

Following a review of the field logs and the condition of the retrieved samples, a laboratory testing program was set-up that took into consideration the probable foundation type for the proposed structures. A brief description of the laboratory tests performed along with the test results are included in Appendix B, or shown on the logs.

Geologic Setting

The property is situated at an elevation of approximately 32 feet above Mean Sea Level. The moderately rugged Peninsular Range comprises the geologic province and it has a southeasterly trend that extends from the Santa Monica Mountains into Baja California.

The major geologic formations of the region include alluvium at the valley floors and sedimentary and metamorphic bedrock in the mountainous areas. Major fault lines include the nearby Newport-Inglewood (L.A. Basin) and Palos Verdes systems. Geology of significance at the site consists of two basic units, alluvium and artificial fill. The geologic maps contained in Appendix C present an overview of the site.

The alluvium constitutes the main unit and extends to the maximum depth drilled of approximately 52.5 feet at the rotary wash boring location and approximately 40 feet below the existing pavement at the CPT probe location. Artificial fill soils cover the alluvium.

Subsurface Conditions

Artificial fill soils were encountered in all of our borings, ranging in depth from approximately 2.0 to 3.0 feet below the existing asphalt pavement. These soils are comprised of soft to stiff, moist, silty clays, and loose, very moist, silty sand.

Alluvium was encountered to the depths drilled below the upper soils. These soils are comprised of firm to very stiff, moist and very moist, silty clays and sandy silt, with interbeds of stiff, moist, lean clay, and medium dense and dense, moist and very moist, silty sand. Very dense, poorly graded sand with silt was encountered in the rotary wash boring beginning at a depth of approximately 35 feet below ground surface and continued until the boring was terminated at a depth of approximately 52.5 feet. Granular material was also encountered at the CPT probe location as indicated on the CPT-01.txt data sheets at approximately 32.5 feet below existing ground surface, which became impenetrable at a depth of approximately 40 feet.

Groundwater

Groundwater seepage was encountered at an approximate depth of 36 feet below the existing pavement in Boring 5. The historically highest groundwater depth at the site was estimated to be approximately 12 feet below existing grade.

The above estimate was derived in part by using, "Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Long Beach, Quadrangle," which is contained in California Geologic Survey's web page, "Seismic Hazard Evaluation of the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California," 1998, and National Geographic's, 2003 Topo.

Fluctuation of the groundwater level at the site could occur due to other variations such as changes in precipitation patterns, runoff, irrigation, basin management and other numerous factors.

Faulting and Seismicity

No known active or potentially active faults are shown on reviewed published maps as crossing, or projected to the property. The site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the potential for surface rupture on the subject site is considered to be low. However, it should be recognized that recent earthquakes have resulted in surface rupture where no faults had been previously mapped. Also, the property is located in an area of high regional seismicity and is likely to be subjected to strong ground shaking during the life of the project from nearby and distant faults, which is characteristic of all Southern California.

Seismic Conditions

Relatively nearby active faults of significance to the site include the following:

Fault Zone	Approximate Location	Earthquake Magnitude*
Newport-Inglewood (L.A. Basin)	2 miles southwest	7.2
Palos Verdes	8 miles west	7.3
Whittier	14.6 miles northeast	6.8

(*) Maximum probable moment magnitude, CGS 2008.

The accompanying fault and earthquake epicenter maps included in Appendix C present an overview.

Ground Motion

In order to assess the ground motion that could be induced at the site, the attenuation relationships of Boore, et al. (1997), Campbell and Bozorgnia (1994/1997), and Sadigh, et al. (1997), were used in our analyses in order to estimate an average Peak Ground Acceleration (PGA). The results of our analyses are presented in Appendix D and are summarized in the table below.

Boore, et al. (1997)	Campbell & Bozorgnia (1994/1997)	Sadigh, et al (1997)	Average PGA Value
0.6g	0.53g	0.51g	0.55g

Seismic Hazard Zones

This office has reviewed the Seismic Hazard Zones Map of the Long Beach Quadrangle prepared by the California Division of Mines and Geology, Released: March 25, 1999, with revisions through 1/13/06. The purpose of this map is to delineate areas that may be subject to liquefaction and/or landsliding during a strong seismic event. As defined, a liquefaction area is an area where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code 2693© would be required. An Earthquake-Induced Landslide area, is defined as an area where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693© would be required. According to the Seismic Hazard Zones Map of the Long Beach Quadrangle, the property is located within an area of study for earthquake-induced liquefaction, however, it is not located in an area of study for seismic-induced landsliding.

Liquefaction Potential and Earthquake-Induced Settlements

Liquefaction is the loss of shear strength experienced in saturated granular soils below the groundwater level during the event of a strong earthquake. The occurrence of this phenomena is dependent on several factors, including: the intensity and duration of ground shaking, particle size distribution, density of the soil, and elevation of the water table.

Based on the above referenced map, and the historic high groundwater levels within the vicinity of the site have been as shallow as 12 feet, a liquefaction potential and earthquake-induced settlement evaluation was performed using the data from Boring Log 5. The analysis was performed using the computer program EQLique&Settle"2", based on the historic and static groundwater levels, and a Site Magnitude Earthquake of 7.2 and a site PGA of 0.55g (see Appendix D). The boring log data was selected for analysis in lieu of the CPT data, as the probe was terminated at a depth of approximately 40 feet below ground surface due to impenetrable granular material as previously mentioned. Local groundwater was encountered at a depth of approximately 36 feet.

The results of our analysis, included in Appendix E, show a 6-ft. thick potentially liquefiable layer beginning at approximately 20 feet below existing ground surface, and a total cumulative earthquake-induced settlement above and below the historic high groundwater of 0.95 (say 1-inch). A maximum seismic differential settlement of 50 percent of the total earthquake-induced settlement, or 0.5 inch in a distance of 30 feet may also be expected. Due to the 20-ft. non-liquefiable cap, surface manifestation (Ishihara 1985) is not expected at the site. Also, due to the relatively low to normal total cyclic settlement, no special mitigation measures for the liquefiable layer and cyclic settlements is considered to be necessary. However, five feet of removal and recompaction or three feet below the bottom of proposed building foundations, whichever is deeper, is recommended to provide a uniformly strong cap at the site for bearing capacity and mitigation of any static settlements. The computer program used for analyses is based on the specific references listed at the end of Appendix E.

Lateral Spreading

Lateral spreading is generally caused by liquefaction of soils within gentle slopes. Since the property is relatively level, and there is no close free face toward which lateral spreading could occur, we judge that there is little risk of lateral spreading caused by an earthquake.

Lurching

Soil lurching is the relative displacement of adjacent land surfaces during an earthquake and refers to the rolling motion on the surface due to the passage of seismic surface wave. Effects of this nature are not considered significant on the subject site where the thickness of alluvium is considered uniform throughout the site.

Surface Rupture Hazard

Surface rupture is a break in the ground surface during or as a consequence of an earthquake. The potential for surface rupture on the subject site is considered low due to the absence of known active faults at the site.

Ground Shaking

Ground shaking, as a result of earthquakes, is a constant potential hazard throughout all Southern California. The relative potential for damage from this hazard is a function of the type and magnitude of the earthquake and the distance of the site from the event. Accordingly, the proposed structures should be designed and constructed in accordance with applicable portions of the 2007 CBC.

Seismic Information

Lateral forces due to seismic loading may be calculated utilizing the formulas presented in the 2007 California Building Code (CBC), based on the following:

Latitude = 33.8037° Longitude = -118.1433°	Short Period (0.2s)	One-Second Period (1.0s)
Site Class from Table 1613.5.2	D	
Spectral Accelerations from Figures 1613.5(3) & 1613.5(4) & USGS	S_s = 1.72(g)	S_1 = 0.659(g)
Site Coefficients from Figures 1613.5.3(1) & 1613.5.3(2) & USGS	F_A = 1.00	F_v = 1.50
Spectral Response Accelerations from Equations 16-37 & 16-38	S_{MS} = 1.72(g)	S_{M1} = 0.988(g)
Design Accelerations from Equations 16-39 & 16-40	S_{DS} = 1.147(g)	S_{D1} = 0.659(g)
Seismic Design Category from Section 1613.5.6 & Tables 1613.5.6(1) & 1613.5.6(2)	D	

Reference: U.S.Geological Survey, Earthquake Hazards Program, Seismic Design Values for Buildings, USGS Web Page, 2009.

It should be noted that conformance with the criteria listed above for seismic design does not constitute any kind of warranty, guarantee, or assurance that significant structural damage or permanent ground displacement will not occur if a maximum level seismic event occurs. The CBC provisions are generally intended to protect human life and catastrophic failure, and not to avoid all damage, since such design may be economically prohibitive.

Hydrocollapse Potential

The addition of water to four out of the five loaded consolidation test samples resulted in less than one percent soil collapse, and the fifth sample resulted in minor swell rather than collapse. Based upon these results, we judge that the potential for hydroconsolidation adversely affecting the proposed development is low.

Conclusions & Recommendations

The proposed carwash and retail buildings are considered to be feasible from a geotechnical engineering standpoint, subject to the conclusions and recommendations that follow.

In accordance with Section 111 of the Los Angeles County Building Code, it is the finding of this firm that the building site and the recommended site grading to be performed for the proposed development will be safe from a geotechnical engineering standpoint against hazard from future landsliding, settlement, and slippage, and will not have an adverse geologic effect on the stability of properties outside of the building site, provided all of our recommendations presented in this report are followed.

Detailed recommendations to be utilized in the design and construction of the proposed structures are presented in the following sections of this report.

The recommendations provided in this report are based upon observations made in the field, the results of laboratory tests on samples of the materials encountered during the subsurface exploration, our engineering analyses, and the past experience of this office.

Notification of Governing Authorities

Site grading and construction should be performed in accordance with the City of Long Beach Department of Building and Safety and the rules and regulations of those governmental agencies having jurisdiction over the subject construction. Permits should be obtained, and inspections made by the proper authorities as required.

Prior to initiating grading operations, a meeting should be conducted at the site with the owner's representative, the grading contractor, the grading inspector, and a representative of this company. The grading contractor is responsible to notify the required governmental agencies and the geotechnical engineer prior to initiating grading operations, and any time grading is resumed after an interruption.

Temporary Excavations & Shoring

Any unsurcharged temporary excavation without shoring may be cut at a maximum slope of 1.25h:1v (horizontal to vertical) to a maximum height of 8.0 feet. A visual inspection should be performed during the excavation by a representative of this firm. If any signs of sloughing or lateral movement are observed, immediate measures for support should be implemented by the contractor. Temporary construction cut slopes are suitable for a limited time duration, possibly four weeks maximum.

Shoring of vertical excavation walls should be provided where temporary slopes aren't feasible. The shoring system used should be designed by a registered civil engineer who is thoroughly familiar with design of shoring systems and their performance in the field. The design should accommodate support of adjacent soils, structures, streets, and appurtenances, and safeguarding personnel.

Permanent Slopes

Permanent cut and fill slopes should be no steeper than 2h:1v. Slopes should be planted with fast-growing, deep-rooted ground cover to reduce sloughing and erosion.

Site Grading and Compaction

Prior to commencing grading operations, all demolition debris should be removed from the site. Any remaining vegetation and soils containing organic matter should be stripped and also hauled from the site. Utility lines that are to be abandoned should be excavated and also removed from the site.

The existing artificial fill soils and firm native soils encountered during our exploration are not suitable in their present condition for pavement or structural support. This also includes soils disturbed during demolition and removal of old foundations and utility lines. These soils should be excavated to a minimum depth of five (5) feet below existing grade or a minimum of three (3) feet beneath the bottom of proposed building foundations, whichever is deeper.

The removals should extend throughout the building pad areas, extending a minimum horizontal distance of five (5) feet beyond the exterior building lines of the proposed structures, where feasible. The native soils exposed within the excavations bottoms should be scarified to a minimum depth of 6 inches, be moisture-conditioned as required (typically 3 to 5 percentage points above optimum moisture), and compacted to at least 90 percent of the maximum dry density of the materials as determined by the latest version of ASTM D 1557 laboratory compaction test procedure.

The excavated soils may be used for recompaction (engineered fill) provided they are free of root structures and deleterious debris, be moisture-conditioned as required (typically 3 to 5 percentage points above optimum moisture), spread in 8-inch thick loose lifts, and compacted to at least 90 percent relative compaction in accordance with the "General Specifications for Compacted Fill Soils."

Existing artificial fill soils within the proposed parking area should be excavated to a minimum depth of two feet below proposed subgrade elevation or the thickness of existing fill soils, whichever is deeper. The exposed native subgrade should be scarified to a minimum depth of 6 inches and be compacted to a minimum of 90 percent of the latest version of ASTM D 1557. The excavated soils should then be replaced in 8-inch thick loose lifts, be moisture-conditioned as required, and compacted to a minimum of 90 percent of the latest version of ASTM D 1557.

Unstable subsurface conditions are sometimes encountered when grading operations are conducted when the ground is wet. If areas of unstable subgrade are encountered during grading operations, stabilization will be required prior to placement of fill soils, construction of slabs or foundations. Stabilization may entail adequately reducing the moisture of the exposed soils and placement of a stabilization layer that may be comprised of compacted base material or crushed angular rock, geotextile fabrics or geogrid, etc. Unit prices should be obtained from the contractor in advance for this work.

The Geotechnical Engineer or his representative may require that additional shallow excavations be made periodically in the exposed bottom to determine whether sufficient removal has been made prior to replacement and compaction of fill material.

If import fill is required during the grading operation, the fill should be approved by the Geotechnical Engineer prior to transporting it to the site. Representative samples of soils planned to be imported to the site should be provided to the Geotechnical Engineer at least 48 hours before importing begins in order that they may be examined and evaluated as to their potential impact on project design and construction.

Utility Trenches

Backfill of utilities within right-of-ways should be placed in strict conformance with the requirements of the governing agencies.

Following placement of utility lines within private property, the space under and around the line (bedding and shading zone) should be backfilled with clean sand or approved granular soil, having a minimum Sand Equivalent value of 30, to approximately one foot over the pipe. The sand bedding and shading should be properly compacted below, above and around the pipe, manually if practicable. No flooding or jetting shall be permitted for compaction of the bedding and shading material.

Backfill over the bedding and shading material should be mechanically compacted to at least 90 percent of the maximum density obtainable by the latest version of ASTM D 1557 method. Jetting or flooding of the backfill should not be permitted.

Utility trench backfills should be observed and tested during backfill operations as the work progresses. If testing of a backfill is performed after completion, without observing the backfill operations, then only the test results at the test locations can be given, and no guarantee of the condition of the remaining backfill can be provided.

Spread Footing Foundations

Following completion of the grading operation and field density testing, the proposed one-story structures should be supported on a conventional continuous foundation system excavated to a minimum width of 12 inches. The foundation trenches should be excavated to a minimum depth of 24 inches below lowest adjacent grade into compacted/engineered fill soils. Continuous foundations having the preceding minimum dimensions should be designed utilizing an allowable bearing pressure of 1800 pounds per square foot (psf). This bearing pressure reflects a reasonable reduction in order to limit potential static settlements to tolerable values.

The proposed structures should be designed to accommodate total foundation static and earthquake-induced settlement combined of approximately 2.0 inches. A combined static and seismic differential settlement of 1.0 inch may be used for design along a 30-foot span of continuous footings. These estimates are based upon the foundations being designed and constructed in accordance with the recommendations contained in this report.

Resistance to lateral loadings may be provided by a combination of passive pressure on the footing walls and lateral sliding resistance acting on the base of the footings that are in contact with compacted/engineered fill soils. Passive earth pressure should be computed as an equivalent fluid unit weight of 270 pounds per cubic foot (pcf), to a maximum value of 2700 psf. The lateral sliding resistance may be 130 pounds per square foot (psf) to be multiplied by the contact area, to a maximum of one-half the dead load.

The allowable soil pressures may be increased one-third for combinations of vertical and horizontal wind or seismic forces where permitted by the Building Code.

Foundations should be stepped as necessary to produce level tops and bottoms. Foundations should be deepened to provide a minimum of H/3 feet of horizontal confinement between the bottom outside edge of the foundation and the face of the nearest slope, where H equals the overall height of the slope. The foundation setback should be a minimum of ten feet, but need not be greater than forty feet.

All foundations within the influence zone of underground lines or associated backfills, shall be deepened below a one horizontal to one vertical plane projected from the invert of the underground line or the native soil/backfill contact to ground surface. Also, for adjacent footings constructed at different levels, the bottom of the deeper footing should be below a 1h:1v line drawn down from the bottom of the shallower footing.

The following concrete footing reinforcement recommendations are minimums as based upon the medium expansion potential of the tested on-site soils. Continuous foundations should be reinforced with a minimum of two (2) #4 deformed reinforcing bars. One (1) bar shall be placed near the bottom and one (1) bar near the top. A continuous foundation system is recommended throughout the structures.

Foundation excavations should be observed by a representative of this company prior to the placement of reinforcing steel to verify uniform soil conditions and conformance with the recommendations in this report.

Slabs-on-Grade

The concrete thickness and reinforcement recommendations provided below are minimums as based upon the medium expansion potential of the tested on-site soils. The Structural Engineer for the project may need to address other factors which may require enhancement of these recommendations.

Concrete slabs-on-grade should be a minimum of 5 inches thick and be reinforced with a minimum of #4 Bars, placed 18 inches on center in both directions, and positioned in the center of the slab upon concrete dobies.

Slabs should be underlain with a capillary moisture break consisting of a minimum of 4 inches of clean sand and an impermeable membrane moisture vapor barrier (10 mil polyethylene or equivalent). The membrane should be encased within the sand layer to protect it during construction.

Due to the expansive characteristics of the on-site (prevailing) soils, it is recommended that the upper 12 inches of the subgrade soils comprising the building pads should be confirmed to have a moisture content between 3 and 5 percent above the optimum moisture content of the material tested prior to the placement of the sand and visqueen section.

Joints should be utilized within the slab to induce and control cracking. Control joints should be spaced a maximum distance of 12.0 feet in each direction, and should be cut to a depth equivalent to ¼ of the thickness of the slab. Joints must be cut as soon as the concrete surface is firm enough not to be torn or damaged by the blade, and before random shrinkage cracks can form in the concrete.

The concrete contractor should provide the mix design to the owner, and should place, finish, and provide means for the concrete to cure in accordance with PCA and ACI recommended practices. Some cracking is normal due to drying and shrinkage as the concrete cures. Some of the parameters contributing to concrete cracking are high water cement ratio, small nominal aggregate size, and adverse conditions during placement and/or curing such as rapid moisture loss due to high temperature or windy weather. Potential shrinkage cracking may be reduced by using low slump concrete.

Retaining Structures

Retaining structures constructed at the site should be designed to resist active lateral earth pressures plus surcharge loadings. Cantilever retaining structures should be designed to resist pressures calculated using Equivalent-Fluid unit weights presented in the following table.

Surface Slope of Retained Material (Horizontal to Vertical)	Equivalent-Fluid Unit Weight (Lbs./Cu.Ft.)
Level	50
2 to 1	67

Retaining walls that are restrained from horizontal movement should be designed using an equivalent fluid pressure of 71 pcf.

Retaining walls should be supported on spread footing foundations designed in conformance with the recommendations presented in the preceding portions of this report. Retaining walls should structurally be designed to have a minimum factor of safety of 1.5 in resisting sliding and overturning.

Retaining walls should be waterproofed by a qualified contractor using a proven product to prevent seepage through the walls.

Retaining walls should be backdrained to collect accumulated moisture and prevent hydrostatic pressures from accumulating. One of the three options presented below should be used for design. Options 1 and 2 present backdrainage alternatives. Option 3 assumes that design will incorporate hydrostatic pressures.

Option 1:

A perforated 4-inch diameter Schedule 40, PVC pipe or equivalent, should be placed at the base of the proposed backfill to collect any accumulated moisture. The drain pipe should be encased in a minimum of one cubic foot of clean, free-draining crushed rock or gravel, per lineal foot of pipe. The perforations should be pointing down and out to the side. The crushed rock should be encapsulated in a geofabric (e.g. Mirafi 140NL or equivalent). The geofabric should be laid down prior to the placement of the drain pipe and crushed rock and should run the entire length of the proposed backfill. The width of the geofabric should be of such size, so that when complete, it encapsulates the crushed rock and drain pipe in the form of a burrito. The pipe should be sloped to drain to appropriate receptacles by gravity. The remainder of the backfill should be comprised of compacted clean sand, having a minimum width of 1.0 foot on top of the geofabric, extending vertically to within two feet of proposed final grade. The remaining two feet of the backfill should be comprised of fill material commensurate with the on-site soils compacted in accordance with the grading recommendations contained herein.

Option 2:

The retaining wall backdrainage system may consist of a geocomposite (e.g. Greenstreak or Miradrain) placed against the wall which allows accumulated moisture to flow to an encased 4-inch diameter perforated drain pipe placed at the base of the backfill. The drain pipe should be sloped to drain to appropriate receptacles by gravity. Native soils may be then be placed as compacted/engineered fill behind the wall in accordance with the grading recommendations contained herein. Care should be taken during the backfilling operation so as not to damage the backdrainage system or the wall.

Option 3:

If a daylight point for the subdrain does not exist, or if the owner chooses not to place the subdrain pipe behind the walls, undrained conditions will apply and an equivalent fluid pressure of 87 pcf should be used for design to account for hydrostatic pressures on cantilever walls. For restrained walls, this value should be 98 pcf. Native soils may then be placed as compacted/engineered fill behind the walls in accordance with the grading recommendations contained herein. Care should be taken during the backfilling operation so as not to damage the wall.

All drainage from the drain pipes should be transferred to an approved drainage area via non-erosive devices.

Retaining walls should be backfilled prior to building on, as the walls will yield slightly during the backfilling operation.

To prevent the build up of lateral soil pressures in excess of the recommended design pressures, overloading the walls should be avoided. This can be accomplished by placement of the backfill above a 45 degree plane, projected upward from the base of the wall, in lifts not exceeding eight inches in loose depth, and compacting it with hand-operated equipment or small self-propelled vibrating plates.

Exterior Slabs-on-Grade

The following recommendations are provided with the intent of reducing the risk of substantial differential movement and cracking of exterior concrete slabs-on-grade, and supplement the previous recommendations. Areas to receive concrete slabs-on-grade should be graded in accordance with the Site Grading and Compaction recommendations contained in this report.

Exterior slabs that are immediately adjacent to the proposed structures should be structurally continuous with the building by the use of reinforcing steel, or a separation should be provided. If the latter is chosen, an elastomer should be placed between the building foundation and exterior flatwork to allow some movement at the joint.

In order to minimize the potential for differential movement of the outer edge of exterior slabs, cut-off walls should be provided having a minimum width of 8 inches and a minimum depth of 12 inches below lowest adjacent grade. Cut-off walls should be reinforced with a minimum of one #4 bar, top and bottom.

Areas for exterior concrete slabs-on-grade should be prepared following the grading operation by scarifying to a depth of at least 6 inches, moisture-conditioning to wet of optimum, and compacting to a minimum of 90 percent of the latest version of ASTM D 1557. A 4-inch thick layer of clean sand or crushed miscellaneous base (CMB) should then be spread and moisture-conditioned as necessary, and compacted to a minimum of 90 percent of the latest version of ASTM D 1557. The sand or aggregate base material should be smooth and non-yielding.

Owing to the expansiveness of the on-site subgrade soils, exterior concrete slabs-on-grade should be a minimum of 5 inches thick and be reinforced with a minimum of #4 Bars, placed 18 inches on center in both directions, and positioned in the center of the slab. The reinforcement should be supported upon concrete dobies in order to ensure proper positioning during the slab pour.

Preliminary Pavement Section Design

Pavement design is a direct function of the amount and weight of traffic over time represented by the Traffic Index (TI) and the strength of the subgrade soil represented by the R-Value. The asphaltic concrete (AC) pavement design sections presented below are based on an estimated (not tested) R-Value of 30 of the subgrade soils, and estimated TI's that were not based upon a traffic survey or engineering study. The TI loading for pavement section design was assumed to range between 4.0 and 6.0 based on planned usage and usual engineering practice. If specific TI information for the site becomes available and is different from what we used, WL should be notified so that we may review, and if necessary, amend the following design sections. Final pavement section designs shall be determined through testing. The R-Value of the upper two feet of the finished subgrade soils comprising the parking and drive areas shall be determined when the recommended grading operation nears completion.

Traffic Utilization	Traffic Index	Minimum A.C. Thickness	Minimum Base Thickness
Passenger Car Parking Areas	4.0	3"	4"
Access Drive	6.0	4"	6½"

The top 6 inches of the subgrade to support the Base should be compacted to a minimum of 95 percent of the latest version of ASTM D 1557. The finished subgrade should be smooth and unyielding. The recommended Base section should then be spread and moisture-conditioned as necessary, and compacted to a minimum of 95 percent of the latest version of ASTM D 1557C. The aggregate Base should also be smooth and unyielding.

The specified Base and asphaltic concrete (AC) thickness should be evenly spread upon the subgrade or Base, respectively, to such a depth that, after rolling, it will be the specified cross-section. The AC should be rolled to achieve a minimum compaction of 95 percent.

Aggregate Base should comply with applicable sections of the Standard Specifications for Public Works Construction, latest edition, including but not limited to Section 200-2-Untreated Base Materials. Base should be Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB). If CMB is used, the R-Value of this material should be tested to verify it has an R-Value of at least 78, if not, the above design sections should be revised.

Soil Corrosivity

It is recommended that chemical tests be performed on the soils that will be in contact with the proposed improvements to aid in the evaluation of soil corrosion potential and the attack on concrete by sulfates and soil corrosivity effects to metals. The tests should be performed when the recommended grading operation nears completion for the building pads and prior to foundation construction. In this way the material to be tested and the results obtained, will be based upon the as-graded soil conditions. Pending the results of the tests, Type V cement should be specified for concrete that will be in contact with the earth.

If any proposed subsurface utilities have metallic elements associated with them, it is recommended that the services of a qualified corrosion specialist be contracted by the owner of the property to evaluate soil corrosion potential at the site. No corrosion protection measures are required for buried utility lines comprised of vitrified clay, PVC, or other flexible plastic piping.

Site Surface Drainage

Control of site drainage is important for the overall performance of the proposed project. The project civil engineers should make appropriate recommendations with regard to drainage and erosion control.

Surface water should be diverted away from slopes and foundations. Roofs should be provided with gutters or appropriate drainage devices. All surface run-off should be diverted to appropriate discharge facilities in a controlled manner via non-erosive devices. Roof and surface drains should be maintained entirely separate from retaining wall backdrains.

Planters to be placed adjacent to the proposed buildings, and flatwork areas, shall be properly sealed to prevent intrusion of moisture into these areas. The excess irrigation water from the planter should be collected in a drain and discharged appropriately, or be allowed to disperse over the top of the planter in areas where ample drainage is available to disperse it.

Structures constructed in expansive soil areas typically perform best when subgrade moisture conditions are uniform. A properly designed, installed, adjusted and maintained automatic irrigation system with drainage appropriately conducted away from the structures and flatwork areas can achieve this desired effect.

Closure

This report is prepared for the specific use of Long Beach Gateway LLC, for the proposed project described herein. Findings in this report are valid as of this date; however, changes in conditions of a property can occur due to the passage of time, whether they are due to natural processes or works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review after a period of one year.

Our services consist of professional opinions and conclusions developed by a consulting California registered Geotechnical Engineer. The warranty or guarantee made by the consultant in connection with the services performed for this project is that such services are performed with the care and skill ordinarily exercised by members of the same profession practicing under similar conditions at the same time and in the same or a similar locality. No other guarantee or warranty, either expressed or implied, is made or attempted by rendition of consulting services or by furnishing written reports of the findings.

The information and recommendations contained in this report are based upon the assumption that the soil conditions do not deviate from those disclosed in our exploratory borings and CPT sounding. If any variations or undesirable conditions are encountered during the grading operation or construction, or if the proposed development will differ from that planned at the present time, WL should be notified so that supplemental recommendations can be provided, if warranted.

This report is issued with the understanding that it is the responsibility of the owner or of his or her representative, to ensure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plans and that the necessary steps are taken to see that the Contractors and Subcontractors carry out such recommendations in the field.

This report is subject to review by the controlling authorities for the project.

Our scope of work did not include evaluation of potential hazardous material contamination of soil or ground water.

Supplemental Services

During the recommended grading operation and construction, we should observe the conditions encountered in excavations, and if necessary, modify our recommendations.

We should observe excavations for proposed foundations prior to placement of forms or reinforcement. Our services during foundation construction are limited to observation of soil conditions and depth of the excavations.

Our services do not include observation or approval of steel, concrete, or asphalt; nor do they include establishing or verifying construction lines and grades. These services should be performed by the appropriate parties.

Our supplemental services are performed on an as-requested basis, and WL cannot accept responsibility for items that we are not notified to observe or test. These supplemental services are in addition to this geotechnical engineering report, and will be billed for on a time and materials basis in accordance with our Professional Fee Schedule and our General & Commercial Terms and Conditions.

Maintenance

Periodic land maintenance will be required. Surface and subsurface drainage facilities must be checked frequently to assure that they are clean and working properly. Any damage to the drainage facilities must be repaired immediately.

General Specifications for Compacted Fill Soils

Preparation

The existing artificial fill and firm native soils should be removed to a depth as specified in the body of this report, under the observation of a representative of WL to expose subgrade competent to support the engineered fill. After the foundation for the engineered fill has been exposed, it shall be scarified until it is uniform and free from large clods, moisture-conditioned where necessary and compacted, as specified in the body of this report, in accordance with the latest version of ASTM D 1557.

Materials

On-site soils may be used for the engineered fill, or imported fill materials shall consist of materials approved by the Geotechnical Engineer, and may be obtained from the excavation of banks, borrow pits or any other approved source. The materials used should be free of organic matter and other deleterious substances and should not contain rocks or lumps greater than six inches in maximum dimension.

Placing, Spreading and Compacting Fill Materials

A. The selected fill material should be placed in layers that when compacted shall not exceed six inches in thickness. Each layer should be spread evenly and thoroughly mixed during the spreading to attain uniformity of material and moisture of each layer.

B. Where the moisture content of the fill material is below the limits specified by the Geotechnical Engineer, water should be added until the moisture content is satisfactory to attain thorough bonding and compaction.

C. Where the moisture content of the fill material is above satisfactory limits, the fill materials should be aerated, blended or dried until the moisture content is satisfactory.

D. After each layer has been placed, mixed and evenly spread, it should be compacted as specified in the body of this report. Compaction should be by sheepsfoot roller, multi-wheel pneumatic tire roller, or other types of acceptable rollers. Compaction equipment should be selected by the contractor and be of such design that they will be able to compact the fill to the specified density.

Compaction should be accomplished while the moisture content of the fill material is within the compactable range. Compaction of each layer should be accomplished by rolling the entire area with sufficient trips to attain the desired density. The final surface of areas to receive slabs-on-grade should be rolled to a dense, smooth, unyielding surface.

E. The outside of all fill slopes should be compacted by means of sheepsfoot rollers or other suitable equipment. Compaction operations should be continued until the outer face of the slope is compacted to a minimum of 90 percent relative compaction. Compacting of the slopes should be done progressively in increments not to exceed 2.0 feet as the fill is brought to grade.

F. Field density tests should be performed by a representative of this company. Density tests should be performed at vertical intervals not to exceed two feet. Where sheepsfoot rollers are used, the soils may be disturbed to a depth of several inches. Consequently, density readings should be taken in the compacted material below the disturbed surface. When these readings indicate the density of any layer of fill is below the required density, the fill should be reworked until the required density has been obtained.

Observation

A representative of WL should observe all filling and compacting operations to verify that the fill is consistent and in compliance with the recommendations.

Seasonal Limitations

No fill materials should be placed, spread or rolled during unfavorable weather conditions. When work is interrupted by heavy rains, fill operations should not be resumed until field tests performed by a representative of the Geotechnical Engineer indicate that the moisture content and density of the fill are as previously specified.

APPENDIX A

BORING LOGS
&
CPT-01.txt DATA SHEETS

KEY TO EXPLORATORY LOGS

MAJOR DIVISIONS		USCS	DESCRIPTION	MAJOR DIVISIONS		USCS	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	FINE GRAINED SOILS	SILTS AND CLAYS LL <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
		GP	Poorly-graded gravels or gravel-sand mixture. Little or no fines			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays silty clays, lean clays
		GM	Silty gravels, gravel-sand-silt mixtures			OL	Organic silts and organic silt-clays of low plasticity
		GC	Clayey gravels, gravel-sand-clay mixtures				
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sand, little or no fines		SILTS AND CLAYS LL >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		SP	Poorly-graded sands or gravelly sands, little or no fines			CH	Fat clays, inorganic clays of high plasticity
		SM	Silty sands, poorly graded sand-silt mixtures			OH	Organic clays of medium to high plasticity
		SC	Clayey sand, poorly graded sand-clay mixtures	HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

NOTES: (1) Dual USCS symbol, such as "SP-SM" or "SP-SC", denotes 5 to 12% of minor constituent (i.e. "M" for silt or "C" for clay), except "SC-SM" which denotes "silty, clayey sand".

(2) Dual symbols, such as "SM/ML" and "SC/CL", denote borderline coarse grained/fine-grained soils "silty sand-to-sandy silt" and "clayey sand-to-sandy clay", respectively.

RELATIVE DENSITY	SPT* (# blows/ft)	RELATIVE DENSITY(%)
Very loose	<4	0 – 15
Loose	5 – 10	15 – 35
Medium Dense	11 – 30	35 – 65
Dense	31 – 50	65 – 85
Very Dense	>51	85 – 100

CONSISTENCY	SPT* (# blows/ft)	UNCONFINED COMPRESSIVE STRENGTH (tsf)
Soft	0 – 4	0.25 – 0.5
Firm	5 – 8	0.5 – 1.0
Stiff	9 – 15	1.0 – 2.0
Very Stiff	16 – 30	2.0 – 4.0
Hard	>31	>4

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE THE SPT SAMPLER

SAMPLE TYPE:

CD = California Drive (or California Modified)(3 in. O.D./2.416 in. I.D.)
R = Relatively Undisturbed Ring Sample (3.25 in. O.D./2.42 in. I.D.)
SP = Standard Sampler for SPT Test (2 in. O.D./1-3/8 in. constant I.D.)*
SPT = Standard Penetration Test with SP Sampler (using Standard Hammer/Drop)**
**Where: Standard Hammer/Drop = 140lb/30"
WL = Western Laboratories Hammer/Drop (32lb/30")
B = Bulk Sample
NR = No Recovery

*References – Terzaghi & Peck (2nd edition), and "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluation" by Seed, Tokimatsu, Harder & Chung (1985)

LABORATORY TESTS.

DS = Direct Shear
CN = Consolidation
EI = Expansion Index
AL = Atterberg Limits
MD = Maximum Density
RV = Resistance Value
COR = Corrosivity
SE = Sand Equivalent Value
UC = Unconfined Compression
HYD = Hydrometer Analysis
#200 = No. 200 Sieve Wash

Western Laboratories

Boring 1

Sheet 1 of 1

Work Order: 09-4329
Date Drilled: 01-06-10

Client: LONG BEACH GATEWAY LLC
Project: *Proposed Car Wash & Retail Buildings*

Depth (ft)	Sample Type	Lab Tests	Blows/Foot*	DESCRIPTION 140-lb Hammer w/30" Drop 8" Diameter Hollow Stem Auger		Dry Unit Weight (pcf)	Moisture Content (%/Dry Wt.)	Notes
0				0.0-0.25	3" Asphaltic Concrete			
	R		15	0.25-2.0	ARTIFICIAL FILL-SILTY CLAY (CL-ML), stiff, moist, dark brown	96	25.5	
				2.0-9.0				
	R		11		NATIVE SOIL SILTY CLAY (CL-ML), firm, moist, brown with light brown mottling	109	17.3	
5	R		28		-very stiff @ 5'	114	15.5	
				9.0-20.0	SILTY SAND (SM), medium dense, moist, brown			
10	R	CN	55			115	14.1	
15	R	CN	67		-dense @ 15'	116	13.2	
20	R		27	20.0-26.5	SILTY CLAY (CL-ML), very stiff, moist, gray brown	108	20.0	
25	R		30			106	23.0	
					Boring terminated and backfilled Groundwater not encountered			
30								
35								

Boring 2

Sheet 1 of 1

Work Order: 09-4329
Date Drilled: 01-06-10

Client: LONG BEACH GATEWAY LLC
Project: *Proposed Car Wash & Retail Buildings*

Depth (ft)	Sample Type	Lab Tests	Blows/Foot*	DESCRIPTION 140-lb Hammer w/30" Drop 8" Diameter Hollow Stem Auger		Dry Unit Weight (pcf)	Moisture Content (%/Dry Wt.)	Notes
0				0.0-0.3	3" Asphaltic Concrete over 1" Base			
	SPT		9	0.3-2.0	ARTIFICIAL FILL-SILTY SAND (SM),		16.3	
				2.0-14.0	loose, very moist, dark brown			
	SPT		10		NATIVE SOIL		15.1	
					SANDY SILT (ML), stiff, moist, light gray,			
5	SPT		7		with brown mottling		15.9	
					-firm @ 5'			
	SPT		19		-very stiff, very moist @ 7'		23.5	
10	SPT		23				25.9	
				14.0-20.0	SILTY SAND (SM), medium dense, very			
15	SPT		25		moist, rusty brown, fine sand		19.1	
20	SPT		14	20.0-31.5	SANDY SILT (ML), stiff, very moist,		31.4	
					light gray with brown mottling			
25	SPT		13				25.0	
30	SPT		15				29.6	
				31.5-36.5	SILTY SAND (SM), dense, very moist,			
					blue gray, fine sand			
35	SPT		38				24.4	
					Boring terminated and backfilled Groundwater not encountered			

Western Laboratories

Boring 3

Sheet 1 of 1

Work Order: 09-4329
Date Drilled: 01-06-10

Client: LONG BEACH GATEWAY LLC
Project: *Proposed Car Wash & Retail Buildings*

Depth (ft)	Sample Type	Lab Tests	Blows/Foot*	DESCRIPTION 140-lb Hammer w/30" Drop 8" Diameter Hollow Stem Auger		Dry Unit Weight (pcf)	Moisture Content (%/Dry Wt.)	Notes
0				0.0-0.4	4" Asphaltic Concrete over 1" Base			
	R		20	0.4-3.0	ARTIFICIAL FILL-SILTY CLAY (CL-ML), stiff, moist, black	112	16.0	
	R		11	3.0-10.0	NATIVE SOIL SILTY CLAY (CL-ML), firm, moist, brown with light brown mottling	108	16.5	
5	R		31		-very stiff @ 5'	109	16.9	
	R	DS	26			106	18.9	
10	R	CN	66	10.0-20.0	SILTY SAND (SM), medium dense, moist, brown, with light brown mottling	128	9.2	
15	R	CN	86		-dense @ 15'	114	15.3	
20	R	CN	28	20.0-26.5	SILTY CLAY (CL-ML), very stiff, moist, dark gray	91	29.5	
25	R		30			96	26.6	
					Boring terminated and backfilled Groundwater not encountered			
30								
35								

Boring 4

Sheet 1 of 1

Work Order: 09-4329
Date Drilled: 01-06-10

Client: LONG BEACH GATEWAY LLC
Project: *Proposed Car Wash & Retail Buildings*

Depth (ft)	Sample Type	Lab Tests	Blows/Foot*	DESCRIPTION 140-lb Hammer w/30" Drop 8" Diameter Hollow Stem Auger		Dry Unit Weight (pcf)	Moisture Content (%/Dry Wt.)	Notes
0				0.0-0.4	4" Asphaltic Concrete over 1" Base			
	SPT		7	0.4-2.0	ARTIFICIAL FILL-SILTY CLAY (CL-ML),		15.6	
				2.0-11.0	firm, moist, black			
	SPT		8		NATIVE SOIL		18.9	
					SILTY CLAY (CL-ML), firm, moist, dark			
5	B	EI			brown		23.6	EI=77
	SPT		10		-stiff @ 7'		22.4	
10	SPT		14				16.9	
				11.0-14.0	SANDY SILT (ML), stiff, moist, gray,			
					with brown mottling			
				14.0-18.0	SILTY SAND (SM), medium dense, very			
15	SPT		23		moist, brown, fine sand		20.9	
				18.0-23.0	SANDY SILT (ML), very stiff, very moist,			
					gray with brown mottling			
20	SPT		20				33.1	
				23.0-28.0	SILTY CLAY (CL-ML), stiff, moist, dark			
					gray			
25	SPT		12				30.2	
				28.0-31.5	SILTY SAND (SM), medium dense, very			
					moist, gray brown			
30	SPT		18				23.4	
					Boring terminated and backfilled			
					Groundwater not encountered			
35								

Western Laboratories

BORING 5

Sheet 1 of 2

Work Order: 09-4329
Date Drilled: 01-07-10

Client: LONG BEACH GATEWAY LLC
Project: *Proposed Car Wash & Retail Buildings*

Depth (ft)	Sample Type	Lab Tests	Blows/Foot*	DESCRIPTION 140lb Hammer w/30" Drop 4" Diameter Rotary Wash		Dry Unit Weight (pcf)	Moisture Content (%/Dry Wt.)	Lab Tests Results
0				0.0-0.3	3" Asphaltic Concrete over 1" Base			
	SPT		3	0.3-2.0	ARTIFICIAL FILL-SILTY CLAY (CL-ML),		25.2	
				2.0-8.0	soft, moist, dark brown			
	SPT		7		NATIVE SOIL		21.6	
					SILTY CLAY (CL-ML), firm, moist, dark			
5					brown			
	SPT		21		-very stiff @ 6'		24.3	
				8.0-10.0	SILTY SAND (SM), medium dense, very			
	SPT		28		moist, light brown, fine sand		19.8	
10				10.0-14.0	SILTY CLAY (CL-ML), very stiff, moist,			
					gray brown			
	SPT		20				24.5	
				14.0-16.0	SILTY SAND (SM), medium dense, very			
15	SPT	#200	24		moist, brown		22.9	40
				16.0-26.0	SANDY SILT (ML), very stiff, very moist,			
					grayish brown			
	SPT	AL	18				32.6	PI = 10
20								
	SPT		16				30.3	
	SPT		14		-stiff @ 24'		26.9	
25								
				26.0-35.0	Lean CLAY (CL), stiff, moist, gray brown			
	SPT	#200	14				24.6	88
30	SPT	AL	12				23.5	PI = 9
	SPT		13				23.2	
35				35.0-52.5	Poorly Graded SAND with SILT (SP-SM),			
	SPT		48		very dense, saturated, dk gray, fine sand		27.1	
					-seeping water @ 36'			

BORING 5

Sheet 2 of 2

Work Order: 09-4329
Date Drilled: 01-07-10

Client: LONG BEACH GATEWAY LLC
Project: *Proposed Car Wash & Retail Buildings*

Depth (ft)	Sample Type	Lab Tests	Blows/Foot*	DESCRIPTION 140lb Hammer w/30" Drop 4" Diameter Rotary Wash	Dry Unit Weight (pcf)	Moisture Content (%/Dry Wt.)	Lab Tests Results
	SPT		55	Continued (Poorly Graded SAND with SILT (SP-SM), very dense, saturated, dk gray, fine sand)		26.5	
40							
	SPT	#200	78			26.2	7
45	SPT		83			25.1	
	SPT		87			26.0	
50							
	SPT		95			26.0	
				Boring terminated and backfilled Groundwater encountered @ 36'			
55							
60							
65							
70							
75							

Middle Earth Geo Testing, Inc.
954 North Lemon
Orange, CA 92867

January 7, 2010

Ken Mansir
Western Labs
4030 Spencer Street
Suite 101
Torrance, CA 90503

Project Name: CPT Testing at "Long Beach Gateway LLC"
Project No.: 4329

Dear Mr. Mansir,

Enclosed please find copies of the cone penetrometer testing (CPT) data for the above referenced project along with a copy of the corresponding invoice.

The cone penetrometer testing conducted for this project consisted of pushing an instrumented cone-tipped probed into the ground while simultaneously recording the resistance to penetration at the cone tip and along the friction sleeve.

The cone penetrometer testing described in the report was conducted in general accordance with the current ASTM specifications (ASTM D 5778-95 and D3441-94) using an electronic cone penetrometer.

The CPT equipment operated by Middle Earth Geo Testing, Inc. (MEGT) consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to continuously push the cone and rods into the soil at a rate of 20-mm per second (approximately for feet per minute) while the cone tip resistance and sleeve friction resistance are recorded every 50-mm (approximately two inches) and stored in digital form. A specially designed all wheel drive 25-ton truck provides the required reaction weight for pushing the cone assembly and is also used to transport and house the test equipment.

The cone penetrometer assembly used for the project consists of a conical tip and a cylindrical friction sleeve. The conical tip has a 60° apex angle and a diameter of 35.6mm (1.40-inch) resulting in a projected cross-sectional area of 10 cm^2 (1.5 square inches). The cylindrical friction sleeve is 133-mm (5.25-inch) in length and has an outside diameter of 35.8-mm (1.41-inch), resulting in a surface area of 150 cm^2 (23 square inches)

The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurement of cone tip and friction sleeve resistance during penetration. Continuous electric signals from the strain gauges are transmitted by a shielded cable in the sounding rods to the PC-based data acquisition hardware in the CPT truck. The sounding log is also displayed on a monitor. The CPT data processing is performed using the truck mounted computer based data acquisition and presentation system. The computer generated graphical logs include cone resistance, friction resistance, friction ratio, and pore pressure ratio versus depth at a user selectable scale.

Soil Behavior type interpretations are based on the following reference: Robertson, P.K. and Campanella, R.C., 1989, "Guidelines for Geotechnical Design using the Cone Penetrometer Test and CPT with Pore Pressure Measurement." Soil Mechanics series NO. 120, Civil Engineering Department, University of British Columbia, Vancouver, B.C., V6T 1Z4, September 1989.

Interpretations and plotting has been done using MEGT's proprietary data interpretation and presentation software. It is important to note that the data is not averaged. All interpretations are point interpretations at the corresponding depth listed.

It is also important to note that the soil behavior type correlations are based on a combination of theory, field research, and research performed under laboratory conditions, and literature review. The information presented in the tabulated and /or graphical logs should, therefore, be viewed as a guideline rather than as precise measurements.

Some care is recommended when using the soil behavior type interpretations. If a tabulation depth happens to fall on a soil layer interface or a seam of soil differing from the rest of the layer, the tabulated data can be misleading. The solution to this problem is the proper use of the graphical CPT logs. The tip and sleeve penetration resistance logs are the primary source of profile description; the soil behavior type logs are supplemental. The graphical logs of the tip and sleeve resistance should be examined and layer boundaries delineated in accordance with the project requirements. The soil behavior type interpretations are only representative of the response of the soil to the large shear deformations imposed during cone penetration. This is not necessarily a prediction of the grain size distribution. How, it has been found that the interpreted soil behavior types generally agree will with the soil types defined in accordance with the grain size distribution methods such as used in the Unified Soil Classification System.

Limitations

Middle Earth Geo Testing, Inc. (MEGT) presents the attached data in accordance with ASTM Standards
D5778-95 and D3441-94 and generally accepted cone penetrometer testing practices and standards. The attached data further relates only to the specific project and location discussed in the data. Judgment may be required to verify the CPT soil behavior interpretations.

The "Client" may distribute this data or excerpts therefore provided the following statement is prominently displayed and included with the distribution:

"Neither CLIENT nor MEGT make any guarantee or warranty, express or implied, regarding this data. THE USE OF THIS INFORMATION SHALL BE AT THE USER'S SOLE RISK REGARDLESS OF ANY FAULT OR NEGLIGENCE OF THE CLIENT OR MEG."

Please feel free to call if you have any questions.

Respectfully submitted,

Robert Hancock

CPT Operations Manager

Middle Earth Geo Testing, Inc.

:RC\Enclosures

Data File:CPT-01
Operator:BH
Cone ID:DSG1047
Customer:Western Labs

1/6/2010 8:51:51 AM
Location:Long Beach Gateway LLC
Job Number:4329
Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Zone	Soil Behavior Type UBC-1983	SPT N* 60% Hammer
0.16	0.86	1.0666	123.406	0	<out of range>	0
0.33	0.68	1.4486	214.333	0	<out of range>	0
0.49	39.25	1.3644	3.476	5	clayey silt to silty clay	19
0.66	33.54	1.5197	4.531	4	silty clay to clay	21
0.82	24.47	1.1528	4.710	3	clay	23
0.98	20.08	0.8778	4.372	3	clay	19
1.15	14.09	0.6636	4.710	3	clay	13
1.31	11.58	0.5498	4.746	3	clay	11
1.48	8.93	0.4743	5.310	3	clay	9
1.64	8.37	0.4232	5.058	3	clay	8
1.80	6.64	0.3320	4.997	3	clay	6
1.97	5.99	0.3366	5.619	3	clay	6
2.13	6.17	0.3918	6.354	3	clay	6
2.30	8.42	0.4994	5.929	3	clay	8
2.46	10.41	0.6149	5.905	3	clay	10
2.62	11.44	0.6614	5.783	3	clay	11
2.79	13.83	0.7926	5.730	3	clay	13
2.95	18.39	1.0432	5.673	3	clay	18
3.12	21.24	1.1881	5.595	3	clay	20
3.28	22.25	1.2520	5.626	3	clay	21
3.44	23.98	1.2690	5.292	3	clay	23
3.61	24.13	1.3129	5.442	3	clay	23
3.77	22.19	1.2123	5.464	3	clay	21
3.94	21.14	1.2186	5.763	3	clay	20
4.10	19.43	1.2574	6.471	3	clay	19
4.27	19.46	1.2730	6.543	3	clay	19
4.43	18.42	1.1882	6.449	3	clay	18
4.59	18.53	1.1522	6.219	3	clay	18
4.76	19.47	1.0463	5.375	3	clay	19
4.92	18.60	0.9043	4.861	3	clay	18
5.09	19.28	0.8272	4.290	3	clay	18
5.25	18.58	0.9736	5.241	3	clay	18
5.41	17.02	0.9442	5.548	3	clay	16
5.58	14.16	0.8707	6.147	3	clay	14
5.74	16.19	1.0859	6.706	3	clay	16
5.91	24.56	1.5523	6.321	3	clay	24
6.07	44.37	1.5174	3.420	5	clayey silt to silty clay	21
6.23	145.28	2.0481	1.410	8	sand to silty sand	35
6.40	179.09	2.3327	1.302	8	sand to silty sand	43
6.56	112.72	2.8019	2.486	7	silty sand to sandy silt	36
6.73	72.18	2.7299	3.782	5	clayey silt to silty clay	35
6.89	44.43	2.2728	5.115	3	clay	43
7.05	36.55	1.9842	5.428	3	clay	35
7.22	35.35	2.0119	5.691	3	clay	34
7.38	32.70	1.9394	5.931	3	clay	31
7.55	33.15	1.7238	5.200	3	clay	32
7.71	41.65	1.7762	4.264	4	silty clay to clay	27
7.87	49.96	1.4911	2.985	6	sandy silt to clayey silt	19
8.04	45.37	1.7194	3.789	5	clayey silt to silty clay	22
8.20	49.00	1.6222	3.310	5	clayey silt to silty clay	23
8.37	50.62	2.2189	4.383	4	silty clay to clay	32
8.53	49.58	2.6699	5.385	3	clay	47
8.69	40.62	2.8519	7.022	3	clay	39
8.86	54.39	3.1490	5.790	3	clay	52
9.02	102.05	3.8290	3.752	5	clayey silt to silty clay	49
9.19	115.55	4.2868	3.710	6	sandy silt to clayey silt	44
9.35	115.29	4.5251	3.925	5	clayey silt to silty clay	55
9.51	121.16	4.7597	3.929	5	clayey silt to silty clay	58
9.68	123.84	4.8463	3.914	11	very stiff fine grained	119
9.84	127.87	5.0201	3.926	6	sandy silt to clayey silt	49
10.01	132.05	5.1503	3.900	12	sand to clayey sand	63
10.17	138.65	5.2799	3.808	12	sand to clayey sand	66
10.33	151.78	5.6298	3.709	12	sand to clayey sand	73

10.50	157.14	5.7875	3.683	12	sand to clayey sand	75
10.66	153.57	5.6950	3.708	12	sand to clayey sand	74
10.83	147.46	5.5059	3.734	12	sand to clayey sand	71
10.99	136.83	5.2295	3.822	12	sand to clayey sand	66
11.15	123.44	4.8229	3.907	5	clayey silt to silty clay	59
11.32	110.55	4.7626	4.308	11	very stiff fine grained	106
11.48	88.56	4.4154	4.986	11	very stiff fine grained	85
11.65	63.12	3.2845	5.204	11	very stiff fine grained	60
11.81	42.46	2.7946	6.582	3	clay	41
11.98	33.38	2.0564	6.161	3	clay	32
12.14	28.94	1.7152	5.927	3	clay	28
12.30	28.35	1.6643	5.870	3	clay	27
12.47	26.35	2.0846	7.911	3	clay	25
12.63	40.41	2.1656	5.360	3	clay	39
12.80	30.24	2.3981	7.930	3	clay	29
12.96	47.13	2.9530	6.266	3	clay	45
13.12	58.80	3.5666	6.065	11	very stiff fine grained	56
13.29	76.92	4.0874	5.314	11	very stiff fine grained	74
13.45	103.96	4.4340	4.265	11	very stiff fine grained	100
13.62	114.64	4.7168	4.114	11	very stiff fine grained	110
13.78	117.36	4.8544	4.136	11	very stiff fine grained	112
13.94	118.44	4.9189	4.153	11	very stiff fine grained	113
14.11	121.49	5.0018	4.117	11	very stiff fine grained	116
14.27	124.36	5.1462	4.138	11	very stiff fine grained	119
14.44	124.26	5.2364	4.214	11	very stiff fine grained	119
14.60	121.19	5.1690	4.265	11	very stiff fine grained	116
14.76	119.99	4.7181	3.932	5	clayey silt to silty clay	57
14.93	124.68	4.7633	3.821	6	sandy silt to clayey silt	48
15.09	128.50	5.0380	3.921	6	sandy silt to clayey silt	49
15.26	137.44	5.4948	3.998	6	sandy silt to clayey silt	53
15.42	159.70	5.9148	3.704	12	sand to clayey sand	76
15.58	169.93	6.0477	3.559	12	sand to clayey sand	81
15.75	175.24	6.5872	3.759	12	sand to clayey sand	84
15.91	168.84	6.3401	3.755	12	sand to clayey sand	81
16.08	142.90	5.4923	3.844	12	sand to clayey sand	68
16.24	127.15	4.9046	3.857	6	sandy silt to clayey silt	49
16.40	119.70	4.7505	3.969	11	very stiff fine grained	115
16.57	113.28	4.4315	3.912	5	clayey silt to silty clay	54
16.73	103.59	3.9924	3.854	5	clayey silt to silty clay	50
16.90	99.05	3.6593	3.694	6	sandy silt to clayey silt	38
17.06	100.14	3.6519	3.647	6	sandy silt to clayey silt	38
17.22	90.97	3.6118	3.970	5	clayey silt to silty clay	44
17.39	76.74	3.4417	4.485	5	clayey silt to silty clay	37
17.55	71.63	3.2670	4.561	5	clayey silt to silty clay	34
17.72	70.75	3.2152	4.544	5	clayey silt to silty clay	34
17.88	62.06	3.0169	4.862	4	silty clay to clay	40
18.04	66.09	3.0003	4.540	4	silty clay to clay	42
18.21	72.48	3.0389	4.193	5	clayey silt to silty clay	35
18.37	60.98	2.6351	4.321	5	clayey silt to silty clay	29
18.54	32.41	2.0446	6.309	3	clay	31
18.70	23.60	1.2658	5.364	3	clay	23
18.86	20.47	0.9850	4.811	3	clay	20
19.03	20.32	0.8611	4.238	4	silty clay to clay	13
19.19	20.03	0.7128	3.559	4	silty clay to clay	13
19.36	20.46	0.6683	3.266	5	clayey silt to silty clay	10
19.52	22.86	0.7841	3.431	5	clayey silt to silty clay	11
19.69	23.38	0.8623	3.688	4	silty clay to clay	15
19.85	22.15	0.8237	3.719	4	silty clay to clay	14
20.01	19.57	0.8401	4.293	3	clay	19
20.18	18.91	0.8846	4.678	3	clay	18
20.34	21.33	0.8509	3.990	4	silty clay to clay	14
20.51	18.01	0.7252	4.027	4	silty clay to clay	11
20.67	16.23	0.6047	3.726	4	silty clay to clay	10
20.83	15.50	0.5353	3.453	4	silty clay to clay	10
21.00	15.27	0.3616	2.369	5	clayey silt to silty clay	7
21.16	11.51	0.2307	2.004	5	clayey silt to silty clay	6
21.33	11.33	0.3118	2.753	4	silty clay to clay	7
21.49	13.70	0.3702	2.703	5	clayey silt to silty clay	7
21.65	19.37	0.5674	2.929	5	clayey silt to silty clay	9
21.82	24.56	0.9457	3.851	4	silty clay to clay	16
21.98	28.48	1.2929	4.540	4	silty clay to clay	18

22.15	29.39	1.5223	5.180	3	clay	28
22.31	29.86	1.7514	5.864	3	clay	29
22.47	33.29	1.8863	5.667	3	clay	32
22.64	31.03	1.6329	5.262	3	clay	30
22.80	29.68	1.5509	5.226	3	clay	28
22.97	26.28	1.2462	4.742	3	clay	25
23.13	20.93	1.2041	5.754	3	clay	20
23.29	24.30	1.2906	5.311	3	clay	23
23.46	28.88	1.8347	6.352	3	clay	28
23.62	31.56	2.1877	6.932	3	clay	30
23.79	37.61	2.0857	5.545	3	clay	36
23.95	28.49	1.7839	6.263	3	clay	27
24.11	26.97	1.8742	6.950	3	clay	26
24.28	40.78	2.2694	5.565	3	clay	39
24.44	46.02	2.8896	6.278	3	clay	44
24.61	36.35	2.2752	6.258	3	clay	35
24.77	33.22	1.4956	4.502	4	silty clay to clay	21
24.93	24.26	0.8675	3.575	5	clayey silt to silty clay	12
25.10	20.10	1.4052	6.992	3	clay	19
25.26	20.10	1.0567	5.258	3	clay	19
25.43	48.77	1.7088	3.504	5	clayey silt to silty clay	23
25.59	40.93	1.4900	3.641	5	clayey silt to silty clay	20
25.75	25.56	1.3873	5.428	3	clay	24
25.92	24.56	1.3884	5.652	3	clay	24
26.08	27.74	1.6108	5.806	3	clay	27
26.25	26.10	1.5741	6.030	3	clay	25
26.41	24.25	1.4354	5.918	3	clay	23
26.57	23.42	1.2812	5.470	3	clay	22
26.74	24.08	1.2134	5.040	3	clay	23
26.90	23.94	1.4276	5.962	3	clay	23
27.07	32.89	1.8121	5.510	3	clay	31
27.23	38.76	1.9512	5.035	3	clay	37
27.40	37.10	2.2677	6.112	3	clay	36
27.56	37.17	2.7255	7.333	3	clay	36
27.72	66.09	3.1678	4.793	4	silty clay to clay	42
27.89	60.10	2.9129	4.847	4	silty clay to clay	38
28.05	66.68	3.1890	4.783	4	silty clay to clay	43
28.22	82.15	3.8270	4.659	11	very stiff fine grained	79
28.38	106.43	3.9201	3.683	6	sandy silt to clayey silt	41
28.54	111.77	3.9112	3.499	6	sandy silt to clayey silt	43
28.71	109.31	4.0358	3.692	6	sandy silt to clayey silt	42
28.87	96.10	4.1196	4.287	11	very stiff fine grained	92
29.04	76.09	3.6511	4.799	11	very stiff fine grained	73
29.20	55.16	2.8771	5.216	3	clay	53
29.36	43.16	1.7543	4.065	5	clayey silt to silty clay	21
29.53	34.25	1.4832	4.331	4	silty clay to clay	22
29.69	24.71	1.0458	4.232	4	silty clay to clay	16
29.86	20.44	0.7305	3.573	4	silty clay to clay	13
30.02	17.25	0.5267	3.053	5	clayey silt to silty clay	8
30.18	14.96	0.3902	2.609	5	clayey silt to silty clay	7
30.35	13.29	0.4041	3.041	4	silty clay to clay	8
30.51	15.28	0.3230	2.115	5	clayey silt to silty clay	7
30.68	16.19	1.2769	7.887	3	clay	16
30.84	33.46	1.8115	5.414	3	clay	32
31.00	35.07	1.7268	4.924	3	clay	34
31.17	23.76	1.1799	4.966	3	clay	23
31.33	28.13	0.8136	2.892	5	clayey silt to silty clay	13
31.50	22.56	0.4413	1.956	6	sandy silt to clayey silt	9
31.66	20.40	0.3673	1.800	6	sandy silt to clayey silt	8
31.82	20.91	0.3104	1.484	6	sandy silt to clayey silt	8
31.99	21.94	0.7933	3.616	4	silty clay to clay	14
32.15	24.33	2.1332	8.769	3	clay	23
32.32	67.69	3.0943	4.572	4	silty clay to clay	43
32.48	113.29	4.1093	3.627	6	sandy silt to clayey silt	43
32.64	120.15	4.8449	4.033	11	very stiff fine grained	115
32.81	120.60	5.8203	4.826	11	very stiff fine grained	115
32.97	150.02	7.1189	4.745	11	very stiff fine grained	144
33.14	194.37	7.4650	3.841	12	sand to clayey sand	93
33.30	233.51	7.7198	3.306	12	sand to clayey sand	112
33.46	244.11	8.3237	3.410	12	sand to clayey sand	117
33.63	255.00	8.9973	3.528	12	sand to clayey sand	122

33.79	264.16	9.4257	3.568	12	sand to clayey sand	126
33.96	272.63	9.7471	3.575	12	sand to clayey sand	131
34.12	281.87	10.8353	3.844	12	sand to clayey sand	135
34.28	306.22	11.2780	3.683	12	sand to clayey sand	147
34.45	321.13	12.8764	4.010	12	sand to clayey sand	154
34.61	339.45	13.1030	3.860	12	sand to clayey sand	162
34.78	335.78	12.2976	3.662	12	sand to clayey sand	161
34.94	329.85	11.7275	3.555	12	sand to clayey sand	158
35.10	323.49	10.7580	3.326	12	sand to clayey sand	155
35.27	325.83	11.5125	3.533	12	sand to clayey sand	156
35.43	343.60	11.9795	3.486	12	sand to clayey sand	164
35.60	392.81	13.6933	3.486	12	sand to clayey sand	188
35.76	391.70	14.5381	3.712	12	sand to clayey sand	188
35.93	390.90	14.1939	3.631	12	sand to clayey sand	187
36.09	358.33	13.1114	3.659	12	sand to clayey sand	172
36.25	322.35	11.6550	3.616	12	sand to clayey sand	154
36.42	294.98	10.9052	3.697	12	sand to clayey sand	141
36.58	288.28	10.5140	3.647	12	sand to clayey sand	138
36.75	289.17	10.5745	3.657	12	sand to clayey sand	138
36.91	304.90	11.2356	3.685	12	sand to clayey sand	146
37.07	335.53	11.8838	3.542	12	sand to clayey sand	161
37.24	366.14	12.5666	3.432	12	sand to clayey sand	175
37.40	365.41	12.6485	3.461	12	sand to clayey sand	175
37.57	350.83	12.1922	3.475	12	sand to clayey sand	168
37.73	337.58	11.9777	3.548	12	sand to clayey sand	162
37.89	383.32	14.1656	3.696	12	sand to clayey sand	183
38.06	434.97	15.2980	3.517	12	sand to clayey sand	208
38.22	437.64	14.0004	3.199	12	sand to clayey sand	209
38.39	425.47	14.0198	3.295	12	sand to clayey sand	204
38.55	435.76	14.3674	3.297	12	sand to clayey sand	209
38.71	423.51	14.8404	3.504	12	sand to clayey sand	203
38.88	408.60	13.9202	3.407	12	sand to clayey sand	196
39.04	404.29	12.8266	3.173	12	sand to clayey sand	194
39.21	443.99	16.9793	3.824	12	sand to clayey sand	213
39.37	456.76	19.0792	4.177	12	sand to clayey sand	219
39.53	524.81	16.3549	3.116	12	sand to clayey sand	251
39.70	481.43	18.6918	3.883	12	sand to clayey sand	230
39.86	493.90	20.0469	4.059	12	sand to clayey sand	236
40.03	518.51	20.2428	3.904	12	sand to clayey sand	248
40.19	494.42	19.7944	4.004	12	sand to clayey sand	237
40.35	474.54	17.2442	3.634	12	sand to clayey sand	227

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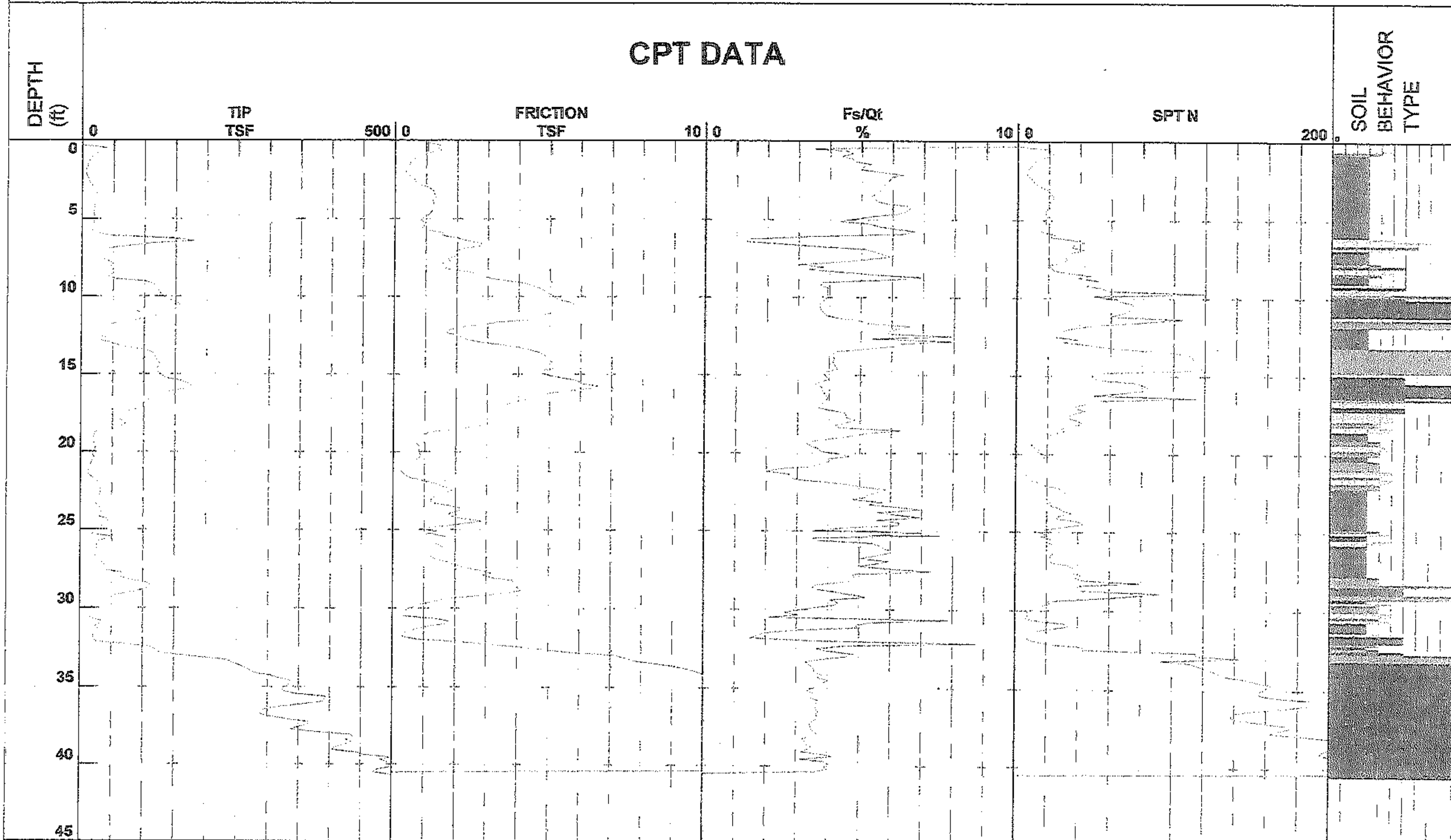
Project	Long Beach Gateway LLC	Operator	BH	Filename	CPT-01.cpt
Job Number	4329	Cone Number	DSG1047	GPS	
Hole Number	CPT-01	Date and Time	1/6/2010 8:51:51 AM	Maximum Depth	40.68 ft
Water Table Depth	25.00 ft				

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

*Soil behavior type and SPT based on data from UBC-1983

APPENDIX B

LABORATORY TESTING

Laboratory Tests

In-situ Unit Weights:
The moisture content and dry unit weight of selected samples recovered from our exploratory borings was determined in general accordance with the latest version of ASTM D 2216. The results are presented on the boring logs at the selected depths.

Shear Strength:
A Direct Shear test was performed on a selected relatively undisturbed ring sample retrieved during our subsurface exploration. The test was performed in general accordance with the latest version of ASTM D 3080. Three test specimens from the selected sample depth were placed in a ELE Soiltest D-500A Direct Shear Machine. A different normal load was applied vertically to each of the three specimens which were slowly inundated with distilled water and allowed to soak. The samples were then sheared in a horizontal direction at a constant strain rate (0.002 inch per minute) slow enough to allow for drainage. The results of this test are presented graphically on Figure DS-1 in this appendix.

Expansion:
An Expansion Index test was performed on a typical specimen of the upper soils encountered during our exploration. The test was performed in general accordance with UBC Standard 18-2. This test measures the expansion index of soils due to inundation in water of a sample with an initial saturation rate within the range of 40%-60%, and under a surcharge of 144 psf until the rate of expansion becomes constant, or for a maximum period of 24 hours. The result of this test is presented on Boring Log 4 and reveal the upper soils to be medium in expansion potential.

Consolidation:
Five (5) one-dimensional consolidation tests were initiated on specimens at in-situ moisture from the relatively undisturbed ring samples retrieved during our exploration. The tests were performed in general accordance with the latest version of ASTM D 2435. Successive load increments were applied to the top of the sample and progressive and final settlements under each increment were recorded to an accuracy of 0.0001 inch. The consolidometer, like the direct shear machine, is designed to receive the specimens in the field condition. Water was added after consolidation was achieved for the pressure as noted. Porous stones, placed at the top and bottom of the sample, permit the free flow of water into or from the sample during testing. The results of these tests are presented graphically in this appendix.

Percent Passing No. 200 Sieve:
A quantitative determination of the amount of material finer than the No. 200 sieve was performed on selected samples retrieved from exploratory Boring 5. The tests were performed in general accordance with the latest version of ASTM D 1140. Clay and other particles that are dispersed by the wash water, as well as water-soluble materials, are removed from the soil during the test. The loss in mass resulting from the wash treatment is calculated as mass percentage of the original sample and is reported as the percentage of material finer than No. 200 Sieve by washing. The results of the tests are presented on the aforementioned log.

Atterberg Limits:
Atterberg limits were performed on selected samples of fine-grained soils retrieved from our exploratory Boring 5. The tests were performed in general accordance with latest version of ASTM D 4318. The liquid limit, plastic limit and plasticity index of these soils were determined to aid in their classification and in evaluation of certain engineering parameters. The results of these tests are presented graphically on Figure AL-1 in this appendix.

Direct Shear Test Report

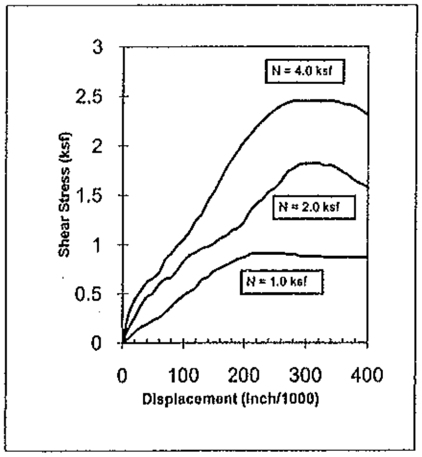
PROJECT: Long Beach Gateway LLC

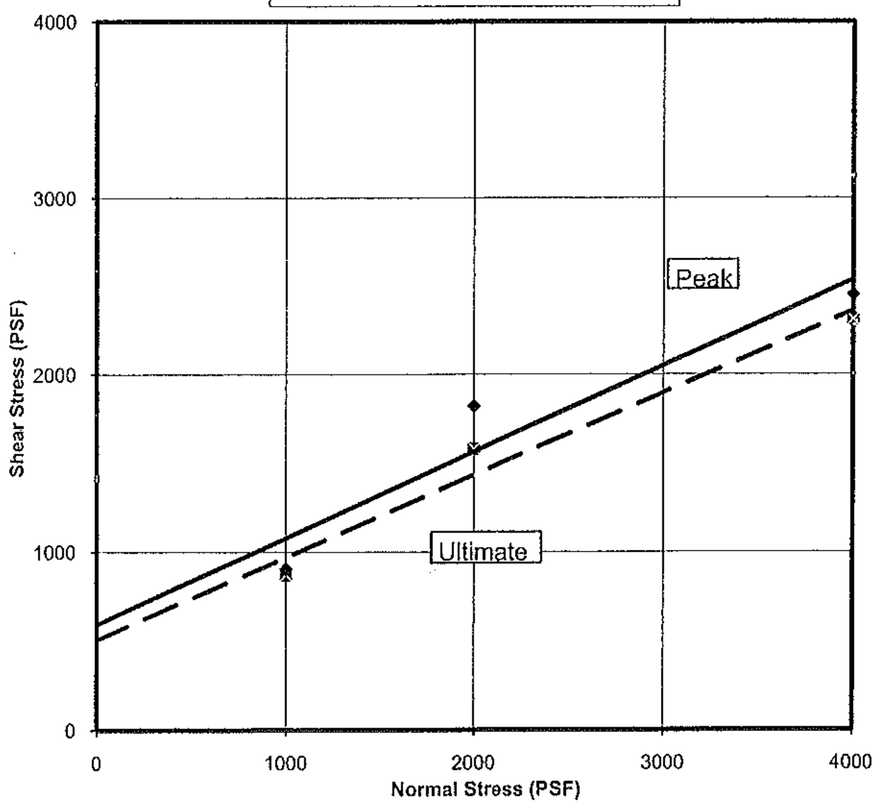
FIGURE NO.:	DS-1
JOB No.:	09-4329
DATE:	01/20/10

Sample Identification	Sample Description	Sample Test State
B-3 @ 7'	Silty Clay	Saturated-Consolidated

Peak:	**Phi (Degrees)**	**25.9**	(Avg.Dry Dens. = 104.0 pcf) (Avg.Moist. = 18.6 %)
	Cohesion (PSF)	**590.0**	
Ultimate:	**Phi (Degrees)**	**24.9**	
	Cohesion (PSF)	**505.0**	

[X] Undisturbed
[ ] Remolded





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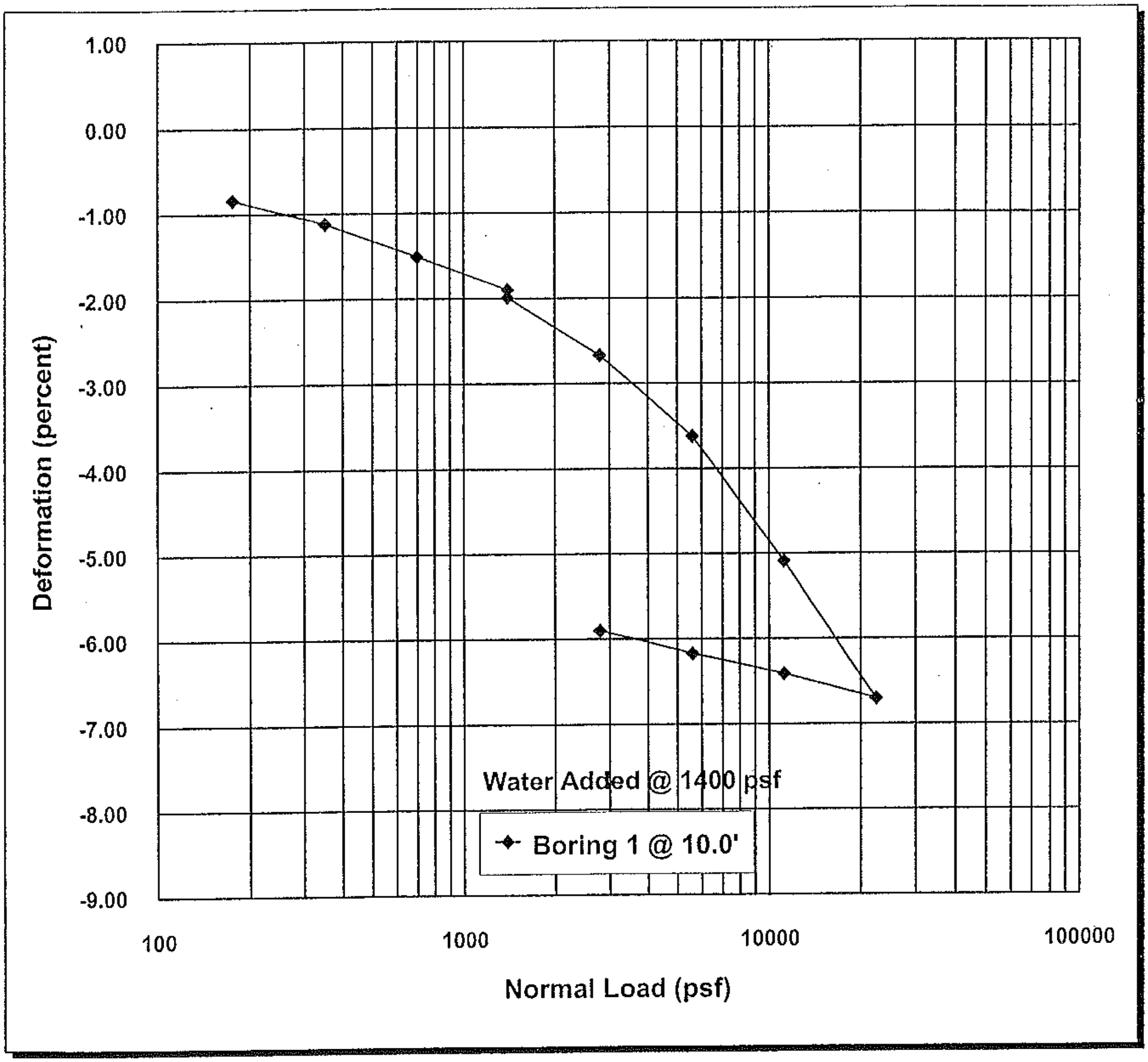

PROJECT: PROPOSED CAR WASH & RETAIL BUILDINGS	WORK ORDER 09-4329

CONSOLIDATION TEST

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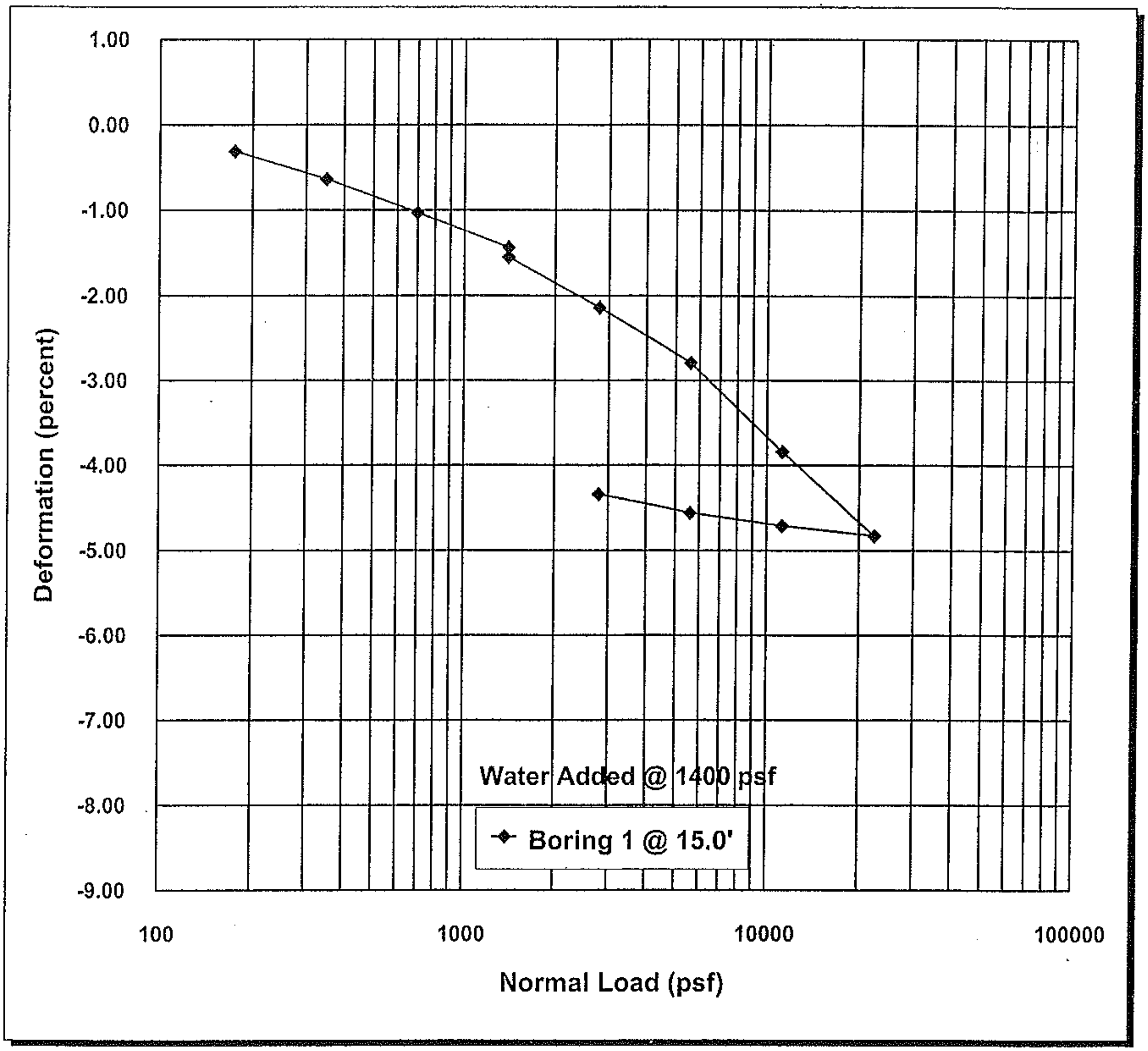
(ASTM D 2435)

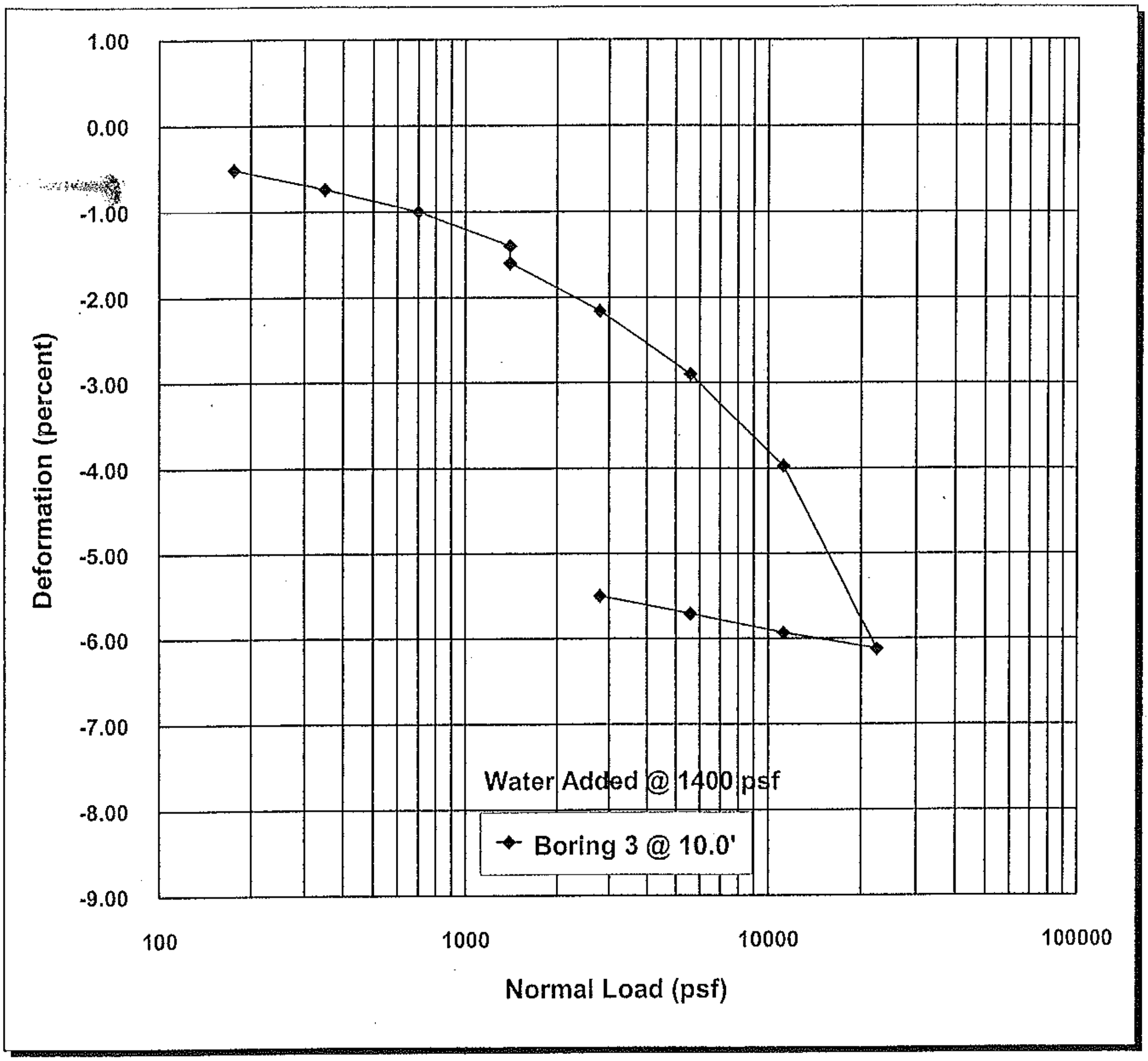


PROJECT: PROPOSED CAR WASH & RETAIL BUILDINGS	WORK ORDER 09-4329

CONSOLIDATION TEST

Western Laboratories
Geotechnical Engineering

(ASTM D 2435)

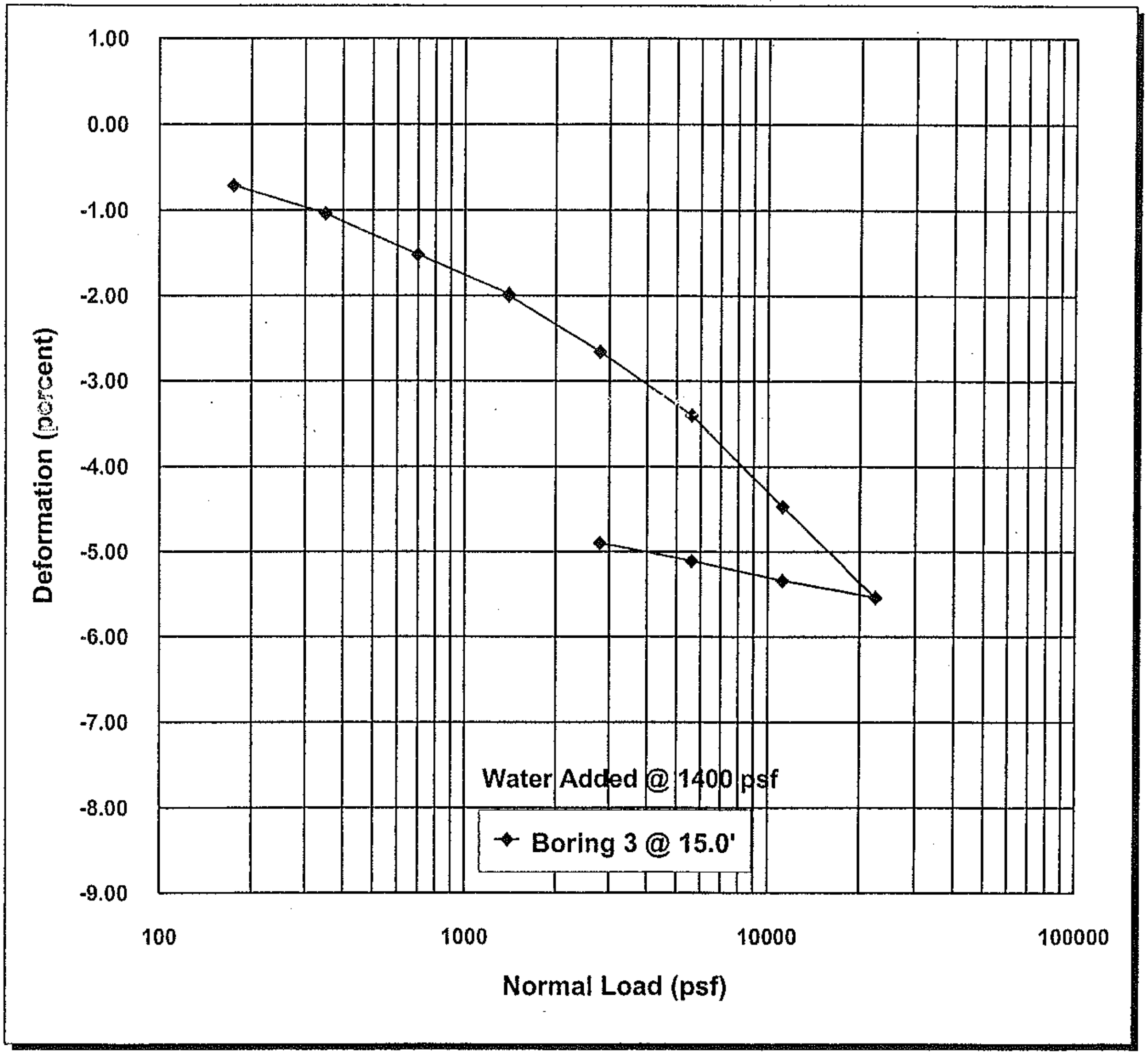


PROJECT: PROPOSED CAR WASH & RETAIL BUILDINGS | WORK ORDER 09-4329

CONSOLIDATION TEST

Western Laboratories
Geotechnical Engineering

(ASTM D 2435)

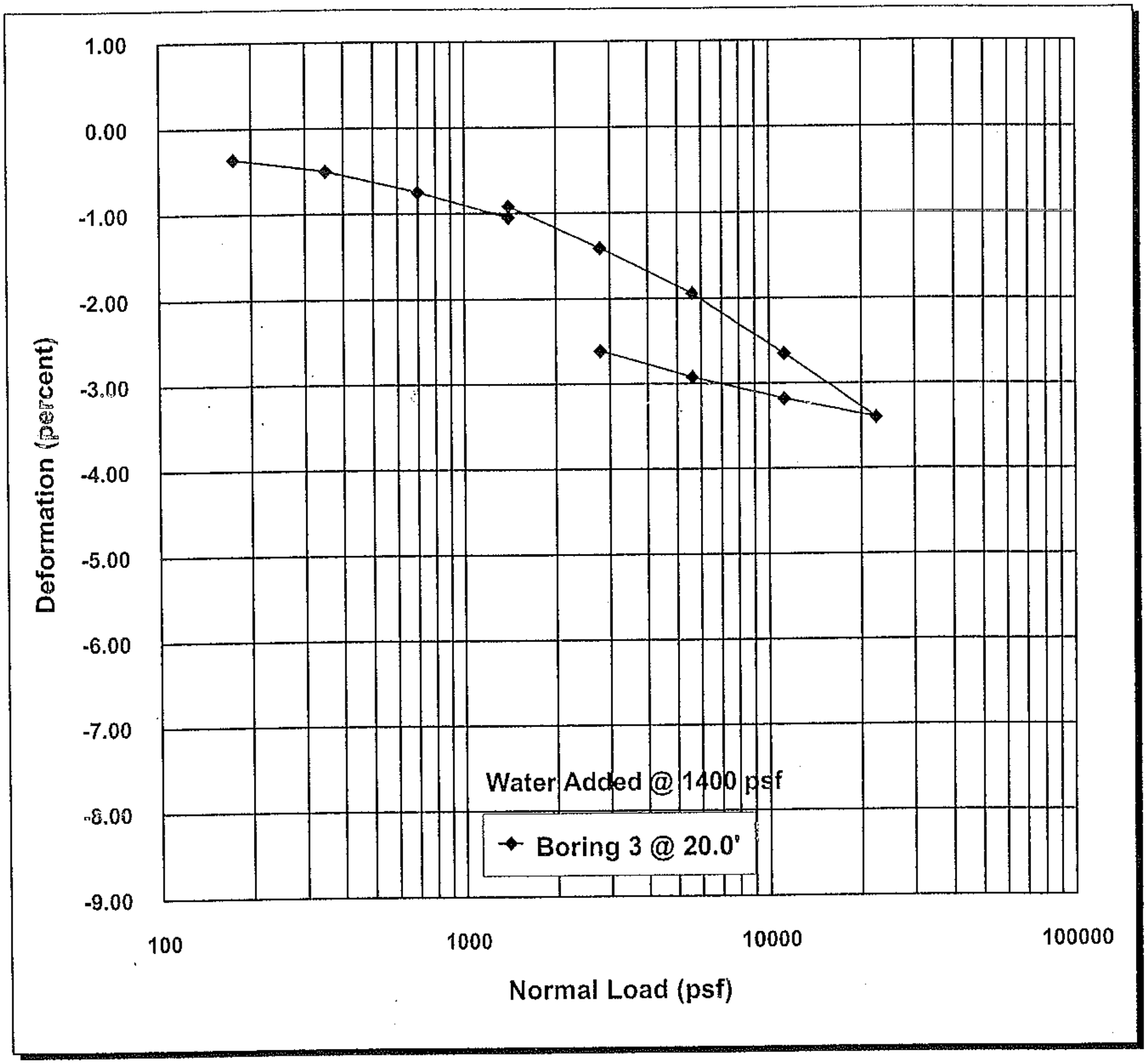


PROJECT: PROPOSED CAR WASH & RETAIL BUILDINGS	WORK ORDER 09-4329

CONSOLIDATION TEST

Western Laboratories
Geotechnical Engineering

(ASTM D 2435)

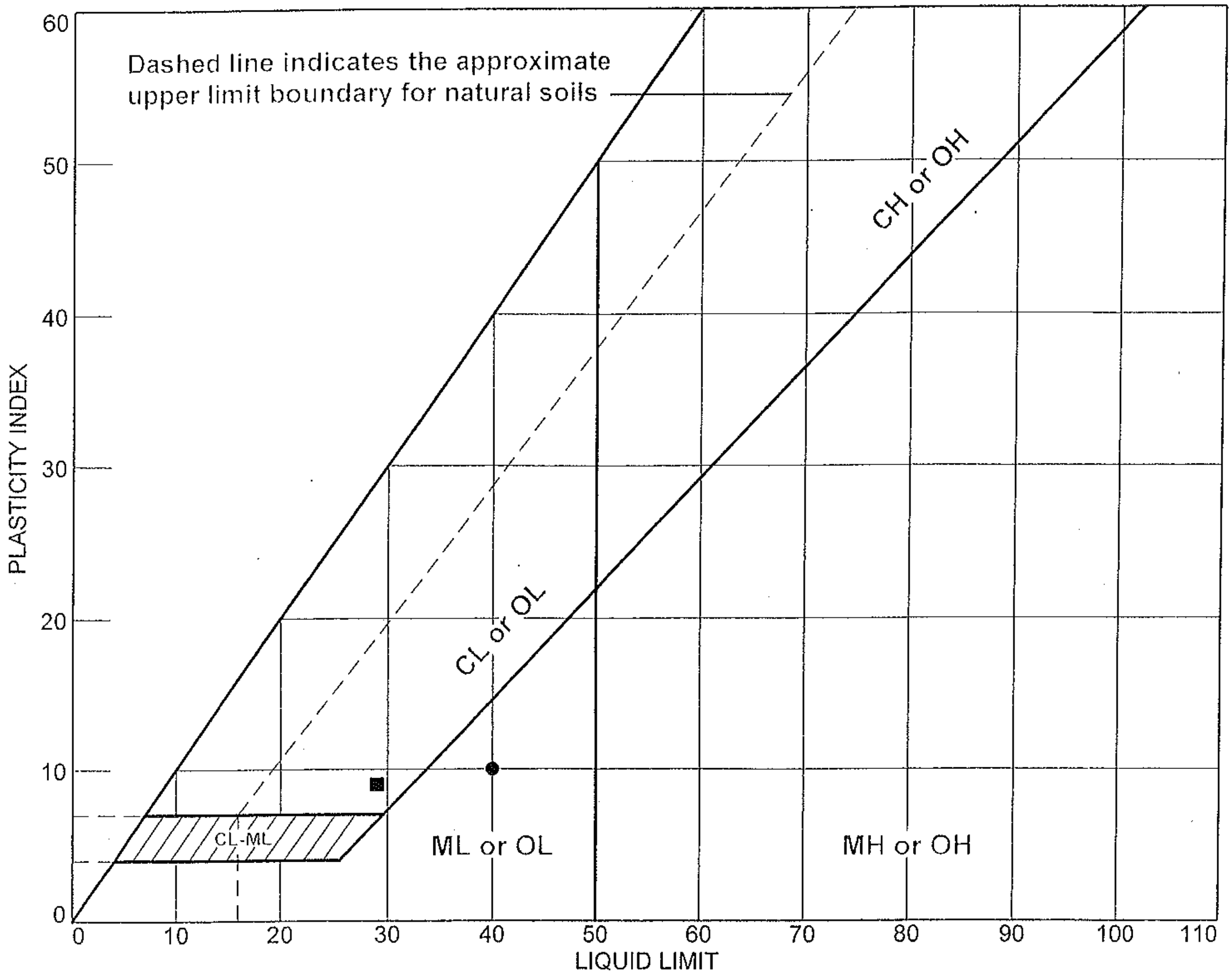


PROJECT: PROPOSED CAR WASH & RETAIL BUILDINGS | WORK ORDER 09-4329

CONSOLIDATION TEST

Western Laboratories
Geotechnical Engineering

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Boring 5	1	18'	32.6	30	40	10	ML
■	Boring 5	2	30'	23.5	20	29	9	CL

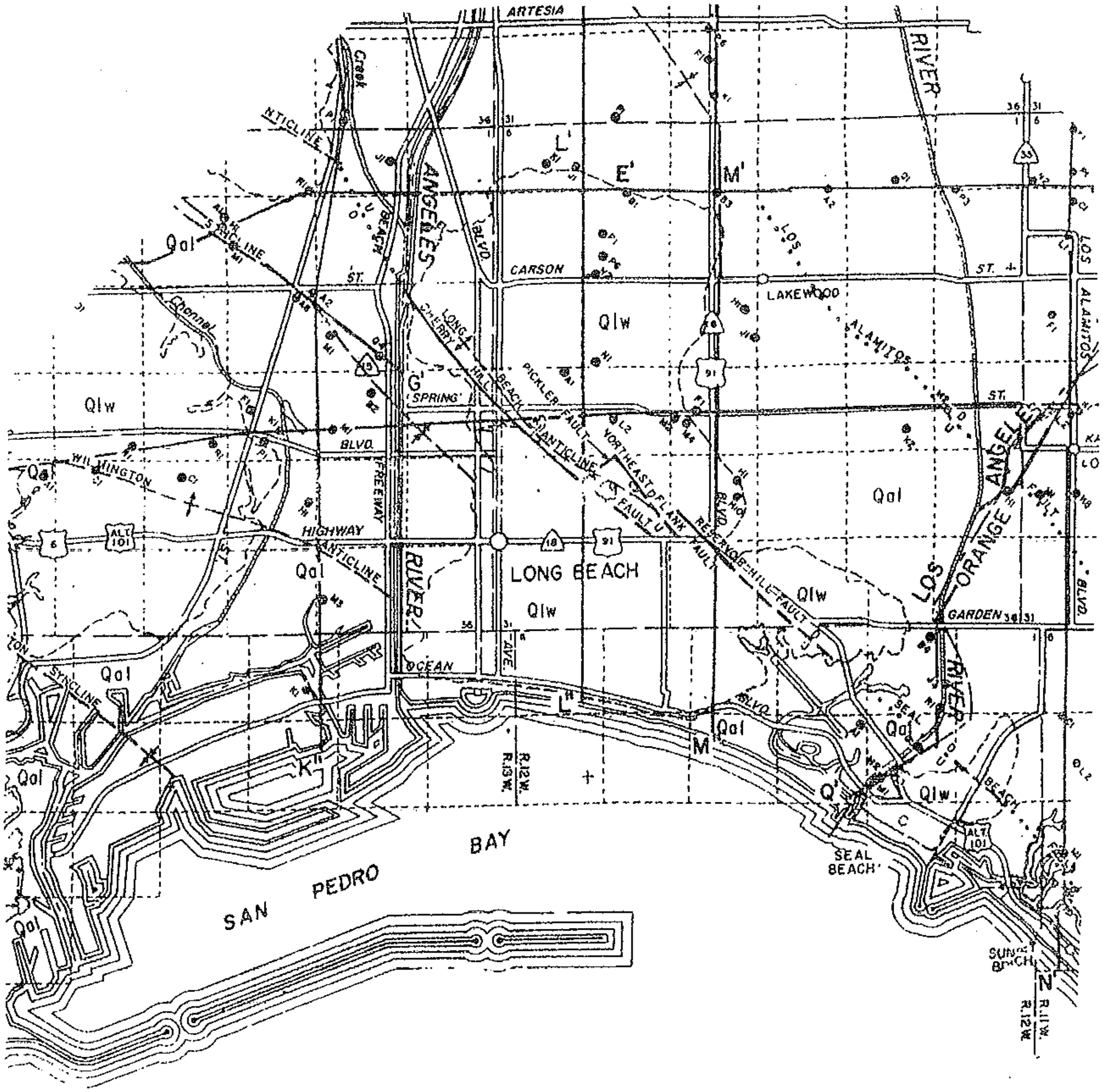
WESTERN LABORATORIES

Torrance, California

Client: LONG BEACH GATEWAY LLC
Project: 4201 Willow Street, Long Beach, California

Project No.: 09-4329 **Figure** AL-1

APPENDIX C
GEOLOGIC MAPS & PLOT PLAN

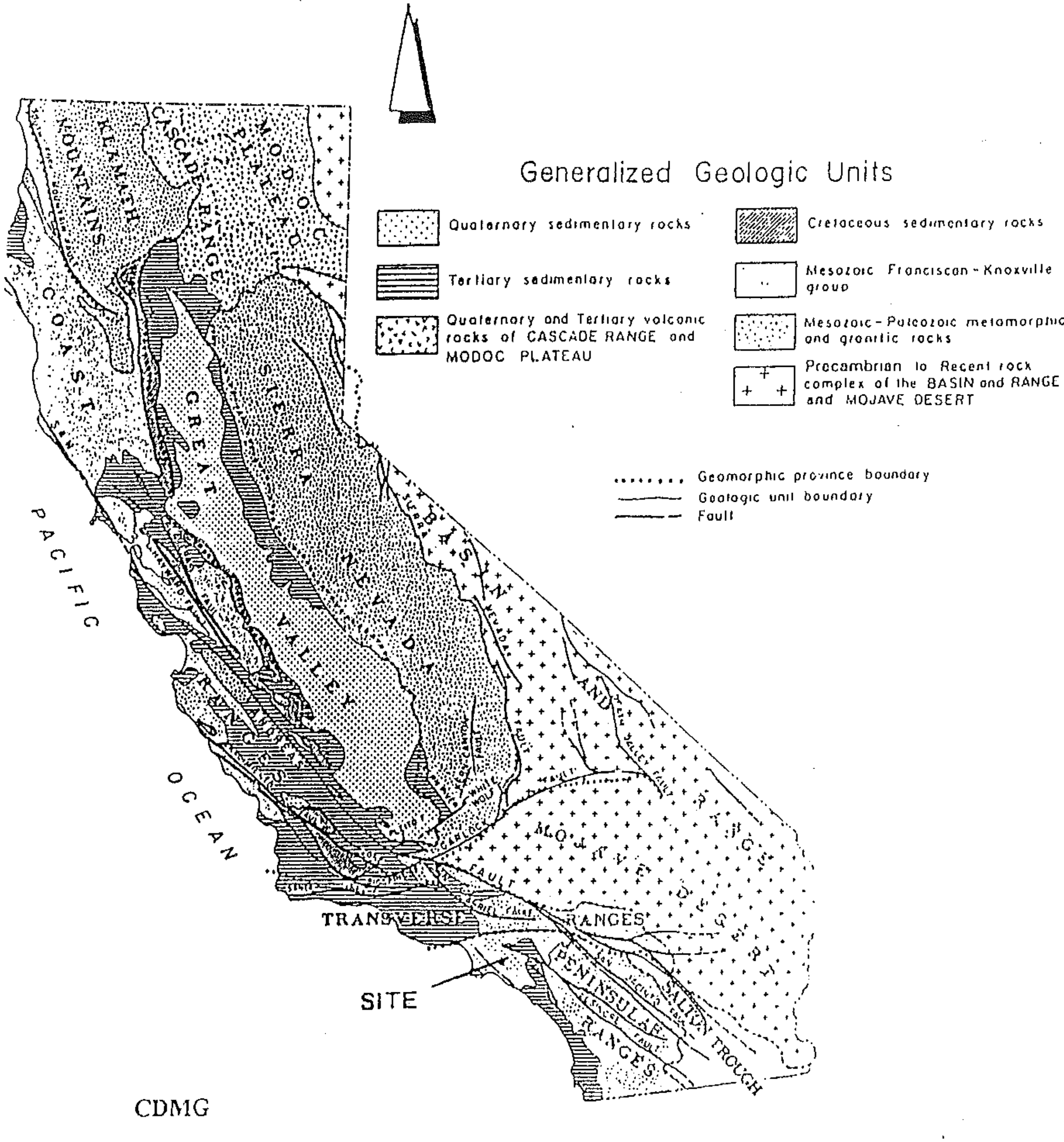


CDWR

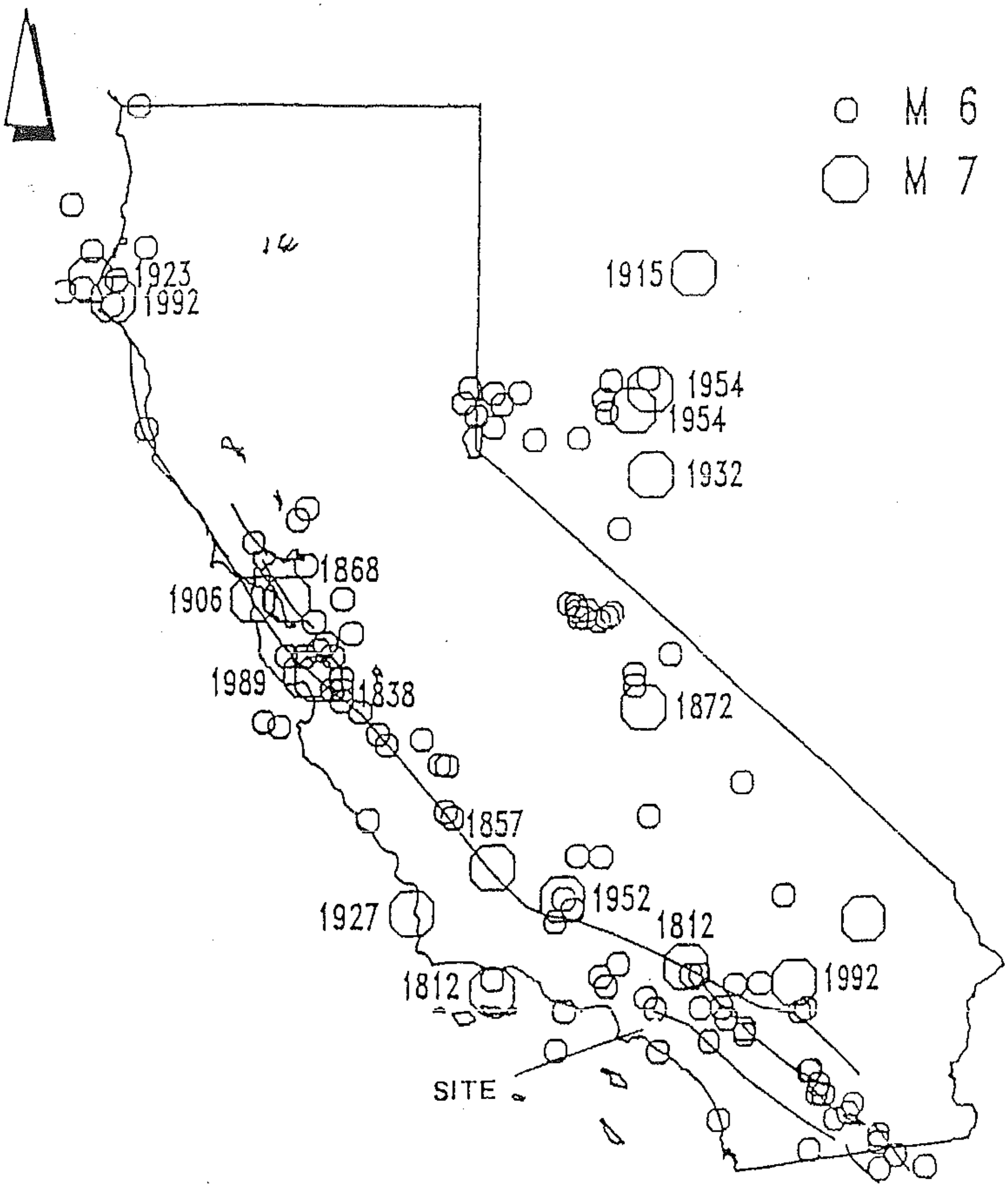
Symbol	Description
Qal	ALLUVIUM GRAVEL, SAND, SILT, AND CLAY
Qsr	ACTIVE DUNE SAND WHITE OR GREYISH, WELL SORTED SAND
Qso	OLDER DUNE SAND FINE TO MEDIUM SAND WITH SILT, AND GRAVEL LENSES
Qlw	LAKEWOOD FORMATION (INCLUDES "TERRACE DEPOSITS," "PALOS VERDES SAND," AND "UNNAMED UPPER PLEISTOCENE DEPOSITS") MARINE AND CONTINENTAL GRAVEL, SAND, SANDY SILT, SILT, AND CLAY WITH SHALE PEBBLES

1in ~ 2 mi

AREA GEOLOGIC MAP



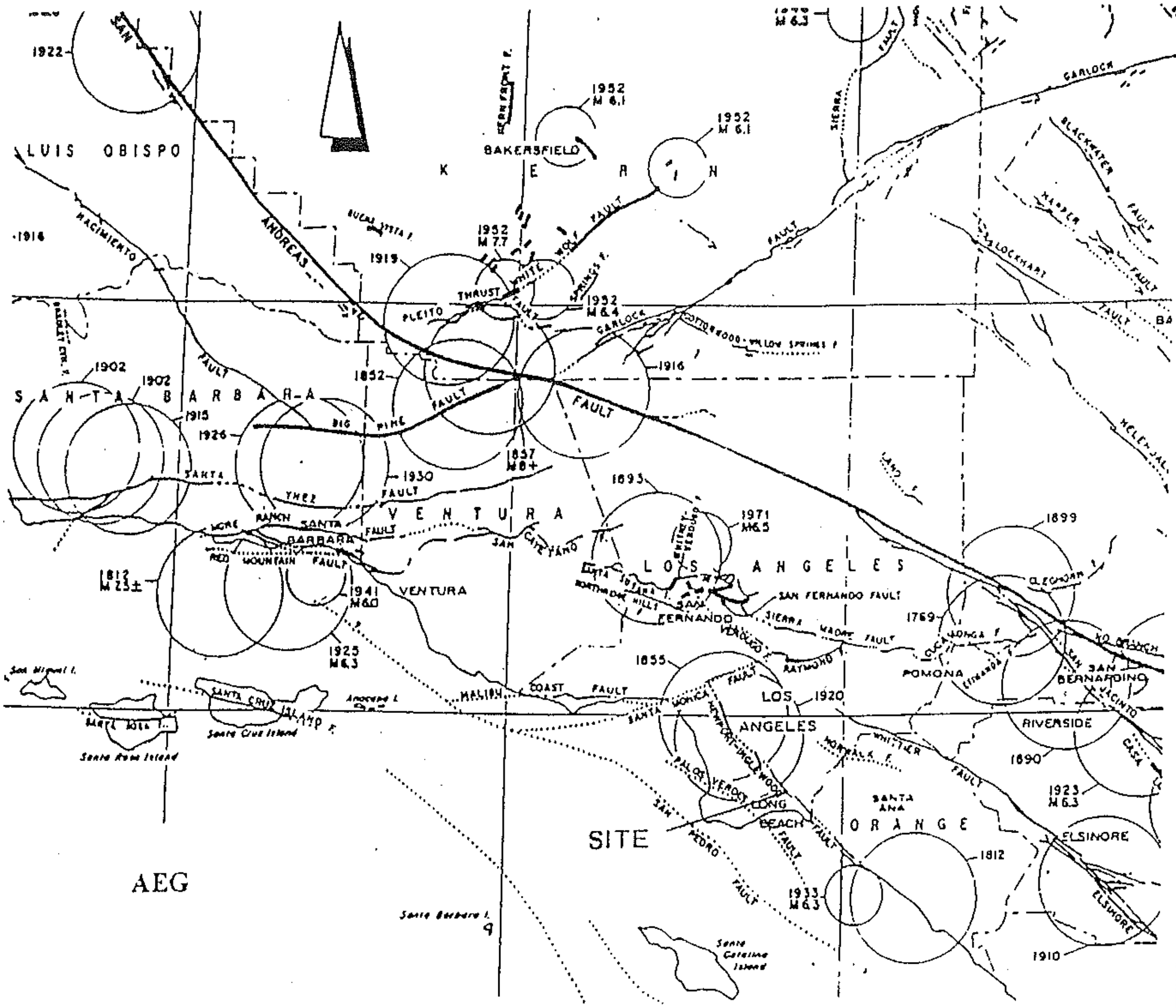
REGIONAL GEOLOGIC MAP 1 in ~ 100 mi



CDMG

EPICENTER MAP

1 in ~ 100 mi



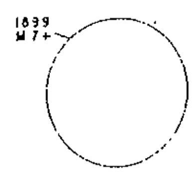
EXPLANATION

ACTIVE FAULTS

Total length of fault zone that breaks Holocene deposits or that has had seismic activity

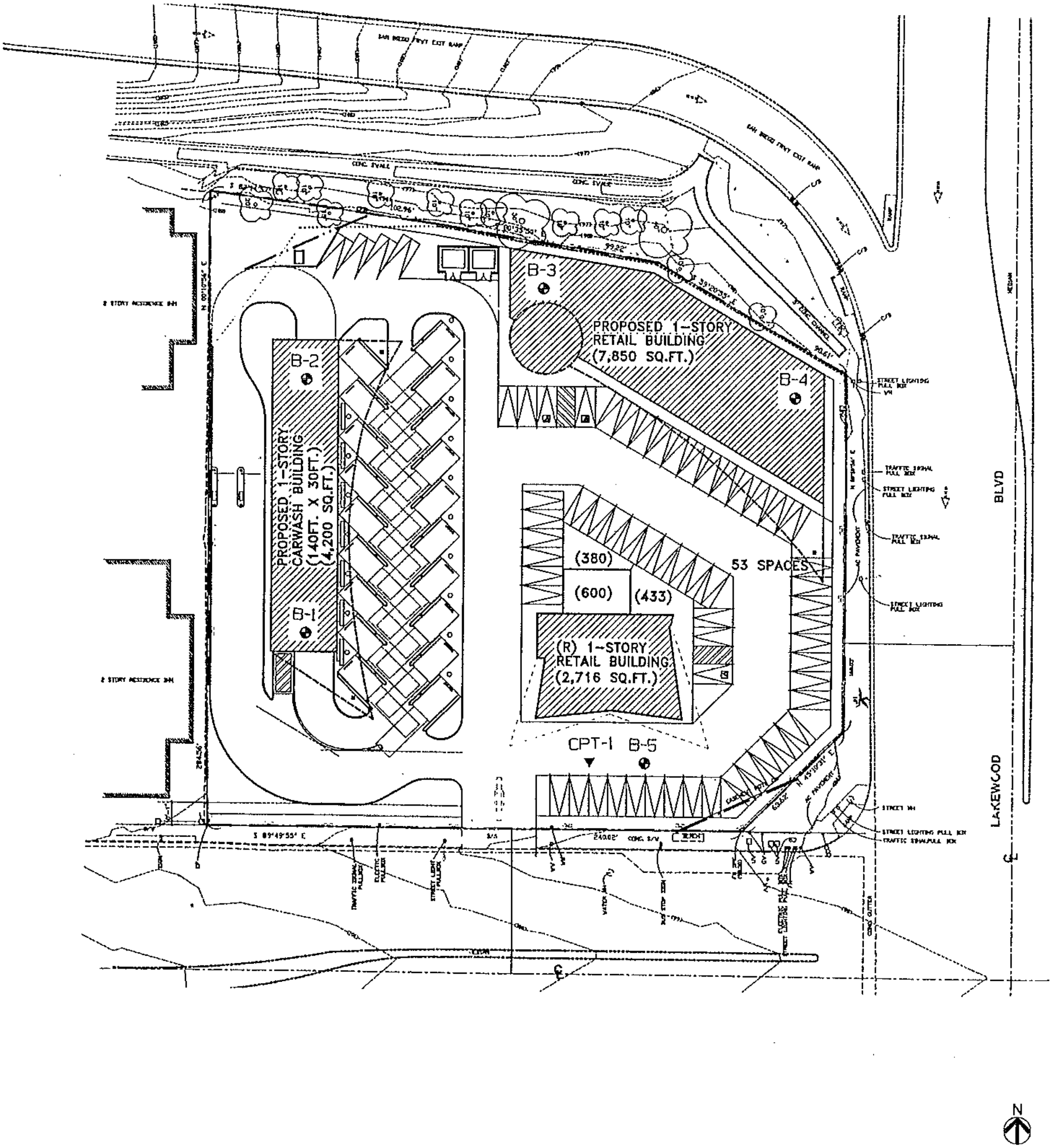
Fault segment with surface rupture during an historic earthquake, or with aseismic fault creep.

EARTHQUAKE LOCATIONS



FAULT MAP

1 in ~ 30 mi



LEGEND

= APPROXIMATE LOCATION OF EXPLORATORY BORING

= APPROXIMATE LOCATION OF CPT SOUNDING

PLOT PLAN

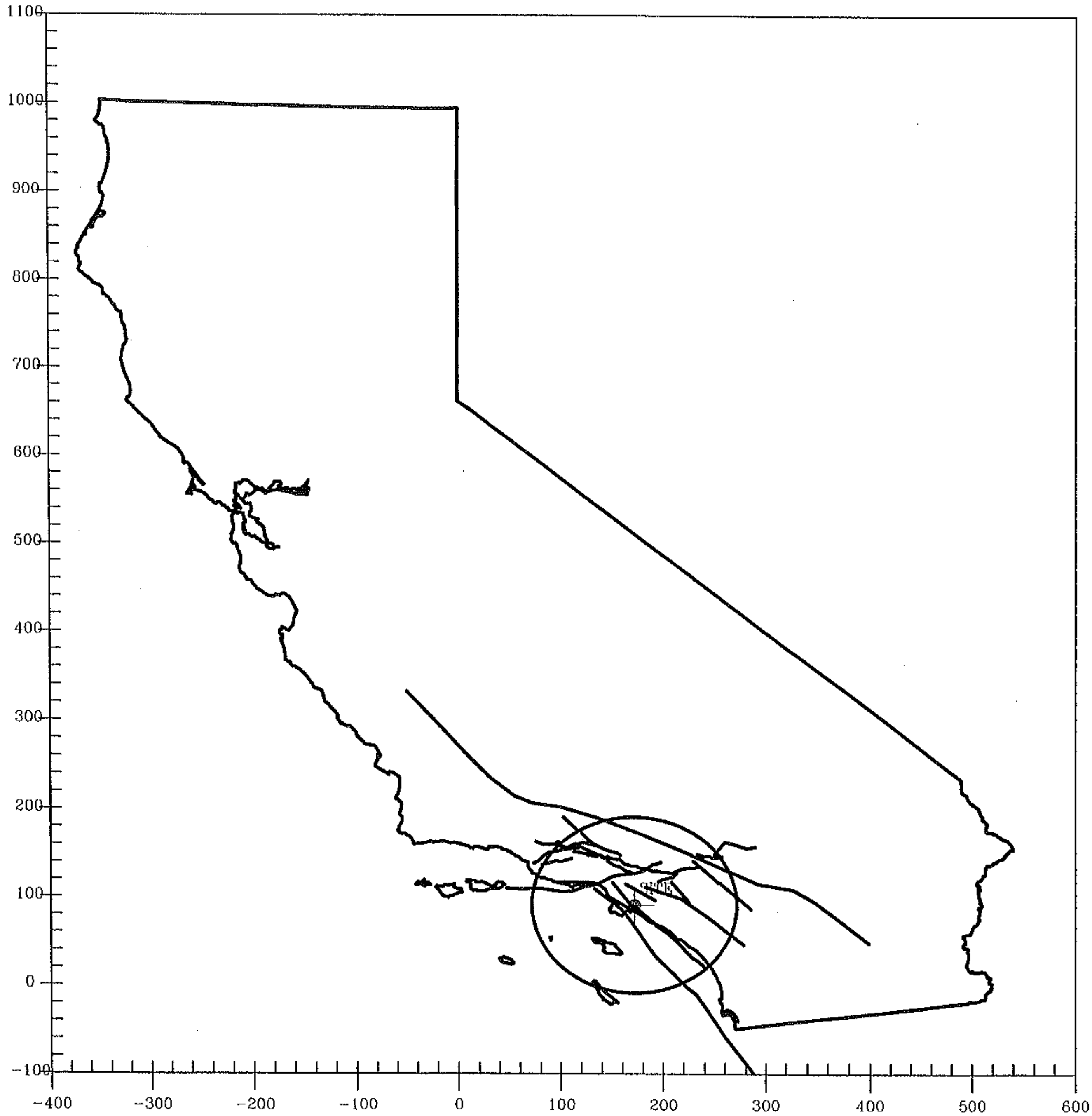
SCALE:	N.T.S.		DRAWN BY
DATE:	02-26-10	LONG BEACH GATEWAY LLC	REVISED
		4201 E. WILLOW STREET LONG BEACH, CALIFORNIA	
		WESTERN LABORATORIES	DRAWING NUMBER 09-4329

SPACE MASTERS
ARCHITECTURE ■ INTERIORS ■ PLANNING ■
3435 WILSHIRE BLVD. SUITE 2906, LOS ANGELES, CA 90010
TEL: (213) 386-3693 FAX: (213) 386-4062

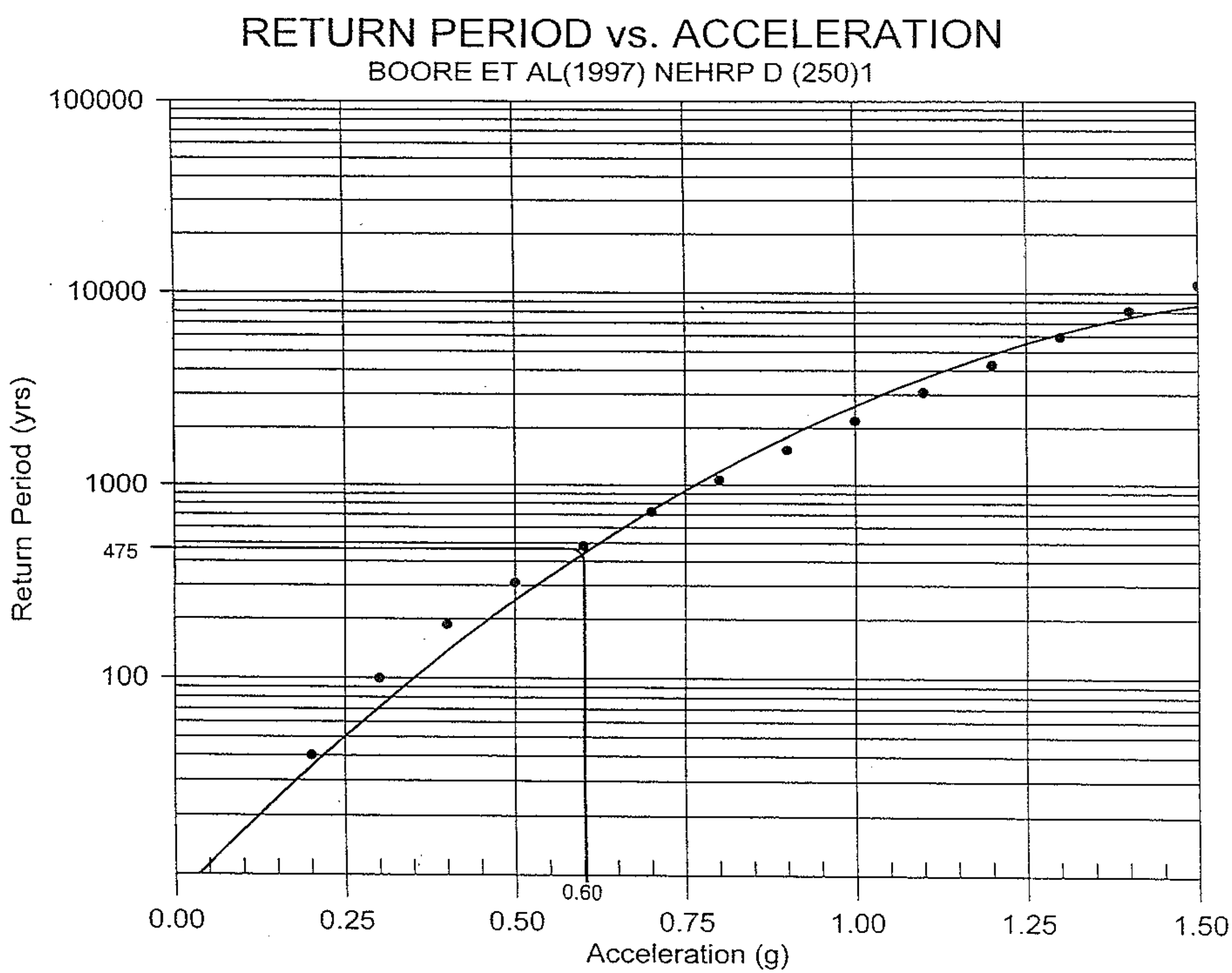
APPENDIX D

SITE SEISMIC HAZARD ANALYSIS

CALIFORNIA FAULT MAP



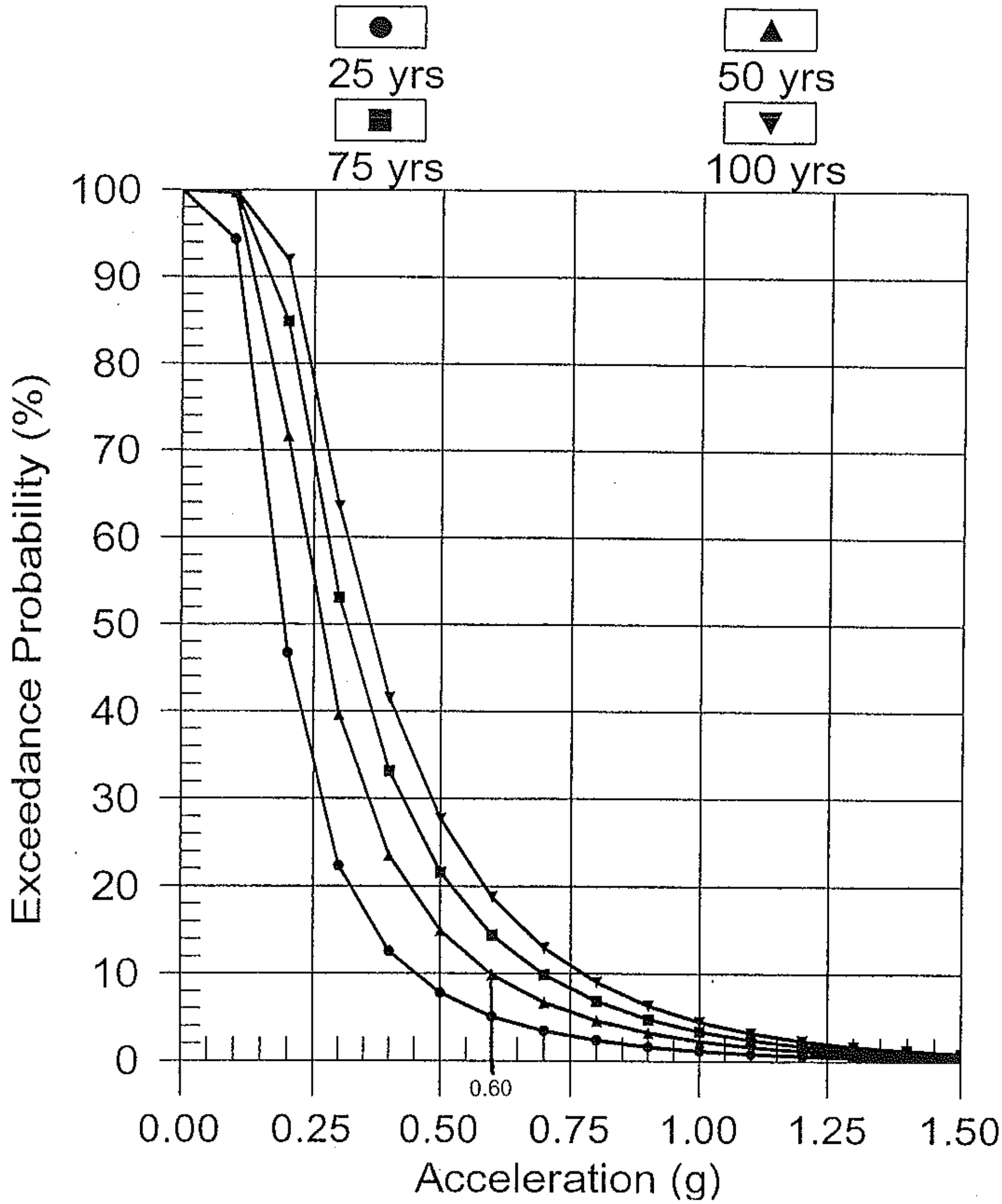
Long Beach Gateway LLC
Latitude - 33.8037
Longitude - -118.1432
Work Order - 09-4329



LONG BEACH GATEWAY LLC
4201 E. WILLOW STREET, LONG BEACH
WORK ORDER 09-4329

PROBABILITY OF EXCEEDANCE

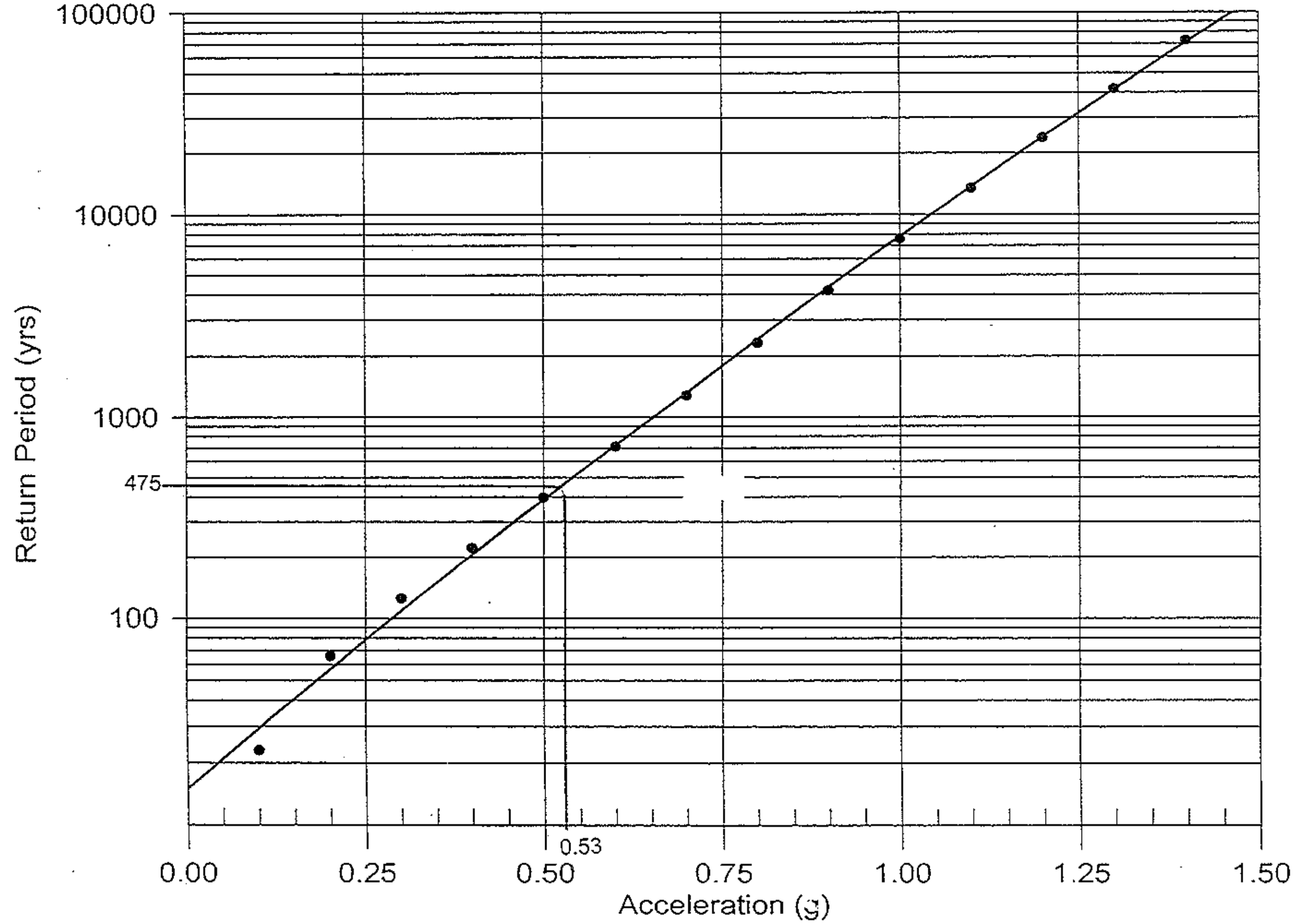
BOORE ET AL(1997) NEHRP D (250)1



LONG BEACH GATEWAY LLC
4201 E. WILLOW STREET, LONG BEACH
WORK ORDER 09-4329

RETURN PERIOD vs. ACCELERATION

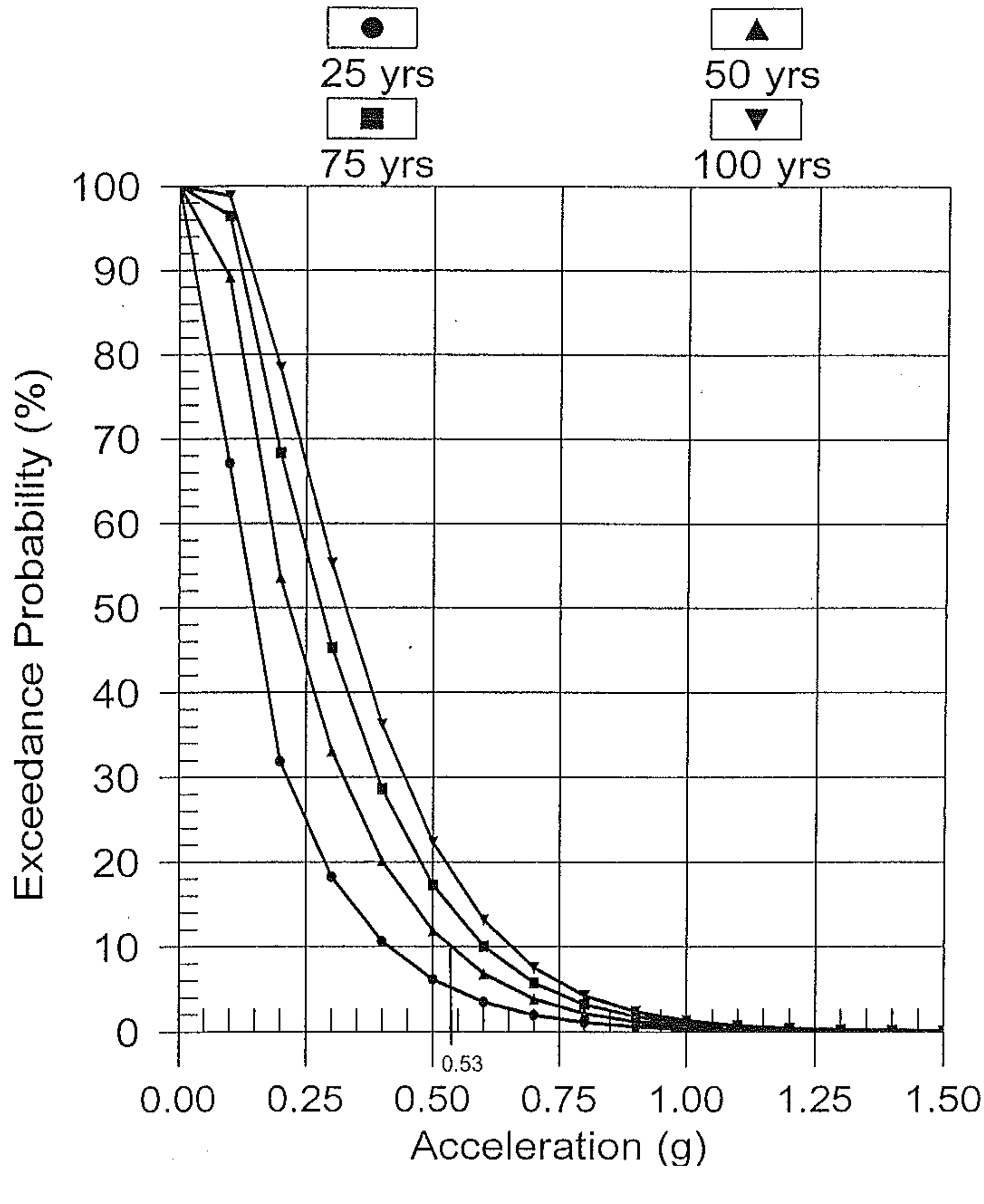
CAMP. & BOZ. (1994/1997) AL 1



LONG BEACH GATEWAY LLC
4201 E. WILLOW STREET, LONG BEACH
WORK ORDER 09-4329

PROBABILITY OF EXCEEDANCE

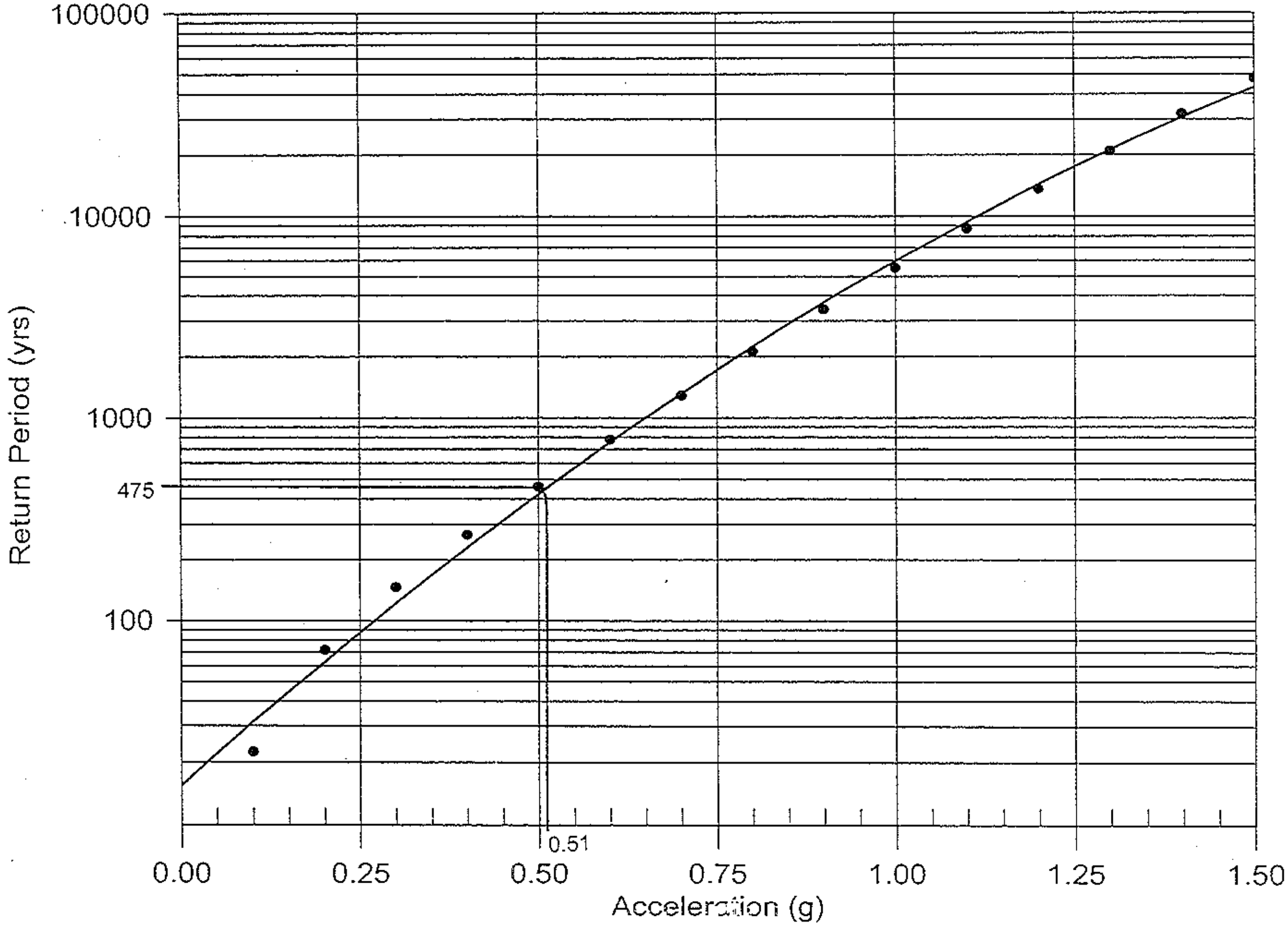
CAMP. & BOZ. (1994/1997) AL 1



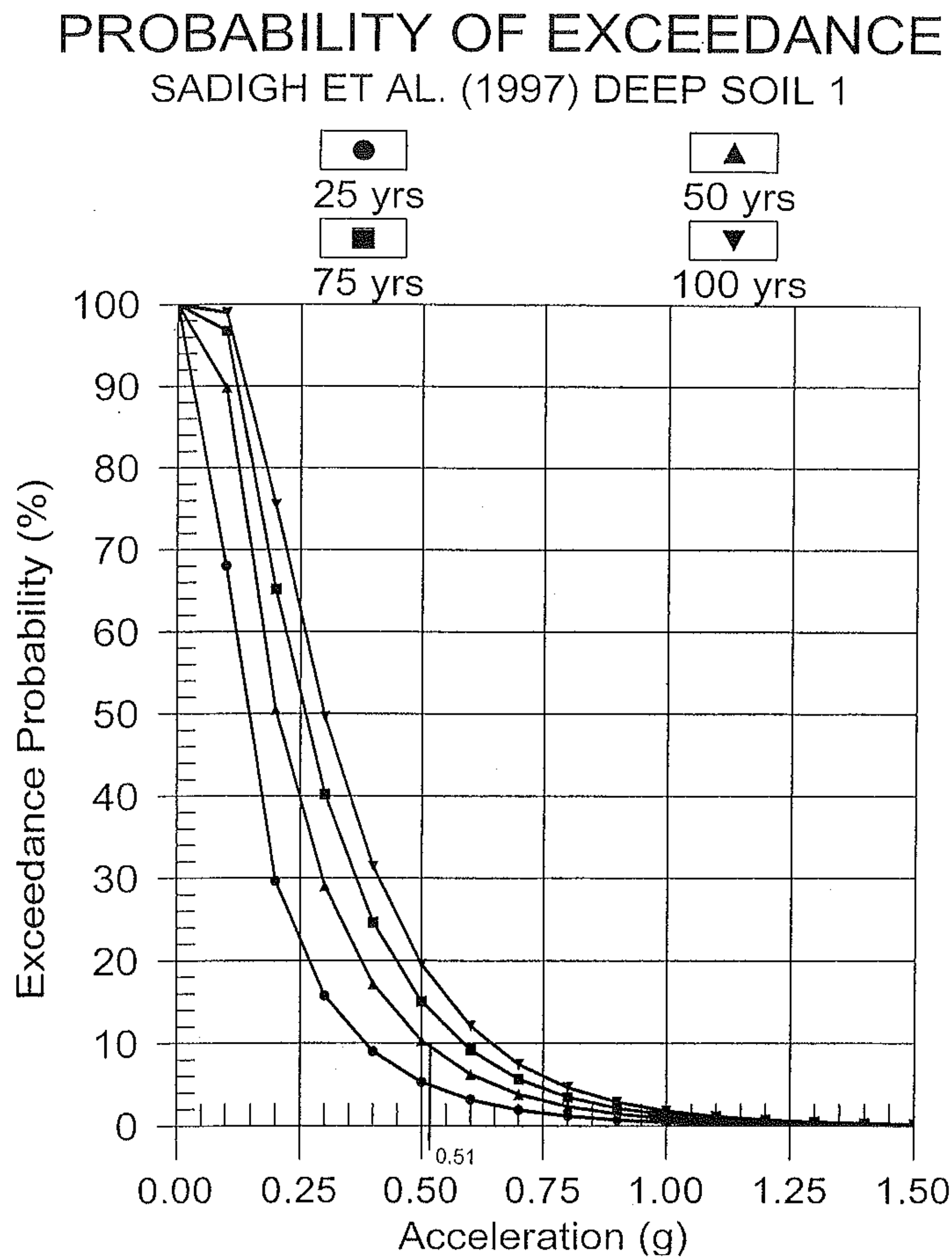
LONG BEACH GATEWAY LLC
4201 E. WILLOW STREET, LONG BEACH
WORK ORDER 09-4329

RETURN PERIOD vs. ACCELERATION

SADIGH ET AL. (1997) DEEP SOIL 1



LONG BEACH GATEWAY LLC
4201 E. WILLOW STREET, LONG BEACH
WORK ORDER 09-4329



LONG BEACH GATEWAY LLC
4201 E. WILLOW STREET, LONG BEACH
WORK ORDER 09-4329

APPENDIX E

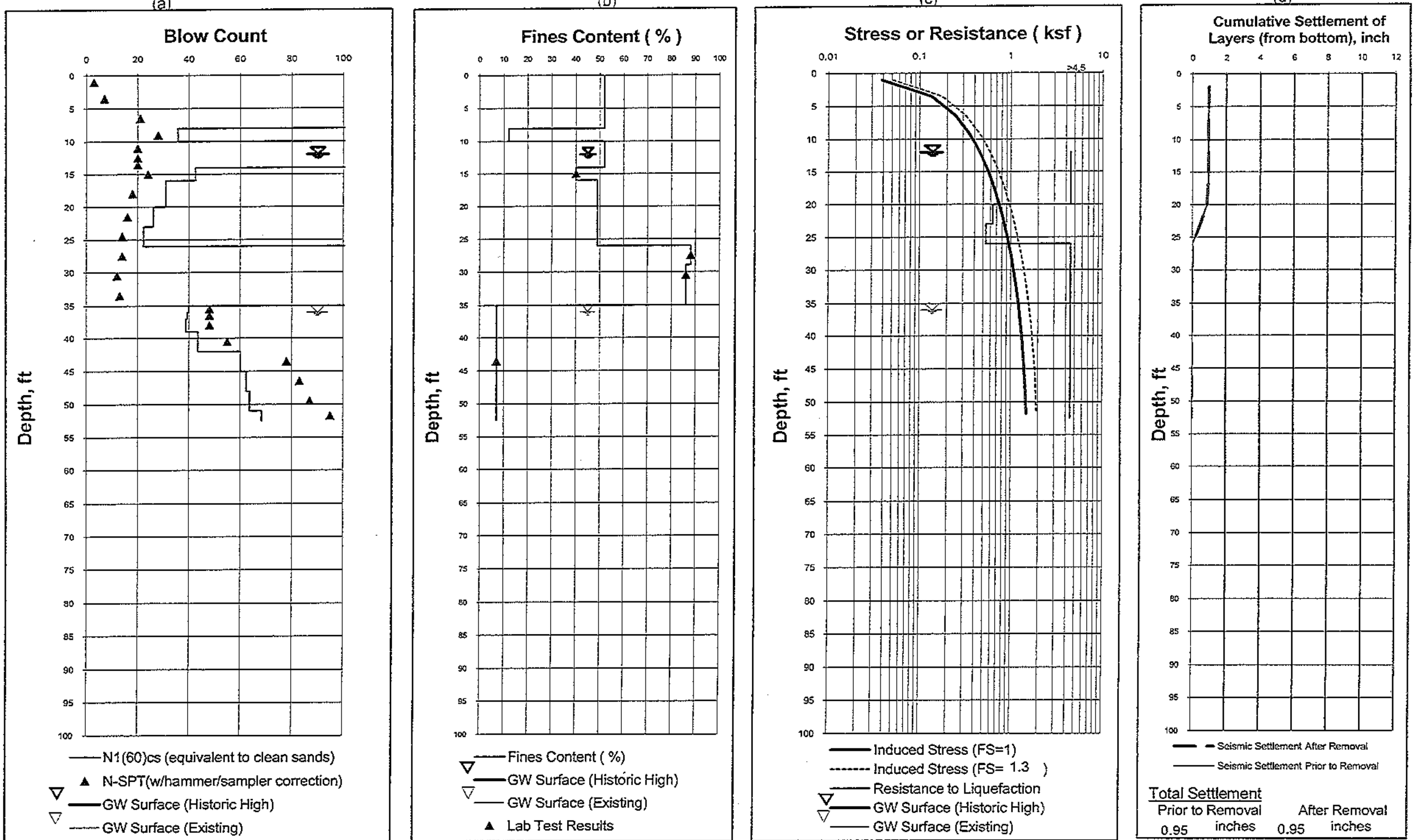
LIQUEFACTION POTENTIAL
&
SEISMIC SETTLEMENT EVALUATIONS

COMPUTER PROGRAM: EQLique&Settle"2"

Location.......... B-5 Surcharge 0.00 ksf

Elevation (MSL) (ft) 32

NOTE: If the total settlement is very small (e.g.<0.05"), it will not be seen due to the scale used, and should be reported as "negligible".



Removal & Recomp. Depth (ft) = 5

PROJECT: 4201 Willow Street Long Beach, L.A. County, California Long Beach Gateway LLC	Weighted Ground Accel. (M=7.5) = 0.50 g Site Magnitude = 7.2	**Liquefaction Potential and Seismic Settlements Based on Boring Data** **Western Laboratories** **Geotechnical Engineering Consultants**	Job No.: 09-4329 Date: 2-22-2010 Figure No. Liq B-5

GENERAL INPUT DATA FOR THE SITE

LIQUEFACTION POTENTIAL AND SEISMIC SETTLEMENT EVALUATIONS

Western Laboratories		Geotechnical Engineering Consultants		
PROJECT:	4201 Willow Street	Long Beach, L.A. County, California		Long Beach Gateway LLC
Job No.:	09-4329	Calculated By.:	LS	Date: 2/22/2010
		Checked By:	EC	Date 2/22/2010

Location (Boring No.)	B-5	Surcharge 0.00 ksf	Ref. Earthquake Magnitude	7.5	
Type of Sampler (SPT/Other)	Various		Approx. Distance From Site		(optional)
Ground Surface Elevation	32	ft. MSL	Site Earthquake Magnitude	7.2	
Existing Ground Water Depth	36	ft.(Minimum 0.1ft. below GL)	Peak Ground Accel (M = 7.5)	0.50	g.<<Calculated by program (=K10/M11), or entered by user.
			PGA (for site M = 7.2)	0.55	g
Historic High Ground Water Depth	12	ft.(Minimum 0.1ft. below GL)	Magnitude Scaling Factors (MSF)	1.0 (M=7.5)	1.11 (M= 7.2) <<<Calculated by program.

Agency-Required Factor of Safety (FS) to classify layers as "liquefiable": (min. 1.0) (enter 1 if no special agency-required FS, or enter your selection)>>>>	1.3

LiqueBmgb

PROGRAM

EQLIQUE & SETTLE"2"

April 2005

4372 Mentone Street, San Diego, CA. 92107
Phone (619) 225-2233; FAX & Phone (310) 370-7746 Cellular: (858) 220-3000; (310) 713-9005
For Order Form please send e-mail to: **applgeo@aol.com OR edcastellan@hotmail.com**

CAUTION: DATA MUST BE ENTERED IN LIGHT-SHADED CELLS ONLY. (See Appendix "A" for eventual changes under "other equipment" in Column "V").

Boring No. B-5 | Borehole Diameter = 4 inches | For CD:see Hamm\Samp tables >>

Figure No. Liq B-5 (for graphs)

Prop. Removal/Recomp. Depth (Ft) = 5

Layer No.	Depth of layer below ground surface (ft) Top	Bottom	Mid Layer Depth (Ft)	Main soil USCS Symbol	Field Blow-Count (2).	Type of Sampler (See Note 1)	N-SPT (corrected for Hammer/ sampler)	Percent fines (Lab)	Percent fines (for the layer)	(decide per boring log) "N" field Conversn. factor for hammer\ sampler	(do not type in this column) Layer Unit WT kcf	Layer Unit Weight (pcf) (lab or assumed)
1	0.0	2.00	1.0	CL-ML	3	SPT	3		52	1.0000	0.12	120
2	2.00	5.00	3.5	CL-ML	7	SPT	7		52	1.0000	0.12	120
3	5.00	8.00	6.5	CL-ML	21	SPT	21		52	1.0000	0.12	120
4	8.00	10.00	9.0	SM	28	SPT	28		12	1.0000	0.125	125
5	10.00	12.00	11.0	CL-ML	20	SPT	20		52	1.0000	0.12	120
6	12.00	13.00	12.5	CL-ML	20	SPT	20		52	1.0000	0.12	120
7	13.00	14.00	13.5	CL-ML	20	SPT	20		52	1.0000	0.12	120
8	14.00	16.00	15.0	SM	24	SPT	24	40	40	1.0000	0.125	125
9	16.00	20.00	18.0	SM	18	SPT	18		49	1.0000	0.12	120
10	20.00	23.00	21.5	SM	16	SPT	16		49	1.0000	0.12	120
11	23.00	26.00	24.5	SM	14	SPT	14		49	1.0000	0.12	120
12	26.00	29.00	27.5	CL	14	SPT	14	88	88	1.0000	0.12	120
13	29.00	32.00	30.5	CL	12	SPT	12	86	86	1.0000	0.12	120
14	32.00	35.00	33.5	CL	13	SPT	13		86	1.0000	0.12	120
15	35.00	36.00	35.5	SP-SM	48	SPT	48		7	1.0000	0.125	125
16	36.00	37.00	36.5	SP-SM	48	SPT	48		7	1.0000	0.125	125
17	37.00	39.00	38.0	SP-SM	48	SPT	48		7	1.0000	0.125	125
18	39.00	42.00	40.5	SP-SM	55	SPT	55		7	1.0000	0.125	125
19	42.00	45.00	43.5	SP-SM	78	SPT	78	7	7	1.0000	0.125	125
20	45.00	48.00	46.5	SP-SM	83	SPT	83		7	1.0000	0.125	125
21	48.00	51.00	49.5	SP-SM	87	SPT	87		7	1.0000	0.125	125
22	51.00	52.50	51.8	SP-SM	95	SPT	95		7	1.0000	0.125	125
23	52.50		#N/A				#N/A			#N/A	#N/A	
24	#N/A		#N/A				#N/A			#N/A	#N/A	
25	#N/A		#N/A				#N/A			#N/A	#N/A	
26	#N/A		#N/A				#N/A			#N/A	#N/A	
27	#N/A		#N/A				#N/A			#N/A	#N/A	
28	#N/A		#N/A				#N/A			#N/A	#N/A	
29	#N/A		#N/A				#N/A			#N/A	#N/A	
30	#N/A		#N/A				#N/A			#N/A	#N/A	
31	#N/A		#N/A				#N/A			#N/A	#N/A	
32	#N/A		#N/A				#N/A			#N/A	#N/A	
33	#N/A		#N/A				#N/A			#N/A	#N/A	
34	#N/A		#N/A				#N/A			#N/A	#N/A	
35	#N/A		#N/A				#N/A			#N/A	#N/A	
36	#N/A		#N/A				#N/A			#N/A	#N/A	
37	#N/A		#N/A				#N/A			#N/A	#N/A	
38	#N/A		#N/A				#N/A			#N/A	#N/A	
39	#N/A		#N/A				#N/A			#N/A	#N/A	
40	#N/A		#N/A				#N/A			#N/A	#N/A	
41	#N/A		#N/A				#N/A			#N/A	#N/A	
42	#N/A		#N/A				#N/A			#N/A	#N/A	
43	#N/A		#N/A				#N/A			#N/A	#N/A	
44	#N/A		#N/A				#N/A			#N/A	#N/A	
45	#N/A		#N/A				#N/A			#N/A	#N/A	
46	#N/A		#N/A				#N/A			#N/A	#N/A	
47	#N/A		#N/A				#N/A			#N/A	#N/A	
48	#N/A		#N/A				#N/A			#N/A	#N/A	
49	#N/A		#N/A				#N/A			#N/A	#N/A	
50	#N/A		#N/A				#N/A			#N/A	#N/A	
51	#N/A		#N/A				#N/A			#N/A	#N/A	
52	#N/A		#N/A				#N/A			#N/A	#N/A	
53	#N/A		#N/A				#N/A			#N/A	#N/A	
54	#N/A		#N/A				#N/A			#N/A	#N/A	

Note 1: The SPT sampler in Column "I" above is the correct Terzaghi & Peck tool to measure N-SPT (OD=2"; ID=1-3/8"constant or 1.5" barrel with liners). (If the 1.5" ID barrel is used w/no liners, select for analysis a file with "nls" added at the end of the filename).

Column "J"

LIQUEFACTION POTENTIAL EVALUATION

Western Laboratories
Geotechnical Engineering Consultants
Long Beach, L.A. County, California

PROJECT: 4201 Willow Street — Long Beach Gateway LLC
Job No.: 09-4329

Calculated By.:	LS	Date:	2/22/2010
Checked By:	EC	Date:	2/22/2010

Borehole diameter (in)=	4	<<<PLEASE DO NOT TOUCH THIS CELL
Correction factor for borehole diameter(Cb) =	1	<<<PLEASE DO NOT TOUCH THIS CELL

Location 8-5

Layer No.	Mid Layer Depth (ft)	Elevation (ft. MSL)	Depth of layer below Ground Level (ft) Top	Bottom	Main soil USCS Symbol	Total Unit Wt. (kcf)	Total Stress (ksf)	Effective Stress (ksf)	Overburden Correction w/Eff.Strs.@ Drill. time (Cn)	Field Blow Count (w/borehole diam.factor) (blows/ft)	Sampler Type	Sampler & Hammer\drop combination factor	Short Rod Correction Factor, CR	Hammer Energy ratio Factor CE	$(N_1)_{60}$ (blows/ft)	Percent Fines, % Assigned for layer	Lab	Equivalent $(N_1)_{60cs}$ for Clean Sand	Cyclic Resistance Ratio M=7.5	Cyclic Resistance Ratio for M= 7.5	Cyclic Resistance M = 7.5 (ksf)	Ind.stress Reduction Factor(rd)	Induced Stress for Mag.= 7.5 (ksf)	Theoretical Factor of Safety
1	1.00	31.0	0.0	2.0	CL-ML	0.120	0.120	0.120	1.70	3	SPT	1.000	0.75	1.25	5	52	-	Fine grained	>0.6	>0.6	#N/A	1.000	0.039	Non Liq.
2	3.50	28.5	2.0	5.0	CL-ML	0.120	0.420	0.420	1.56	7	SPT	1.000	0.75	1.25	10	52	-	Fine grained	>0.6	>0.6	#N/A	0.994	0.134	Non Liq.
3	6.50	25.5	5.0	8.0	CL-ML	0.120	0.780	0.780	1.38	21	SPT	1.000	0.75	1.25	27	52	-	Fine grained	>0.6	>0.6	#N/A	0.987	0.248	Non Liq.
4	9.00	23.0	8.0	10.0	SM	0.125	1.085	1.085	1.26	28	SPT	1.000	0.75	1.25	33	12	-	36	>0.6	>0.6	#N/A	0.981	0.343	Non Liq.
5	11.00	21.0	10.0	12.0	CL-ML	0.120	1.330	1.330	1.18	20	SPT	1.000	1	1.25	29	52	-	Fine grained	>0.6	>0.6	#N/A	0.977	0.419	Non Liq.
6	12.50	19.5	12.0	13.0	CL-ML	0.120	1.510	1.479	1.13	20	SPT	1.000	1	1.25	28	52	-	Fine grained	>0.6	>0.6	>4.5	0.974	0.474	Non Liq.
7	13.50	18.5	13.0	14.0	CL-ML	0.120	1.630	1.536	1.09	20	SPT	1.000	1	1.25	27	52	-	Fine grained	>0.6	>0.6	>4.5	0.972	0.510	Non Liq.
8	15.00	17.0	14.0	16.0	SM	0.125	1.815	1.628	1.04	24	SPT	1.000	1	1.25	31	40	40	43	>0.6	>0.6	>4.5	0.968	0.566	Non Liq.
9	18.00	14.0	16.0	20.0	SM	0.120	2.180	1.806	0.96	18	SPT	1.000	1	1.25	22	49	-	31	>0.6	>0.6	>4.5	0.962	0.675	Non Liq.
10	21.50	10.5	20.0	23.0	SM	0.120	2.600	2.007	0.88	16	SPT	1.000	1	1.25	18	49	-	26	0.316	0.316	0.634	0.953	0.798	0.79
11	24.50	7.5	23.0	26.0	SM	0.120	2.960	2.180	0.82	14	SPT	1.000	1	1.25	14	49	-	22	0.245	0.245	0.535	0.943	0.899	0.59
12	27.50	4.5	26.0	29.0	CL	0.120	3.320	2.353	0.77	14	SPT	1.000	1	1.25	13	88	88	Fine grained	>0.6	>0.6	>4.5	0.932	0.996	Non Liq.
13	30.50	1.5	29.0	32.0	CL	0.120	3.680	2.526	0.72	12	SPT	1.000	1	1.25	11	86	86	Fine grained	>0.6	>0.6	>4.5	0.917	1.088	Non Liq.
14	33.50	-1.5	32.0	35.0	CL	0.120	4.040	2.698	0.68	13	SPT	1.000	1	1.25	11	86	-	Fine grained	>0.6	>0.6	>4.5	0.900	1.171	Non Liq.
15	35.50	-3.5	35.0	36.0	SP-SM	0.125	4.283	2.816	0.66	48	SPT	1.000	1	1.25	40	7	-	40	>0.6	>0.6	>4.5	0.886	1.223	Non Liq.
16	36.50	-4.5	36.0	37.0	SP-SM	0.125	4.408	4.376	0.65	48	SPT	1.000	1	1.25	39	7	-	39	>0.6	>0.6	>4.5	0.879	1.248	Non Liq.
17	38.00	-6.0	37.0	39.0	SP-SM	0.125	4.595	4.470	0.64	48	SPT	1.000	1	1.25	38	7	-	39	>0.6	>0.6	>4.5	0.867	1.284	Non Liq.
18	40.50	-8.5	39.0	42.0	SP-SM	0.125	4.908	4.627	0.63	55	SPT	1.000	1	1.25	43	7	-	44	>0.6	>0.6	>4.5	0.846	1.337	Non Liq.
19	43.50	-11.5	42.0	45.0	SP-SM	0.125	5.283	4.815	0.61	78	SPT	1.000	1	1.25	59	7	7	60	>0.6	>0.6	>4.5	0.818	1.391	Non Liq.
20	46.50	-14.5	45.0	48.0	SP-SM	0.125	5.658	5.002	0.59	83	SPT	1.000	1	1.25	62	7	-	62	>0.6	>0.6	>4.5	0.788	1.436	Non Liq.
21	49.50	-17.5	48.0	51.0	SP-SM	0.125	6.033	5.190	0.58	87	SPT	1.000	1	1.25	63	7	-	64	>0.6	>0.6	>4.5	0.757	1.471	Non Liq.
22	51.75	-19.8	51.0	52.5	SP-SM	0.125	6.314	5.331	0.57	95	SPT	1.000	1	1.25	68	7	-	68	>0.6	>0.6	>4.5	0.734	1.493	Non Liq.
23	#N/A	#N/A	52.5	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
24	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
25	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
26	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
27	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
28	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
29	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
30	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
31	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	1.25	#N/A	#N/A	-	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

SEISMIC SETTLEMENT EVALUATION

PROJECT: 4201 Willow Street
Job No.: 09-4329

Calculated By.:	LS	Date:	2/22/2010
Checked By:	EC	Date:	2/22/2010

Location B-5

EQ.Vol.Str.Factor For Dry Sands= 0.940

Layer No.	Layer Interval (ft) Top	Layer Interval (ft) Bottom	Mid-layer depth (ft)	Thickness (ft)	Vertical Effective Stress (psf)	$(N_1)_{60}$ (blows/ft)	Rounded Equiv. $(N_1)_{60cs}$	Rounded Cyclic Str. Ratio M = 7.5 (induced)	Theoretical Factor of Safety	Dry Settlement: a	b	Gmax (ksf)	Stress, (induced) (ksf)	Gama.eff X (Geff/Gmax) (5).	Gama.eff X 100 (%)	Volumetric Strain, % M = 7.5	Volumetric Strain, % M = 7.2	Volumetric Strain (%) 3-D effect	Settlement (inches)	Wet Settlement: Volumetric Strain %	Settlement (inches)	Cumulative Settlement (inches)
1	0.00	2.00	1.0	2.00	120.00	5	Fine grained	0.325	Non Liq.	0.12556	24683	302	0.039	1.28E-04	4.50E-02	0.250	0.235	0.471	0.000			0.000
2	2.00	5.00	3.5	3.00	420.00	10	Fine grained	0.325	Non Liq.	0.12947	14585	728	0.134	1.85E-04	4.76E-02	0.106	0.100	0.200	0.000			0.000
3	5.00	8.00	6.5	3.00	780.00	27	Fine grained	0.325	Non Liq.	0.13416	11246	1375	0.248	1.80E-04	3.21E-02	0.022	0.021	0.042	0.000			0.000
4	8.00	10.00	9.0	2.00	1085.00	33	36.0	0.325	Non Liq.	0.13814	9790	1731	0.343	1.98E-04	3.41E-02	0.019	0.017	0.035	0.008			0.008
5	10.00	12.00	11.0	2.00	1330.00	29	Fine grained	0.325	Non Liq.	0.14133	8988	1844	0.419	2.27E-04	4.15E-02	0.026	0.024	0.049	0.000			0.008
6	12.00	13.00	12.5	1.00	1478.80	28	Fine grained	0.325	Non Liq.											0.00	0.00	0.008
7	13.00	14.00	13.5	1.00	1536.40	27	Fine grained	0.325	Non Liq.											0.00	0.00	0.008
8	14.00	16.00	15.0	2.00	1627.80	31	43.0	0.350	Non Liq.											0.00	0.00	0.008
9	16.00	20.00	18.0	4.00	1805.60	22	31.0	0.375	Non Liq.											0.20	0.10	0.104
10	20.00	23.00	21.5	3.00	2007.20	18	26.0	0.400	0.795											1.06	0.38	0.486
11	23.00	26.00	24.5	3.00	2180.00	14	22.0	0.425	0.595											1.30	0.47	0.954
12	26.00	29.00	27.5	3.00	2352.80	13	Fine grained	0.425	Non Liq.											0.00	0.00	0.954
13	29.00	32.00	30.5	3.00	2525.60	11	Fine grained	0.425	Non Liq.											0.00	0.00	0.954
14	32.00	35.00	33.5	3.00	2698.40	11	Fine grained	0.425	Non Liq.											0.00	0.00	0.954
15	35.00	36.00	35.5	1.00	2816.10	40	40.0	0.425	Non Liq.											0.00	0.00	0.954
16	36.00	37.00	36.5	1.00	4376.30	39	39.0	0.275	Non Liq.											0.00	0.00	0.954
17	37.00	39.00	38.0	2.00	4470.20	38	39.0	0.275	Non Liq.											0.00	0.00	0.954
18	39.00	42.00	40.5	3.00	4626.70	43	44.0	0.300	Non Liq.											0.00	0.00	0.954
19	42.00	45.00	43.5	3.00	4814.50	59	60.0	0.300	Non Liq.											0.00	0.00	0.954
20	45.00	48.00	46.5	3.00	5002.30	62	62.0	0.275	Non Liq.											0.00	0.00	0.954
21	48.00	51.00	49.5	3.00	5190.10	63	64.0	0.275	Non Liq.											0.00	0.00	0.954
22	51.00	52.50	51.8	1.50	5330.95	68	68.0	0.275	Non Liq.											0.00	0.00	0.954
23	52.50	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
24	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
25	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
26	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
27	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
28	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
29	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
30	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
31	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

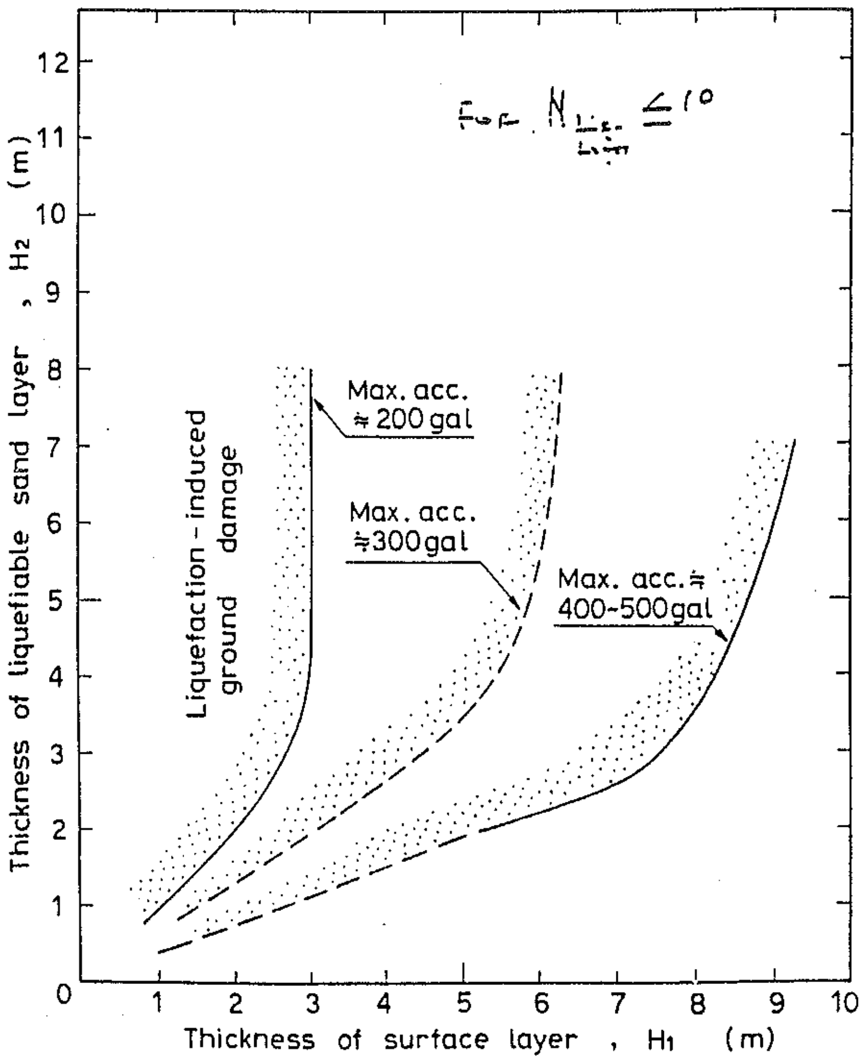


FIGURE 4-16 Proposed boundary curves for site identification of liquefaction-induced damage. Source: Ishihara (1985).

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