Appendix I

Previous Geotechnical Reports
(2010 and 2005)

October 15, 2010
Project No. 207779002

Mr. Raymond Lin
Taki-Sun, Inc.
6400 East Pacific Highway
Long Beach, California 90803

Subject: Supplemental Geotechnical Evaluation
Second & PCH Project
2nd Street and Pacific Coast Highway
Long Beach, California

Dear Mr. Lin:

In accordance with your authorization, we have performed a supplemental geotechnical evaluation for the proposed Second & PCH Project (the "Project") located at 6400 East Pacific Coast Highway (PCH) in Long Beach, California. In 2005, Converse Consultants prepared a geotechnical report (the "Converse Report") for the proposed project (referenced). The Converse report was prepared for the design of a previously proposed development, and included subsurface exploration at the site. Ninyo & Moore provided geotechnical services for the project in 2009 (Ninyo & Moore, 2009) to perform a technical review of the Converse Report and update the seismic evaluation for the Project to 2009 standards. The purpose of our 2009 technical review was to evaluate the report data with regard to the Draft Environmental Impact Report (DEIR) for the Project. We understand that the Converse report and our geotechnical review letter were included as technical documents in the DEIR for the project prepared by PCR Services Corporation dated April 2010 (referenced). The design of the Project has changed since our previous review.

The purpose of our current evaluation was to provide an assessment of potential geologic and seismic impacts associated with the Project, and to develop mitigation methods for the Project. Our current scope of work has included review of the Converse Report, review of our previous geotechnical evaluation, review of current site plans, and review of the geotechnical sections of

the DEIR. Our evaluation was performed in general accordance with the guidelines in the California Environmental Quality Act (CEQA).

PROJECT DESCRIPTION

The Project site is a roughly rectangular parcel of approximately 10.9 acres located at the southwest corner of PCH and 2nd Street. The property is bounded by 2nd Street to the north, a retail center to the south, PCH to the east, and Marina Drive to the west. The site has existing structures, is relatively flat lying and has a ground surface elevation on the order of 10 feet above mean sea level.

Based on our review of the current site plans and our understanding of the proposed project design, the Project would consist of a mixed-use development with retail, residential, hotel, restaurant and entertainment uses, and associated landscaping and open space. Buildings would generally range from two to six stories in height, with one residential tower reaching up to 12 stories. Development would also include a new roadway that would bisect the southern portion of the site. The proposed parking structure would include one level of underground parking roughly covering the boundaries of the site, as well as one at-grade level and one above-grade level of parking at the southern end of the site.

Based on our understanding of the Project, the geotechnical aspects of construction will involve excavations to reach the planned subterranean garage level at approximately 10 feet below grade. Shoring systems are anticipated to support vertical excavations around the perimeter of the proposed below-grade garage areas. In addition, construction dewatering is anticipated to maintain the excavations in a relatively dry condition during construction.

GEOTECHNICAL CONDITIONS

Based on review of referenced published geologic data and the Converse Report, the site is underlain by relatively shallow fill soils overlying unconsolidated alluvial deposits to the depths explored by Converse of approximately 81½ feet. The fill soils in the areas explored consist of silty sand and sandy silt and range from approximately ½ to 2½ feet in thickness. The alluvial

sediments at the site consist of interbedded lenses of loose to medium dense, sand, sand with silt, silty sand, sandy silt, silt, clayey sand, and clay.

Groundwater was encountered during exploration by Converse between depths of approximately 10 and 15 feet. Historical site data reported groundwater ranging from 6½ to 10 feet below the ground surface (CDWR, 2010). Fluctuations in groundwater levels are typical based on seasonal variations, precipitation, irrigation, soil types, regional groundwater pumping, and other factors.

The Project site is located in an area of relatively high seismicity, as is the majority of southern California. There are no known active faults crossing the site and the property is not located in a State of California Earthquake Fault Zone (Hart and Bryant, 1997). The active Newport-Inglewood fault zone is located approximately 1,000 feet northeast of the site. Earthquakes generated from nearby or distant fault zones will result in site ground shaking. The 2007 California Building Code (CBC) recommends that the design of structures be based on the horizontal peak ground acceleration (PGA) having a 2 percent probability of exceedance in 50 years, which is defined as the Maximum Considered Earthquake (MCE). The statistical return period for PGA_{MCE} is approximately 2,475 years. The probabilistic PGA_{MCE} for the site was calculated as 0.68g using the United States Geological Survey (USGS, 2009) ground motion calculator (web-based). The design PGA was estimated to be 0.45g using the USGS ground motion calculator.

The Project site is located in an area designated as potentially liquefiable (California Department of Conservation, Division of Mines and Geology [CDMG], 1998). The Converse report indicated that potentially liquefiable soil layers are present on site at depths of generally between 10 and 40 feet. They estimated ground surface settlement associated with liquefaction ranging from approximately 6.5 to 8.3 inches.

CONCLUSIONS

Our current evaluation included review of published geologic and seismic data, review of the Converse Report, and review of current project plans. In our opinion, the data evaluated from published geotechnical references and the site-specific data in the Converse Report are adequate for use in the planning level evaluation at this stage of the Project, and are adequate for evalua-

tion of potential geologic impacts for the Project. The Project design has changed since the preparation of the Converse Report and the report was prepared five years ago. Therefore, in general conformance with industry standard of care, a final geotechnical evaluation will be performed with additional recommendations during the detailed design phase of the Project. Based on our current evaluation, it is our opinion that the proposed Project is feasible from a geotechnical perspective. Geologic and seismic conditions exist at the site that would involve implementation of mitigation measures to reduce project impacts to less than significant levels, as outlined herein.

THRESHOLDS OF SIGNIFICANCE

According to Appendix G of the CEQA guidelines (2005), a project is considered to have a geologic impact if its implementation would result in or expose people/structures to potential substantial adverse effects, including the risk of loss, injury, or death involving hazards involving one or more of the geologic conditions presented in Table 1 below. Table 1 also presents the impact potential as defined by CEQA associated with the proposed Project and each of the geologic conditions discussed in the following section.

Table 1 – Summary of Potential Geologic Impacts/Hazards

Geologic Condition	Impact Potential[1]			
	Potentially Significant Impact	Less than Significant with Mitigation Incorporation	Less than Significant Impact	No Impact
Surface Fault Rupture			x	
Seismic Ground Shaking		x		
Seismic Ground Failure - Liquefaction		x		
Settlement or Collapse		x		
Subsidence			x	
Tsunami or Seiche		x		
Dam Failure Inundation			x	
Landslides or Mudflows				x
Soil Erosion		x		
Expansive Soil			x	
Construction Dewatering		x		
Construction Activities		x		

Note:
[1]Reference: CERES, 2005, Appendix G – Environmental Checklist Form, Final Text, dated October 26. Website: http://ceres.ca.gov/topic/envlaw/ceqa/guidelines/appendices.html

IMPACT ASSESSMENT AND MITIGATION RECOMMENDATIONS

Based on our review of geologic and seismic background information, the Converse Report, and the current Project plans, we have prepared the following assessment of potential impacts and mitigation methods for the Project.

Surface Fault Rupture

Surface fault rupture is the offset or rupturing of the ground surface by relative displacement across a fault during an earthquake. Ground surface rupture can cause damage to structures, foundations and other improvements. The Converse Report discusses the potential fault rupture hazard at the Project site. In the Faulting and Seismicity section, the report indicates that the potential for surface rupture resulting from movement of nearby major faults is moderate. In the Conclusions section, the report states that the surface rupture potential is remote. Based on further review of published geologic data and the referenced project geotechnical report, there are no active faults known to cross the site and the site is not located in a State of California Special Studies Zone. The active Newport-Inglewood fault zone is mapped approximately 1,000 feet northeast of the site. Therefore, in our professional opinion, the potential for surface rupture at the Project site is less than significant.

Seismic Ground Shaking

The site is located in a seismically active area and strong ground shaking during an earthquake on one of the nearby or distant active faults would potentially impact the Project during the design life of the proposed improvements. Ground shaking could cause damage to project improvements if the appropriate design for the anticipated level of shaking is not considered. The Converse Report provided Uniform Building Code (UBC) seismic design coefficients, which are no longer used. Our evaluation included updated seismic evaluation in accordance with the 2007 CBC. In our opinion, the impacts associated with strong ground shaking would be less than significant with incorporation of mitigation involving appropriate structural design by a qualified structural engineer in accordance with the current CBC design criteria. CBC seismic design factors will be developed for use by the project struc-

tural engineer. Site response spectra shall be developed for the planned buildings, if requested by the project structural engineer.

Seismic Ground Failure – Liquefaction

Liquefaction is a phenomenon in which soil loses its shear strength for short periods of time during an earthquake. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid increase in pore water pressure, causing the soil to behave as a fluid for short periods of time. The effects of liquefaction can include excessive total and/or differential settlement of structures founded on the liquefying soils. To be susceptible to liquefaction, a soil is typically cohesionless, with a grain-size distribution of a specified range (generally sand and silt), loose to medium dense, below the groundwater table, and subjected to a sufficient magnitude and duration of ground shaking.

According to Seismic Hazards Zones Maps published by the State of California (CDMG, 1999), the site is located within an area considered susceptible to liquefaction. The Converse Report concludes that the existing soil conditions at the site are potentially liquefiable during a strong earthquake event. The Converse report estimates dynamic ground settlement to range between approximately 6.5 to 8.3 inches due to liquefaction. Based on review of referenced geologic data and the Converse Report, we concur that without mitigation, the Project site has potential for liquefaction-induced settlement. However, with incorporation of appropriate mitigation methods, the impact of the potential liquefaction hazard at the Project site is less than significant.

Mitigation alternatives for liquefaction include supporting structures on cast-in-place pile foundations or driven pre-cast piles that extend through the liquefiable zones into competent material. Alternatively, densification of the liquefiable soils using vibro-displacement stone columns or compaction grouting would mitigate the liquefaction hazard, and the new structures could then be supported on shallow foundation systems. From a geotechnical engineering perspective, each of these alternative methods suggested in the Converse Report are considered feasible, and would reduce the liquefaction hazard impact to less than significant levels.

Settlement or Collapse

Loose alluvial soils or undocumented/poorly compacted fill may be present in some areas at the Project site. Compressible natural soils and undocumented fills pose the risk of adverse settlement under static loads imposed by new foundations and structures. Differential settlement of soils can cause damage to foundations, buildings and other project improvements. However, with incorporation of appropriate mitigation methods, the impact of settlement at the Project site is less than significant. Our review indicates that the site soils are not susceptible to hydro collapse settlement. To mitigate potential settlement at the Project site, the proposed structures that are supported on pile foundations or shallow foundations with liquefaction mitigation shall be designed to limit settlement to acceptable levels (less than one inch) so that structures are not adversely impacted. In addition, mitigation techniques such as removal and recompaction of compressible soils and in-situ ground modification will be utilized if needed based on detailed design stage recommendations to reduce the settlement potential to less than significant levels.

Subsidence

Subsidence is characterized as a sinking of the ground surface relative to surrounding areas, and can generally occur where deep soil deposits are present. Subsidence in areas of deep soil deposits is typically associated with regional groundwater withdrawal or other fluid withdrawal from the ground such as oil and natural gas. Subsidence can result in the development of ground cracks and damage to foundations, buildings and other improvements. The City of Long Beach Seismic Safety Element includes information and maps regarding regional subsidence associated with oil and gas withdrawal including the locations and magnitude of known subsidence. The Project site is not located in an area of mapped subsidence. Therefore, the impacts due to subsidence are considered less than significant.

Tsunami or Seiche Hazard

Tsunamis are open-sea waves generated by earthquakes that can impact low-lying coastal areas. Water surge caused by tsunamis is measured by distance of run-up on the shore. A seiche is the seismically induced sloshing of water in a large enclosed basin, such as a lake,

reservoir, or bay. The Converse Report indicates that the site is not subject to the potential impacts of a tsunami or seiche. However, further review of the County of Los Angeles Safety Element, City of Long Beach Seismic Safety Element, and California Emergency Management Agency Tsunami Inundation Map indicates that the project site is located in an area that is susceptible to tsunami and seiche hazards. Although the site is located in an area mapped as susceptible to tsunami and seiche hazards, with the incorporation of appropriate mitigation techniques, the tsunami/seiche hazard at the site is less than significant.

Tsunamis (and seiches) are relatively uncommon hazards in California. During historic time, seven significant tsunamis have been recorded in California (City of Long Beach, 1988). In southern California, a significant tsunami was associated with the 1960 Chile Earthquake. Damage occurred in the Long Beach-Los Angeles Harbor, where 5-foot-high waves surged back and forth in channels, causing damage to small boats and yachts. Although the site is located in an area mapped as susceptible to tsunami and seiche hazards, it is more probable that a tsunami or seiche would have damaging impacts to the adjacent harbor areas, which provide some protection to the subject property. Mitigation of tsunami/seiche hazards includes structural engineering evaluation, strengthening of seafront structures and providing emergency warning systems. Tsunami warning systems include the seismic Sea-Wave Warning System for the Pacific Ocean operated by a cooperative program of nations around the Pacific Rim and the Alaska Tsunami Warning Center operated by the National Weather Service. Structural reinforcement at the site can be included for tsunami protection, as deemed appropriate at the detailed design stage by the project structural engineer.

Dam Failure Inundation

The Converse Report indicates that there are no significant up-gradient lakes or reservoirs with a potential for flooding the site. However, based on further review of the County of Los Angeles Safety Element and the City of Long Beach Seismic Safety, the project site is mapped in an area subject to flooding from a failure of the Whittier Narrows Dam or the Prado Dam. Inundation due to dam failure could cause damage to the Project site. However, dams in California are monitored by various governmental agencies (such as the State of

California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design and construction practices, and ongoing programs of review, modification, seismic retrofitting or total reconstruction of existing dams (including recent reconstruction of the Prado Dam) are intended to see that dams are capable of withstanding the maximum credible earthquake for the site. The Whittier Narrows Dam is located approximately 20 miles from the Project site and the Prado Dam is located approximately 30 miles from the site. In addition, drainage channel systems for the San Gabriel River and Los Cerritos Channel are provided in the site vicinity to alleviate flooding conditions. Due to the regulatory monitoring of dams, nearby drainage channels, and the site distances from these dams, the potential impact of inundation due to dam failure is less than significant.

Landslides or Mudflows

Landslides, slope failures, and mudflows of earth materials predominately occur where slopes are too steep and/or the earth materials too weak to support themselves. The subject property is relatively flat and there are no significant slopes within the boundaries of the proposed project site, nor are significant slopes proposed for project development. Therefore, no landslide or mudflow impacts would occur.

Soil Erosion

Soil erosion refers to the process by which soil or earth material is loosened or dissolved and removed from its original location. Erosion can occur by varying processes and can occur in the project area where bare soil is exposed to wind or moving water (both rainfall and surface runoff). The processes of erosion are generally a function of material type, terrain steepness, rainfall or irrigation levels, surface drainage conditions, and general land uses. Construction of the Project would result in ground surface disruption during demolition, excavation, grading, and trenching that would create the potential for erosion to occur. However, with incorporation of appropriate mitigation methods, potential soil erosion would have a less than significant impact.

To mitigate potential erosion at the Project site, a Storm Water Pollution Prevention Program (SWPPP) incorporating Best Management Practices (BMPs) for erosion management is required prior to the start of construction. In addition, the topographic gradients at the project site are relatively gentle. The site would be covered with hardscape and landscape improvements following construction, and the impact of long-term erosion would be less than significant.

Expansive Soil

The Project site is underlain predominantly by granular sands and silts. Laboratory testing presented in the Converse Report indicated an Expansion Index of 26, which indicates a low potential for soil expansion. Accordingly, expansive soils are considered to have a less than significant impact at the site.

Construction Dewatering

Review of published references indicates that the depth to groundwater at the site has been observed as shallow as 6½ feet. Excavations on the order of 10 feet deep are anticipated for the planned below-grade parking garage, and construction dewatering would be involved to maintain the excavations in a relatively dry condition. Lowering the groundwater results in an increase in the effective stress of soil above the groundwater and, in some cases, can result in soil settlement. The Converse Report states that soil settlement would be more at the location of the dewatering system, but would decrease away from the excavations. Estimates of the magnitude of potential settlement will be made prior to site excavation based on detailed parking garage design, and mitigation recommendations would be implemented, as needed. The potential impacts of settlement related to construction dewatering would be less than significant with incorporation of appropriate mitigation methods.

Mitigation methods include limiting the depth of construction dewatering, installation of sheet piles and pumping from within the excavation to reduce the impacts outside the excavation, installation of monitoring wells to evaluate groundwater, monitoring adjacent areas

for indications of settlement, and/or protection to settlement-sensitive structures through ground improvement or foundation underpinning (if appropriate).

Construction Activities

The construction activities to mitigate the liquefaction hazard on site could pose additional site impacts. Depending on the type of liquefaction hazard mitigation selected, construction-related vibrations could impact surrounding properties. In addition, excavations for the below-grade parking structure are anticipated to include shoring systems, which could involve the use of driven sheet piles. The installation of driven piles or vibro-displacement stone columns for liquefaction mitigation, and installation shoring systems (such as sheet piles) involves construction vibrations, which can result in disturbance to people and/or ground settlement.

Based on review of site conditions and Project plans, an existing commercial building is located approximately 50 feet from the southeast side of the Project site. Off-site structures are not located within 100 feet of the other sides of the site. Sensitive receptors (people and structures) located within approximately 50 to 100 feet of the Project and could be impacted by vibrations and ground settlement. However, the impacts of vibrations and ground settlement to surrounding improvements due to construction activities at the project site will be reduced to less than significant levels with incorporation of mitigation techniques.

In order to mitigate the potential impacts due to vibrations during the construction phase of the Project, sensitive receptors (people and structures) within approximately 50 to 100 feet of the Project would be evaluated with regard to potential vibration-related impacts. If vibrations would impact the receptors, mitigation techniques shall be implemented at that time. Mitigation techniques to reduce the impacts of vibrations to less than significant levels include avoiding vibratory types of construction, limiting vibratory types of construction to specified distances from sensitive off-site receptors, monitoring vibration and settlement during construction, and/or protecting sensitive improvements from excessive settlement by ground stabilization or foundation underpinning.

Monitoring methods include installation of ground survey points around the outside of excavations to monitor settlement and/or placing monitoring points on nearby structures or surfaces to monitor performance of the structures. In general, acceptable levels of settlement would be ½ inch or less in non-building areas, and ¼ inch or less for building areas. If monitored movement is unacceptable to surrounding improvements during the course of construction, the work shall stop and the contractor's methods shall be reviewed and changes made, as appropriate; and alternative methods of settlement reduction shall be implemented by the contractor.

We appreciate the opportunity to be of service on this project.

Respectfully submitted,
NINYO & MOORE

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Senior Project Geologist

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Attachment: References

Distribution: (1) Addressee (via-email)
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AERIAL PHOTOGRAPHS

Source	Scale	Date	Flight	Numbers
USDA	1:20,000	11-18-52	AXK-1K	3 and 4

Converse Consultants

Over 50 Years of Dedication in Geotechnical Engineering and Environmental Sciences

September 1, 2005

Mr. Bob Garrison
Mr. Ryan Davis
Lennar Communities
25 Enterprise, Suite 300
Aliso Viejo, CA 92656

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Mixed-Use Community – Seaport Marina
Southeast Corner of 2nd Street and East Pacific Coast Highway (PCH)
Long Beach, California
Converse Project No. 04-31-118-01

Dear Mr. Garrison/ Mr. Davis:

Converse Consultants (Converse) has prepared this report to present the results of our geotechnical investigation for the proposed Mixed-Use Community located in Long Beach, California. This report has been prepared in accordance with our proposal dated February 20, 2004, and your written authorization via e-mail dated February 23, 2004.

It is our opinion that the subject site can be developed from a geotechnical standpoint for the proposed development, provided the findings, conclusions, and recommendations presented in this report are considered in the preparation of the grading plan, foundation design, and construction of the project. Once the existing structures are demolished and the type and location of new structures are established, we may have to investigate further prior to preparing the final report. Foundation, shoring and grading plans should be reviewed for conformance with the recommendations presented in this geotechnical investigation report.

We appreciate the opportunity to be of service to the Lennar Communities. If you should have any questions, please do not hesitate to contact us at (626) 930-1200.

CONVERSE CONSULTANTS

William H. Chu, P.E., G.E.
Senior Vice President/Principal Engineer

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EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions and recommendations as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The project will consist of three one to four-story buildings on one to two-story subterranean garages for condominium homes with commercial/retail spaces. All buildings are proposed to be supported by either shallow or deep foundations.

- The relatively level site is approximately 10.9 acres and is located on the southeast corner of 2nd Street and East Pacific Coast Highway (PCH) at the City of Long Beach, California. The western boundary of the site is Marina Drive. Commercial building is located at the southern boundary of the proposed site.

- The scope of our present investigation included a review of existing information, site reconnaissance, subsurface exploration borings, Cone Penetration Testing (CPT), soil sampling, laboratory testing, engineering and geological analyses, and preparation of this report.

- A total of five (5) borings (BH-1 through BH-5) were drilled within the project site boundaries. The depth of the borings ranged from 61.5 to 81.5 feet below the existing ground surface. A total of four (4) Cone Penetration Testing (CPT-1 through CPT-4), were penetrated to depths ranging from 60 to 72 feet below the existing ground surface.

- Laboratory testing consisted of moisture-density determinations, sieve analysis, percent passing #200, Atterberg Limits, maximum dry density-optimum moisture content, direct shear, consolidation, expansion index, sand equivalent, R-value, and soil corrosivity. These laboratory tests were performed for soil classification purposes and evaluation of relevant physical characterizations and engineering properties.

- Fill material at the site range from 0.5 to 2.5 feet below the existing ground surface (bgs). Slightly oil odor was observed at Borings BH-1, BH-4 and BH-5 within 5 to 10 feet below existing ground surface. Thicker fill may be encounter under the existing buildings. The fill materials mainly consist of silty sand and sandy silt. Alluvial deposits underlie the fill material to the maximum explored depth of 81.5 feet below the existing ground surface. The alluvial deposits within the project site generally consist of silty sand, sandy silt, silt, clay, clayey sand, and sand with silt.

- Groundwater was encountered at all borings at depths ranging from ten (10) to fifteen (15) feet below existing ground surface during drilling of exploratory borings. Based on the historical data of the water wells around the site and reports prepared by others, the

water level at the area around the site range from 6 ½ to 10 feet below the ground surface. Groundwater needs to be considered in the design and construction of the proposed project.

- There are no known active faults projecting toward or extending across the project site. The site is not situated within a currently designated State of California Earthquake Fault Zone. The nearest known active faults to the site are the Newport-Inglewood (L.A. Basin) fault, located approximately 0.5 miles (0.8 kilometers) to the northeast, and Palos Verdes fault, located approximately 7.7 miles (12.3 Kilometers) to the southwest of the site. In accordance with California Building Code (CBC, 2001), the geologic subgrade classification will be S_D in accordance with Table 16-J. The seismic coefficients at the site in accordance with Tables 16-Q through 16-T are:

  N_a=1.3 C_a=0.57
  N_v=1.6 C_v=1.02

- Seismic Hazard Zone Maps indicate that the proposed site is located within the liquefaction zone. Due to the shallow groundwater, anticipated ground acceleration, and isolated layers of low to medium dense sandy soils, the proposed site is considered susceptible to liquefaction.

- Before the excavation, the total seismically induced settlement ranges from 6.5 inches to 8.3 inches, and the differential settlement considered to be half of the total settlement. Considering 25 feet over excavation for the two-story subterranean parking, the total seismically induced settlement ranges from 2.6 inches to 5.7 inches, and the differential settlement can be assume to be half of the total settlement.

- The site soils are not susceptible to hydro-consolidation.

- The corrosion potential of the soil in contact with concrete is low. Type I or II Portland cement may be used. The corrosion potential of the soil in contact with ferrous metal is severe. A corrosion engineer should be consulted to provide mitigative measures.

- The expansion potential of the site soils is low.

- All existing structures within the proposed structures should be demolished and removed from the site. All deleterious material debris and surficial soils containing roots and perishable materials should be removed from the site.

- For the pavement, sidewalk and concrete slab areas, the upper three (3) feet should be removed and replaced with compacted fill. The fill soils should be brought to moisture content within ±2.0 percent of optimum and compacted to at least 90 percent relative compaction as per ASTM Standard D1557-00 test method.

- The below-grade pipelines can be founded on either firm native soils or compacted fill.

- The actual depth of removal should be determined based on observations made during grading. Some over-excavation may be required to remove unsuitable loose soils and to prepare a suitable compacted structural fill mat to receive new fill and/or structures, including piles, pile caps, grade beams, buildings slabs-on-grade and pavement.

- The existing subsurface on-site soil has moisture content above optimum; therefore moisture conditioning is necessary to reduce the moisture to ±2 of the optimized moisture content.

- Placement of base material or geo-grid material may be required for subgrade soft bottom of excavation.

- Temporary dewatering will be required for the project during construction. The dewatering contractor should conduct independent test to determine dewatering quantity.

- Based on our field exploration data, excavation of site soils may be accomplished with conventional heavy-duty excavation equipment.

- The bottom of the excavation should be scarified to a depth of at least 12 inches. The scarified soils should be brought to near-optimum moisture content and compacted to at least 90 percent of the laboratory maximum dry density to produce a firm and unyielding surface. Fill should then be placed on the compacted natural soil in loose lifts of eight (8) inches or less, moisture conditioned to near-optimum moisture content, and compacted to at least 90 percent relative compaction.

- The majority of the site soils may be utilized as both structural and nonstructural fill.

- At least the upper 12 inches of subgrade soils underneath the foundations, slabs and pavements should be compacted to a minimum of 95 percent relative compaction as per ASTM Standard D1557-00 test method.

- Without soil improvement to mitigate the liquefaction potential, the proposed structures have to be supported on either drilled cast-in-place or driven piles. Shallow footings or mat foundation may be used with proper liquefaction potential mitigation measure. Shallow foundations should be supported on firm natural soils or on compacted fill.

- Cantilevered earth retaining walls should be designed based on an active earth pressures equal to that developed by a fluid density of 35 pounds per cubic foot (pcf). These pressures assume a level ground surface behind the walls for a distance greater than the wall height. Hydraulic pressure should be considered for the wall design. For the hydraulic pressure, the incremental earth pressure below the groundwater level should be

reduced by 50 percent and added to hydrostatic pressure for total lateral pressure. Retaining wall needs to be water proofed in order to prevent intrusion of moisture.

- Resistance to lateral loads may be provided by the passive earth pressures acting behind the footings and by the frictional resistance at the base. An allowable value of the passive earth pressure resistance of 250 psf per foot of footing depth may be used for design. The passive resistance should be limited to a maximum of 2,500 psf. An ultimate value of the frictional coefficient of 0.35 may be used to evaluate base frictional forces between soil and concrete.

- The soils encountered during this investigation are relatively sandy and groundwater was encountered at depths varying from 10 to 15 feet below the ground surface. As a result, caving of the sidewalls can be expected during the drilling and construction of the cast-in-drilled-hole piles.

- A tied-back soldier-pile shoring system may be used to maintain temporary support of deep vertical wall excavations. Braced or tied-back shoring, retaining a level ground surface, should be designed for a uniform pressure of 26H psf, where H is the height of the retained cut in feet.

- Results of our investigation indicate that the site is suitable from a geotechnical standpoint for the proposed development, provided that the findings, conclusions, and recommendations contained in this report are incorporated into the design and construction of the project.

This report is prepared solely for geotechnical engineering purpose and the use for Lennar Communities, and the project design team, and may not contain sufficient information for use by others for other purposes.

PROFESSIONAL CERTIFICATION

This report has been prepared by the staff of Converse under the professional supervision of the individuals whose seals and signatures appear hereon.

The findings, recommendations, specifications, or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of southern California. There is no warranty, either expressed or implied.

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TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 PROJECT DESCRIPTION 2

3.0 SCOPE OF WORK 2

3.1 LITERATURE REVIEW 2
3.2 FIELD EXPLORATION 3
3.3 CONE PENETRATION TESTING 3
3.4 LABORATORY TESTING 3
3.5 REPORT PREPARATION 3

4.0 SITE CONDITIONS 4

4.1 SITE DESCRIPTION 4
4.2 SUBSURFACE CONDITIONS 4
4.3 LABORATORY TEST RESULTS 4
4.4 GROUNDWATER 5
4.5 SUBSURFACE VARIATIONS 6

5.0 FAULTING AND SEISMICITY 6

5.1 FAULTING 6
5.2 SEISMICITY 6
5.3 UBC NEAR SOURCE PARAMETERS 7
5.4 LIQUEFACTION EVALUATION AND MITIGATION RECOMMENDATIONS 7
5.5 OTHER EFFECTS OF SEISMIC EVENTS 9

6.0 SOIL CORROSIVITY EVALUATION 10

7.0 CONCLUSIONS 11

8.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS 12

8.1 GENERAL 12
8.2 COMPACTION 14
8.3 RECOMMENDED SPECIFICATIONS FOR PLACEMENT OF COMPACTED FILL 14
8.4 STRUCTURAL BACKFILL 16
8.5 BURIED UTILITIES 16
8.6 DEWATERING 18

9.0 DESIGN RECOMMENDATIONS 19

9.1 GENERAL EVALUATION 19
9.2 FOUNDATION TYPE AND BEARING PRESSURES 19
9.3 SLABS-ON-GRADE 23
9.4 SETTLEMENT 24
9.5 SITE DRAINAGE 24
9.6 LATERAL EARTH PRESSURES AND RESISTANCE TO LATERAL LOADS 25
9.7 GROUNDWATER CONTROL 25
9.8 CONSTRUCTION CONSIDERATIONS 26
9.9 ASPHALT CONCRETE PAVEMENT 29

10.0 PLAN REVIEW 30

11.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION 30

12.0 CLOSURE 31

REFERENCES 32

TABLES

Table No. 1, *Nearby Active Faults* 6
Table No. 2, *Potential for Secondary Effects of Seismic Activity* 10
Table No. 3, *Recommended Pile Design Parameters for Lateral Loads* 20
Table No. 4, *Slope - Temporary Excavation* 26
Table No. 5, *Recommended Pavement Sections* 29

ILLUSTRATIONS

Figures

Following Page No.:
Figure No. 1, *Site Location Map* 1
Figure No. 2, *Site Plan and Approximate Boring/CPT Location Map* 2
Figure No. 3, *Liquefied Zone at Cross Section A-A'* 7
Figure No. 4a, *Friction Load Capacity (Driven Piles)* 19
Figure No. 4b, *Friction Load Capacity (Cast-in-place Piles)* 19
Figure No. 5, *Recommended Lateral Earth Pressure on Permanent Retaining Wall* 25
Figure No. 6, *Schematic Tie-back Design* 28

APPENDICES

Appendix A ..... *Field Exploration*
Appendix B ..... *Laboratory Testing Program*
Appendix C ..... *Liquefaction Analysis*
Appendix D ..... *Guide Specifications for Drilled/Driven Pile Installation*
Appendix E ..... *Guide Specifications for Installation and Acceptance of Tieback Anchors*

1.0 INTRODUCTION

This report contains the findings of our geotechnical investigation performed at approximately 10.9 acres site, located at southeast corner of 2nd Street and East Pacific Coast Highway (PCH) in Long Beach, California as shown in Figure No. 1, *Site Location Map.*

This report is prepared under the following assumptions and conditions:

- The existing structures on-site are slab-on-grade with shallow foundation system.
- For the proposed structures, it was assumed that the maximum column load will be on the order of 200 kips for dead and live loads, and the maximum continuous wall load will be in order of 6 Kips per lineal foot.
- A site-grading plan, structural layout was not available at the time this report was prepared. The site earthwork and design recommendations provided in this report should be considered preliminary and are based on our understanding of the project and on experience with similar projects in the area, as well as the on-site investigation.
- The purpose of this investigation was to evaluate the nature and pertinent engineering properties of the subsurface materials at the site and to provide recommendations regarding general site grading, foundation design, and construction. Our scope of work was not included an evaluation of potential for soil and/or groundwater contamination at the site and does not have sufficient information for final design purpose. Additional investigation will be needed once the building lay-out, grading plans and structural loads are available.
- Environmental study is currently been performed by others.
- Final earthwork building plans should be reviewed for compliance with our recommendations.

Additional investigation will be needed once the building lay-out, grading plans and structural loads are available.

An electronic file of a site plan, entitled, *Alta Survey for Avalonbay Communities, Inc.,* Prepared by Huitt-Zollars, Inc., dated November 12, 2001, was provided to us on March 30, 2005 and was used as a base map for our investigation.

This report is written for the project described herein and is intended for use solely by Lennar Communities, and its design team. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

2.0 PROJECT DESCRIPTION

The project will consist of three, one to four-story condominium homes with commercial/retail spaces. One to two-story subterranean parking will be constructed under, almost the entire site. All buildings will be supported on deep foundations.

Exact number of buildings, grading plans, building layout and structural loads were not available at the time of our preliminary investigation.

3.0 SCOPE OF WORK

The scope of our present investigation included a review of existing information, site reconnaissance, subsurface exploration borings, Cone Penetration Testing (CPT), soil sampling, laboratory testing, engineering and geological analyses, and preparation of this report. The scope of work included the following tasks:

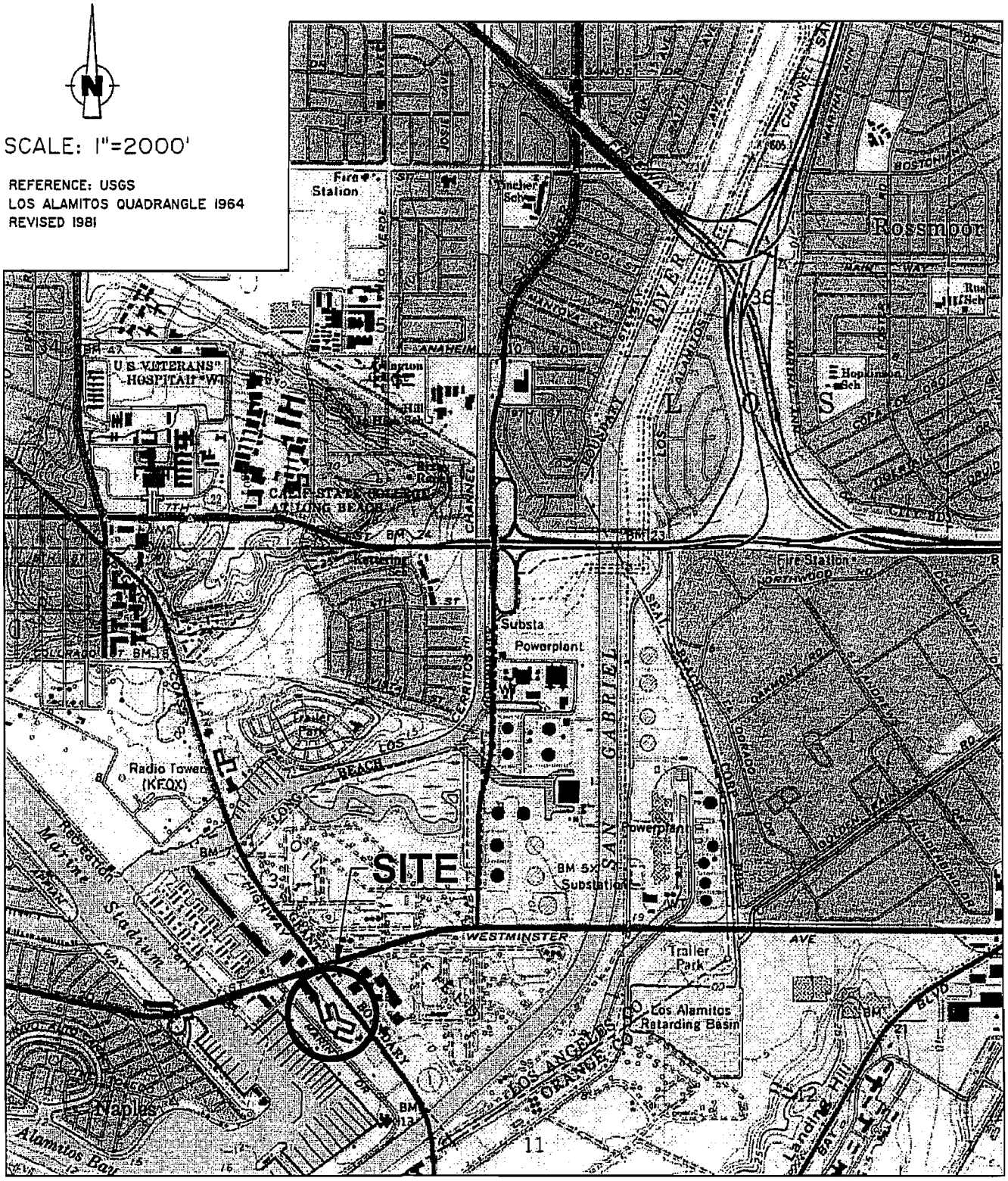
3.1 Literature Review

As part of this investigation, we have reviewed available pertinent environmental, geologic and geotechnical reports and maps included in the References section of this report. The following reports were provided by your office and were used to evaluate the existing conditions of the proposed site:

- Feasibility-Level Geotechnical Evaluation, *Proposed Avalon Naples Apartment Development, 6400 East Pacific Highway, Long Beach, California*, prepared by Geotechnical Professionals Inc., December 13, 2001.

- Subsurface Site Assessment – Phase II Geophysical, Soil and Soil Vapor Survey, *Seaport Marina Hotel – Avalon Naples Project, 6400 East Pacific Coast Highway, Long Beach, California,* prepared by California Environmental, January 2002.

- Draft Phase I Environmental Site Assessment Report, *2680 East 2nd Street and 6400 East Pacific Coast Highway, City of Long Beach, Los Angeles County, California*, prepared by Leighton and Associates, Inc., April 2004.



SITE LOCATION MAP

MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.

04-31-118-01

Figure No.

1

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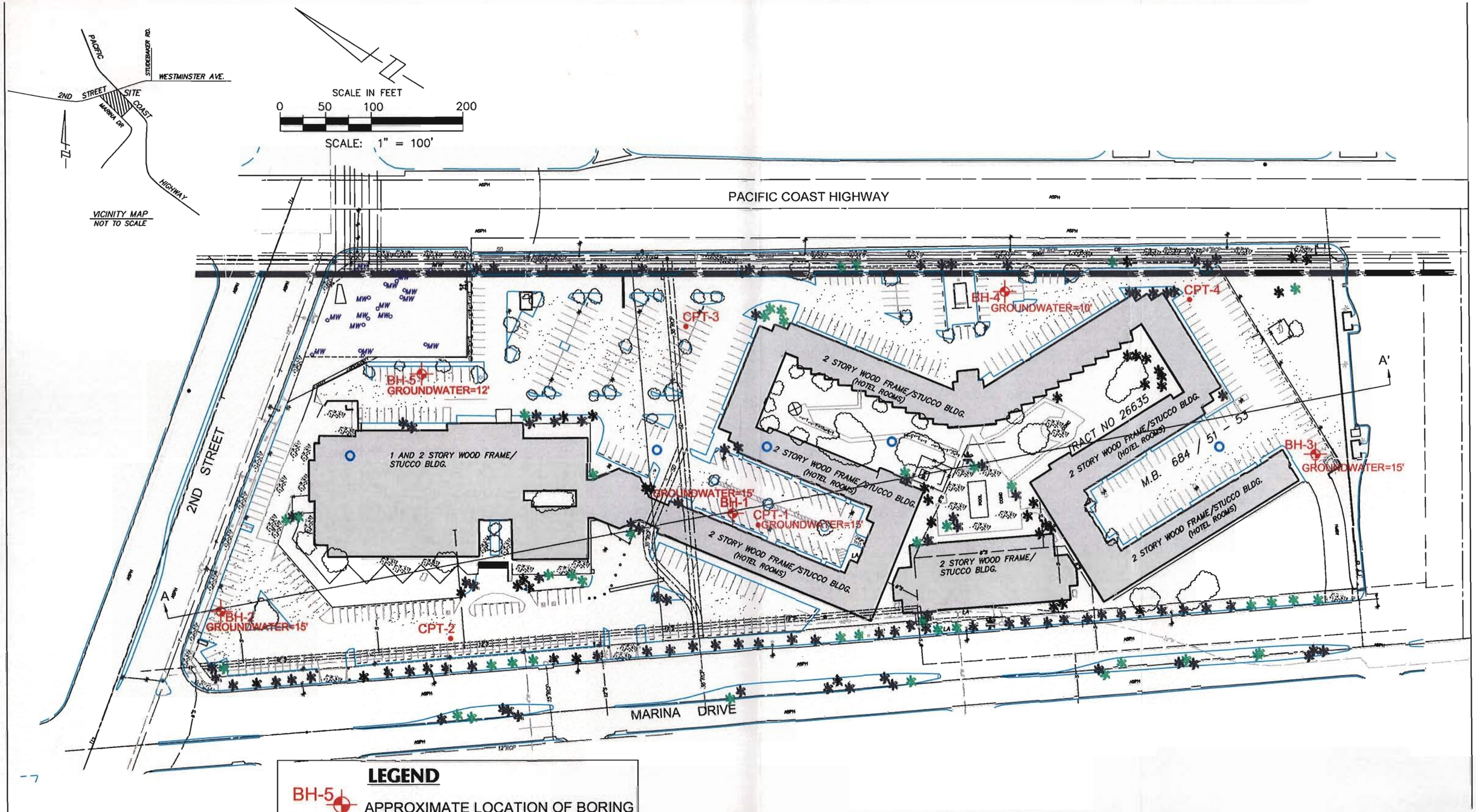

SITE PLAN AND APPROXIMATE BORING / CPT LOCATION MAP

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FOR: LENNAR COMMUNITIES

Project No.	Figure No.
04-31-118-01	2

3.2 Field Exploration

A total of five (5) borings (BH-1 through BH-5) were drilled within the project site boundaries. The depth of the borings ranged from 61.5 to 81.5 feet below the existing ground surface. The exploratory borings were advanced on March 2 and 3, 2004, using an 8-inch diameter hollow-stem auger drill rig. The approximate locations of the boring shown in Figure No. 2, *Site Plan and Approximate Boring/CPT Location Map.*

An engineer visually logged the subsurface conditions encountered in the exploratory borings at the time of drilling. Relatively undisturbed ring and bulk samples of the subsurface materials were obtained from the borings at frequent intervals for the purpose of laboratory testing. A more detailed description of the field exploration procedures and the logs of borings are presented in Appendix A, *Field Exploration.*

3.3 Cone Penetration Testing

A total of four (4) Cone Penetration Testing (CPT-1 through CPT-4), were penetrated to depths ranging from 60 feet to 72 feet below the existing ground surface. The CPT locations are shown on Figure No. 2, *Site Plan and Approximate Boring/CPT Location Map.*

The purpose of the CPTs was to obtain a continuous profile of the subsurface conditions and use them for evaluation of the liquefaction potential of the proposed site. The tests were performed by sub-consultant, Furgo Geosciences, Inc., Santa Fe Springs, California, 90670.

The CPTs were advanced on March 2, 2004. CPT results are included at the end of Appendix A, *Field Exploration.*

3.4 Laboratory Testing

Laboratory tests were performed on selected soil samples. The tests included moisture-density determinations, sieve analysis, percent passing #200, Atterberg Limits, maximum dry density-optimum moisture content, direct shear, consolidation, expansion index, sand equivalent, R-value, and soil corrosivity. Descriptions and results of the laboratory tests are presented in Appendix B, *Laboratory Testing Program.*

3.5 Report Preparation

Data obtained from the field exploration and laboratory testing program were evaluated. Geotechnical analyses were performed and this report was prepared to present our findings, conclusions, and recommendations for the proposed project.

4.0 SITE CONDITIONS

4.1 Site Description

The site is located on the southeast corner of 2nd Street and East Pacific Coast Highway (PCH) at the City of Long Beach, California. The western boundary of the site is Marina Drive. Commercial building is located at the southern boundary of the proposed site. The Long Beach Yacht Club is located about 200 feet west of the proposed site.

The relatively level site is approximately 10.9 acres. An Enterprise Car Rental office and Seaport Marina Hotel currently occupy the site. The rest of the site is covered with asphalt concrete and is used for vehicle parking. A small area is landscape. It is assumed that all existing structures have shallow foundations less than 5 feet in depths.

Based on our review of the above-mentioned reports, site was used to extract oil during 1926-1968. Five abandoned Chevron oil wells exist at the site. Petroleum pipeline operated by Chevron as well as several others exists along the north boundary of the site. Numerous groundwater monitoring wells and extraction wells exists in the area of a former gas station at the northeast corner of the site.

4.2 Subsurface Conditions

Fill material ranging from 0.5 to 2.5 feet below the existing ground surface (bgs) was encountered at the site. Slightly oil odor was observed at Borings BH-1, BH-4 and BH-5 within 5 to 10 feet below existing ground surface. Thicker fill may be encountered under the existing buildings or within the other areas of the site. The fill materials were likely associated with construction of existing buildings and mainly consist of silty sand and sandy silt. Alluvial deposits underlie the fill material to the maximum explored depth of 81.5 feet below the existing ground surface as shown in the logs of borings included in Appendix A, *Field Exploration.* The alluvial deposits within the project site generally consist of silty sand, sandy silt, silt, clay, clayey sand, and sand with silt. For additional information on the subsurface conditions see Appendix A, *Field Exploration.*

4.3 Laboratory Test Results

Results of *in-situ* moisture and density tests are presented in the Logs of Borings in Appendix A, *Field Exploration.*

Result of the sieve analyses performed on one (1) representative soil sample is shown in Drawing No. B-1, *Grain Size Distribution Results*, in Appendix B, *Laboratory Testing Program.*

Results of the soils finer than #200 sieve performed on twenty seven (27) representative soil samples are shown in the Logs of Boring in Appendix A, *Field Exploration*. These results are used to aid soil classification and to evaluate the liquefaction potential of the site soil. These results are also summarized in Table No. B-1, *Percent Finer Than #200 Sieve Test Results,* in Appendix B, *Laboratory Testing Program.*

One (1) representative fine-grained soil sample performed for Atterberg Limit test. The result is summarized in Table No. B-2, *Atterberg Limit Test Results,* in Appendix B, *Laboratory Testing Program.*

Typical moisture density relationships of the representative near surface soils are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density and the optimum moisture content of the sample tested was 121.3 pounds per cubic foot (pcf) and 12.1 percent, respectively.

Results of direct shear test performed on four (4) representative samples of the site soils are presented in Drawings Nos. B-3 through B-6, *Direct Shear Test Results*, included in Appendix B, *Laboratory Testing.* These results are also summarized in Table No. B-3, *Summary of Direct Shear Test Results.*

Two (2) consolidation test were performed on representative samples of the site soils. The test results are presented in Drawings Nos. B-7 and B-8, *Consolidation Test Results,* in Appendix B, *Laboratory Testing Program. Based on the consolidation test results, the site soils are not susceptible to hydro-consolidation.*

One (1) expansion index test was performed on representative sample of the site soils from shallow depths. The test result is presented in Table No. B-4, *Summary of Expansion Index Test Result,* in Appendix B, *Laboratory Testing Program.* Based on the test result, site soils may have a low expansion potential.

One (1) Sand Equivalent test was performed on a representative sample of the site soils. The test result is presented in Table No. B-5, *Summary of Sand Equivalent Test Result,* in Appendix B, *Laboratory Testing Program.*

One (1) R-value test was performed on a representative sample of the site soils. The test result is presented in Table No. B-6, *R-value Test Results,* in Appendix B, *Laboratory Testing Program.*

4.4 Groundwater

Groundwater was encountered within all borings at depths ranging from ten (10) to fifteen (15) feet below existing ground surface. Based on the historical (1957-2000) data of the Department of Water Resources monitoring network of the water wells located about 0.4 to 2

miles around the site, the water level at the area around the site is about ten (10) feet below the ground surface.

Based on the report prepared by others listed in section 3.1, L*iterature review*, groundwater monitor wells installed as part of the site assessment and clean up on the adjacent Unocal gas service station show a historical (1998-2000) depth to groundwater, from 6 ½ feet to 12 feet below the ground surface. According to the Quarterly Monitoring report, October through December 2003, the depth to groundwater ranges from 7.39 to 9.16 feet below ground surface.

Due to the proximity of the site to the coastal zone, the depth to groundwater beneath the site is expected to be influenced by tidal fluctuations. Also, fluctuation in the groundwater level could occur due to change in seasons, variations in rainfall, and other factors. Groundwater needs to be considered in the design and construction of the proposed project facilities.

4.5 Subsurface Variations

Based on our experience and the results of subsurface exploration, some variations in the continuity and nature of subsurface conditions at this site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of alluvial deposits, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the borings. If, during construction, subsurface conditions different than those presented in this report are encountered, this office should be notified immediately so that recommendations can be modified, if appropriate.

5.0 FAULTING AND SEISMICITY

5.1 Faulting

Based on our review of the available information, there are no known active faults projecting toward or extending across the project site. An active fault is defined as one that has had surface displacement within Holocene time (about the last 11,000 years). The site is not situated within a currently designated State of California Earthquake Fault Zone.

5.2 Seismicity

The project site is situated in a seismically active region. As is the case for most areas of Southern California, ground shaking resulting from earthquakes associated with nearby and distant faults may occur. During the life of the project, seismic activity associated with active faults in the area may generate moderate to strong ground shaking at the site.

Based on Boore et. al. (1997), random mean with one standard deviation attenuation relationship, and a 10 percent probability of exceedance in 50 years (MPE), a site-specific

ground acceleration of 0.49g was obtained. The vertical acceleration may be taken equal to two thirds of the horizontal acceleration.

5.3 UBC Near Source Parameters

The faults considered active within 30 kilometers of the project site are listed in Table No. 1, *Nearby Active Faults.*

The nearest known active faults to the site are the Newport-Inglewood (L.A. Basin) fault, located approximately 0.5 miles (0.8 kilometers) to the northeast, and the Palos Verdes fault, located approximately 7.7 miles (12.3 Kilometers) to the southwest of the site.

Table No. 1, Nearby Active Faults

Fault Name	Closest Distance to Site (mi/km)	Seismic Source Type	Moment Magnitude (Mw)	Slip Rate (mm/year)
Newport-Inglewood (L.A. Basin)	0.5/ 0.8	B	6.9	1.0
Palos Verdes	7.7/ 12.3	B	7.1	3.0
Newport-Inglewood (offshore)	16.3/ 26.0	B	6.9	1.5
Elsinore-Whittier	16.8/ 26.8	B	6.8	2.5

In accordance with California Building Code (CBC, 2001), the geologic subgrade classification will be S_D in accordance with Table 16-J. The seismic coefficients at the site in accordance with Tables 16-Q through 16-T are:

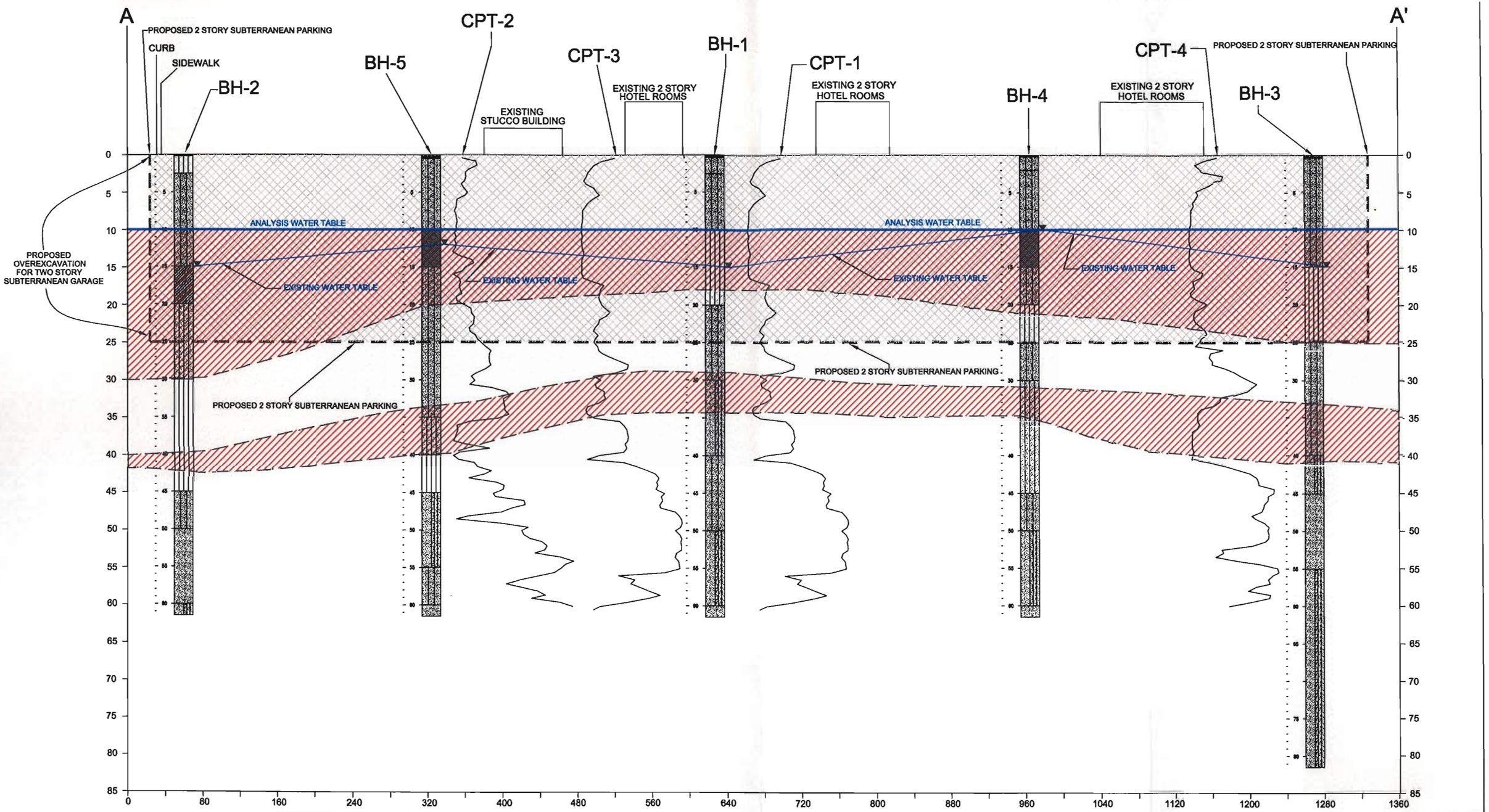
N_a=1.3 C_a=0.57
N_v=1.6 C_v=1.02

5.4 Liquefaction Evaluation and Mitigation Recommendations

5.4.1 Liquefaction Evaluation

Liquefaction is defined as the phenomenon in a soil mass, because of the development of excess pore pressures that suffers a substantial reduction in its shear strength to a constant value, and deforms continuously until the imposed shear stresses become equal to steady-state shear strength. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses resulting in liquefaction.

Soil liquefaction occurs in submerged granular soils during or after strong ground shaking. There are several requirements for liquefaction to occur. They are as follows:



LEGEND

- OVEREXCAVATION ZONE
- LIQUEFIED ZONE

LIQUEFIED ZONE AT CROSS SECTION A-A'

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Project No. 04-31-118-01

Figure No. 3

- Soils must be submerged
- Soils must be primarily granular
- Soils must be contractive, that is, loose to medium-dense
- Ground motion must be intense
- Duration of shaking must be sufficient for the soils to lose shear resistance

Seismic Hazard Zone Maps indicate that the proposed site is located within the liquefaction zone. Due to the shallow groundwater, anticipated ground acceleration, and isolated layers of low to medium dense sandy soils, the proposed site is considered susceptible to liquefaction. Detail description of liquefaction analysis is included in Appendix C, *Liquefaction Analysis.*

The liquefaction analysis was conducted for both site condition below the ground surface before the excavation, and the site soil below the two-story subterranean garage floor after removal 25 feet over excavation. Calculations indicate that during the maximum probable earthquake (10% in 50 years) as defined by the UBC, liquefaction will occur on the site between the depths of 10 and 40 feet bgs as it is presented in Figure 3, *Liquefied Zone At Cross Section A-A'.* Based upon the limited information available, it is expected that this liquefaction will be general in nature and occur over the entire site.

Research indicates that, due to the depth at which the liquefaction is expected to occur, there may be surface expression of the liquefaction in the form of sand boils and that settlement will occur over a large area. As a result, the anticipated settlement of individual structures will be predominately total settlement with moderate amount of differential settlement.

Before the excavation, the total seismically induced settlement ranges from 6.5 inches to 8.3 inches, and the differential settlement considered to be half of the total settlement.

Considering 25 feet over excavation, the total seismically induced settlement ranges from 2.6 inches to 5.7 inches, and the differential settlement can be assume to be half of the total settlement.

5.4.2 Liquefaction Mitigation

There are several methods for densifying and thus increasing the strength of loose, liquefaction susceptible soils at depth, such as stone columns and compaction grouting. Discussion on the two methods is presented below.

<u>Stone Columns</u>

Stone columns (or vibro-displacement stone columns) are used to improve the soil consist of sand, silt and clay. Soils that has been displaced laterally by the vibroflot is replaced

with crushed stone to form stone columns. Stone columns reduce the potential of liquefaction by both densification and drainage. Due to the drainage, the excess pore pressure generated during the earthquake will reduce the extent of liquefaction.

The borehole can be filled with stone from either the bottom of the hole or ground surface. Then, the stone is compacted and forced to surrounding ground. This process in repeated until the borehole is completely filled. With both methods, the penetration can be achieved with either air only or by a combination of compressed air and water.

Stone columns are very effective in sands and can be effective in silts and silty sands. The actual stone columns design and installation should be performed by experienced subcontractors. The design of ground improvement should be reviewed by Geotechnical Engineer and installed under a Geotechnical Engineer observation.

Based on discussion with contractors, the cost of stone columns installation between 30 feet to 40 feet in depth range from $6.00 to $8.00 per square feet.

Compaction Grouting

In compaction grouting, the ground conditions improve by soil displacement. First, a grout pipe casing is driven to the ground to reach the liquefied area. Then, a very viscous (low – mobility), aggregate cement grout is pumped though the pipe and the pipe is withdrawn in stages. At each stage, compaction grout will displace the adjoining soils and densify the immediately surrounding soil. The grout densify the loosest soil and treat the most susceptible material. The grout injection rate should be slow enough to allow pore pressure dissipation. Soils that lose strength during remolding (saturated, fine-grained soils, sensitive clay) should be avoided.

Based on discussion with contractors, the cost of compaction grouting between 30 feet to 40 feet in depth ranges from $7.50 to $10.00 per square feet.

5.5 Other Effects of Seismic Events

Besides generating damaging ground motion, a nearby seismic event may impact a project by inducing landslides, earthquake-induced flooding, tsunamis, seiches, soil liquefaction, differential settlement and ground lurching. A site-specific discussion on each of the above secondary effects is provided below:

Landslides: Seismically induced landslides and slope failures are common occurrences during or soon after large earthquakes. The project site is on relatively flat terrain. The potential for seismically induced landslides affecting the proposed site is considered to be very low.

Earthquake-Induced Flooding: This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Review of the area adjacent to the site indicates that there are no significant up-gradient lakes or reservoirs with the potential of flooding the site.

Tsunamis: Tsunamis are tidal waves generated by fault displacement or major ground movement. Based on the location of the site, tsunamis do not pose a hazard to this site.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Review of the area adjacent to the site indicates that there are no significant up-gradient lakes or reservoirs with the potential of flooding the site.

Surface Fault Rupture: The project site is not located within a currently designated State of California Earthquake Fault Zone. Based on review of existing geologic information, no major surface fault crosses through or extends towards the site. However, the Newport-Inglewood (L.A. Basin) fault is located approximately 0.5 miles to the northeast of the site, therefore, the potential for surface rupture resulting from the movement of nearby major faults is considered moderate.

Differential Settlement and Ground Lurching: The potential of significant differential settlement and ground lurching during earthquakes is considered to be moderate at the site.

The potential of various secondary effects of seismic activity is summarized in Table No. 2, *Potential for Secondary Effects of Seismic Activity.*

Table No. 2, Potential for Secondary Effects of Seismic Activity

Secondary Seismic Hazard	Potential Risk
Landslides	None
Tsunamis	None
Seiches	None
Earthquake-Induced Flooding	Low
Surface Fault Rupture	Moderate
Liquefaction	High
Differential Settlement	Moderate

6.0 SOIL CORROSIVITY EVALUATION

One (1) representative soil sample was tested by EGL Associates, Inc., to determine their electrical resistivity, pH, chloride and sulfate contents. Results are included in Appendix B, *Laboratory Testing Program*, and are discussed herein.

Soluble sulfate test was 0.024% (by weight) indicating that the site soils are not significantly corrosive to concrete.

Minimum resistivity determined by California Test Method 532 provides a measure of the degree of corrosivity of soils when used in contact with steel. The measured value of the minimum electrical resistivity of the saturated site soils is 875 ohm-cm. This range of electrical resistivity indicates that site soils at saturated conditions are severely corrosive to ferrous metal. A corrosion engineer should be retained to provide mitigation recommendations.

Soluble chloride value was 110 ppm and corresponds to not significantly corrosive in contact with ferrous metals.

Soil pH is a general indicator of corrosivity. For the sample tested, the pH was 7.42. This value of pH is slightly alkaline and is not corrosive to ferrous metal.

The above results indicate that site soils, in general, are not corrosive to concrete. Type I or II Portland cement may be used for the construction of concrete structures and foundations.

The scope of this study is limited to an evaluation of soil corrosivity and its general effects on materials likely to be used for the construction of the subject project. If the engineers require more specific information, design, specifications or design review, a corrosion engineer should be consulted.

7.0 CONCLUSIONS

- Based on the results of our field exploration and laboratory testing, combined with engineering analyses and our experience with similar projects, it is our opinion that the locations for the proposed structures are suitable from a geotechnical standpoint.

- There are no known active faults projecting toward or crossing the project site. The project site is not situated within a currently designated State of California Earthquake Fault Zone. The site is, however, located in a seismically active zone. The nearest known active faults to the site are the Newport-Inglewood (L.A. Basin) fault, located approximately 0.5 miles (0.8 kilometers) to the northeast, and Palos Verdes fault, located approximately 7.7 miles (12.3 Kilometers) to the southwest of the site. Due to the relatively close proximity of the project site to these faults and other adjacent faults, there is a high probability of strong shaking at the site during a strong seismic event. However, the potential for ground surface rupture is considered remote.

- Fill material underlie the project site is in depths ranging from 0.5 to 2.5 feet below the existing ground surface (bgs). Thicker fill material may be anticipated under the

existing building or within the site. The fill materials were likely associated with construction of existing buildings and mainly consist of silty sand and sandy silt. Alluvial deposits underlie the fill material to the maximum explored depth of 81.5 feet below the existing ground surface. The alluvial deposits within the project site generally consist of silty sand, sandy silt, silt, clay, clayey sand, and sand with silt.

- Groundwater was encountered at all borings at depths ranging from ten (10) to fifteen (15) feet below existing ground surface during drilling of exploratory borings. Based on the historical data of the water wells around the site and reports prepared by others, the water level at the area around the site is in range of 6 ½ to 10 feet below the ground surface. Groundwater needs to be considered in the design and construction of the proposed project facilities.

- Due to the shallow ground water, low to medium dense sandy soils, and published documentations and maps, the proposed site is considered susceptible to liquefaction.

- The site soils are not susceptible to hydro-consolidation.

- The corrosion potential of the soil in contact with concrete is low. The corrosion potential of the soil in contact with ferrous metal is severe. Type I or II Portland cement may be used.

- The expansion potential of the site soils is 26 and is considered as low. UBC recommends post-tension slab placed on soils with expansion index greater than 20.

8.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

8.1 General

Site grading will likely contain excavation of about twenty five (25) feet. Prior to the start of grading, utilities should be located in the field and either re-routed or protected. All existing structures within the proposed structures should be demolished and deleterious material debris and surficial soils containing roots and perishable materials should be removed from the site. Any unsuitable materials or undocumented fill uncovered by the stripping operation should be excavated to expose a firm surface or undisturbed natural soil. It is assumed that all existing foundations will be completely removed during excavation.

For the pavement, sidewalk and concrete slab areas, the upper three (3) feet should be removed and replaced with compacted fill. The fill soils should be brought to moisture content within ±2.0 percent of optimum and compacted to at least 90 percent relative compaction as per ASTM Standard D1557-00 test method. The below-grade pipelines can be founded on either firm native soils or compacted fill.

Deeper removal may be needed if loose materials are encountered after proposed depth of excavation is performed. Therefore, some variations in the depth and lateral extent of the over-excavation recommended in this section should be anticipated.

The excavation bottom should be scarified to a depth of at least 12 inches, brought to ±2.0 percent of optimum moisture and compacted to at least 90 percent relative compaction as per ASTM Standard D1557-00 test method. Fill should then be placed on the compacted natural soil in loose lifts of eight (8) inches or less, moisture-conditioned to near-optimum moisture content, and compacted to at least 90 percent relative compaction as per ASTM Standard D1557-00 test method.

Placement of base material or geo-grid material may be required for subgrade soft bottom of excavation. Temporary dewatering will be required for the project.

At a minimum, the upper 12 inches of soil below foundations and slabs should be compacted to at least 95 percent relative compaction as per ASTM Standard D1557-00 test method. No fill or base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant including an engineering geologist. Details of recommended specifications for placement of compacted fill are listed in Section 8.3, *Recommended Specifications for Placement of Compacted Fill.*

Excavated on-site materials to be used for site preparation should meet the following criteria:

- No particles larger than three (3) inches in the largest dimension
- Free of all perishable material
- Have an expansion index of 20 or less

The native soils encountered within the project site, free of debris or organic matter, are suitable as fill after removal of oversize materials and the criteria listed above has been met.

The site soils can be excavated utilizing conventional heavy-duty earth-moving equipment. The excavated site soils free of vegetation may be placed as compacted fill in structural areas after proper processing. Rocks larger than three (3) inches in the largest dimension should not be placed as fill of total soil weight. Rocks larger than one inch should not be placed within the upper 12 inches of subgrade soils. Processing may involve cleaning roots and debris, removal of oversized particles, mixing, and moisture conditioning before placing as compacted fill. The existing near surface on-site soil has moisture content greater than optimize moisture content and special re-working processing is necessary to reduce the moisture within the ±2 of the optimized moisture content.

Backfill behind any structural wall should be compacted using lightweight construction equipment to avoid overstressing the walls. Compaction of backfill adjacent to structural walls can produce excessive lateral pressures. Improper type and location of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of five (5) feet from the walls. Only hand-held compactors should be permitted to perform compaction within the recommended 5-foot zone.

8.2 Compaction

This section contains our recommendations for compaction of fill placed in accordance with the specifications provided in Section 8.3, *Recommended Specifications for Placement of Compacted Fill*, and in Section 8.4, *Subgrade Preparation*.

- All fill, if not noted otherwise, should be compacted to a relative compaction of at least 90 percent as per ASTM Standard D1557-00 test method at moisture content within ±2.0 percent of optimum. Relative compaction is defined as the ratio of the in-place soil dry density to the laboratory maximum dry density as determined by the ASTM Standard D1557-00 test method.

- At least the upper 12 inches of subgrade soils underneath the foundations, slabs, pavements and truck parking areas should be compacted to a minimum of 95 percent relative compaction as per ASTM Standard D1557-00 test method.

- All bases and subbases, if any, for pavement structures should be compacted to a relative compaction of at least 95 percent as per ASTM Standard D1557-00 test method.

8.3 Recommended Specifications for Placement of Compacted Fill

The following specifications are recommended to provide a basis for quality control during the placement of compacted fill.

- Areas to receive compacted fill shall be stripped of all vegetation, organic matter, and debris. Unsuitable topsoil shall be excavated and removed as recommended by the project's geotechnical consultant based on observation during grading. All areas that are to receive compacted fill shall be observed by the project's geotechnical consultant prior to placement of fill. All excavations for subterranean structures shall be observed by an engineering geologist.

- Subsequent to the removal of unsuitable materials, subgrade soil surfaces that will receive compacted fill shall be scarified to a depth of at least 12 inches to provide a bond between existing ground and fill. The scarified soil shall be moisture-conditioned within ±2.0 percent of optimum. Scarified soil shall be compacted to a relative compaction of at

least 90 percent.

- Fill shall be placed in controlled lifts, with lift thickness modified as necessary to achieve adequate compaction. The thickness of the compacted fill layer shall not exceed the maximum allowable thickness of eight (8) inches. Each layer shall be compacted mechanically to the minimum relative compaction as specified in Section 8.2, *Compaction*. The density of the compacted soil shall be determined by ASTM Standard D1556-00 or D2922-96 test methods or equivalent.

- Fill soils shall consist of excavated on-site soils essentially clean of organic and deleterious material, or imported soils approved by the project's geotechnical consultant. All imported soil shall be granular and non-expansive with an Expansion Index (EI) less than 20, as defined by California Building Code (CBC, 2001) standard. Imported structural fill, if there is any, should have a sand equivalent of at least 20. Rocks larger than three (3) inches in the largest dimension shall not be placed as fill. Rocks larger than one inch in the largest dimension should not be placed within the upper 12 inches of the building pad, footing foundation, and pavement subgrade soil.

- The project geotechnical consultant shall evaluate and/or test import materials for conformance with specifications prior to delivery to the site. The material used shall be free of organic matter and other deleterious material.

- The project geotechnical consultant shall observe the placement of compacted fill and conduct in-place field density tests on the compacted fill to check for adequate moisture content and relative compaction as required by the project specifications. Where less than the required relative compaction is indicated, additional compactive efforts shall be applied and the soil moisture-conditioned as necessary until the required relative compaction is attained. The contractor shall provide level testing pads upon which the soils engineer may conduct field density tests. The contractor shall provide safe and timely access for the geotechnical consultant's personnel throughout the grading operation to allow continuous monitoring and testing.

- Wherever, in the opinion of the owner's or the geotechnical consultant's representatives, an unstable condition is being created, either by cutting or filling, the work shall not proceed in that area until an investigation has been made and the grading plan revised, if necessary.

- Fill material shall not be placed, spread, or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain and/or ponding of water, fill operations shall not be resumed until field tests by the geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

8.4 Structural Backfill

Loose soil, formwork, and debris should be removed prior to backfilling. Backfill behind retaining walls and foundations should be placed and compacted in accordance with the recommended specifications as referenced in Section 8.3 *Recommended Specifications for Placement of Compacted Fill.* Heavy construction equipment should be kept away from existing walls to avoid overstressing.

The native soils encountered within the project site, free of debris, large size particles or organic matter is suitable as structural fills.

Imported materials, if required, should meet the following criteria prior to being used as structural backfill (engineered fill):

- No particles larger than three inches in the largest dimension
- Granular, free of organic material, loam, trash, or other deleterious material
- Have an expansion index of 20 or less
- Liquid limit less than 35 and plastic index of 15 or less
- Contain less than 30 percent by weight retained in 3/4-inch sieve
- Contain at least 15 percent by weight passing No. 20 sieve
- Contain less than 40 percent fines (passing #200 sieve)

8.5 Buried Utilities

Buried utility conduits should be bedded with selected material as per the pipe designer's recommendations. Excavated site soils may be used to backfill the trench zone. The trench zone is defined as the zone from one foot above the pipe to the top of the trench, above the pipe zone. Where the conduit underlies concrete slab-on-grade and pavement, the trench backfill should be placed and compacted in accordance with Section 8.3, *Recommended Specifications for Placement of Compacted Fill.* Otherwise, the trench should be backfilled in accordance with the specifications set forth in Section 8.5.2, *Recommended Specifications for Placement of Trench Backfill.*

Open cuts adjacent to existing roadways and/or adjacent structures are not recommended within a 1:1 (horizontal:vertical) plane extending beyond and down from the roadway or structure perimeter.

Soils from the trench excavation should not be stockpiled more than six (6) feet in or within a distance H (ft) from the top of trench edge, where H is the depth of the trench in feet. Soils, if any, should not be stockpiled behind the shoring within a horizontal distance equal to the depth of the trench, unless the shoring has been designed for such loads.

8.5.1 Trench Backfill

Backfill for the pipe trench zone above the pipe bedding should be placed in lifts as recommended in Section 8.5.2, *Recommended Specifications for Placement of Trench Backfill.* Excavated on-site soils may be used to backfill the trench zone above the pipe bedding. Imported soils used as trench zone backfill should meet the following criteria:

- No particles larger than three inches in the largest dimension
- Less than 30 percent by weight retained in 3/4-inch sieve
- Free of all perishable materials
- Plasticity index of 10 or less

8.5.2 Recommended Specifications for Placement of Trench Backfill

The following recommendations are provided as a general guide to be followed during placement and compaction of utility trench backfill:

- Trench excavations to receive backfill shall be free of trash, debris, or other unsatisfactory materials at the time of backfill placement.

- Trench backfill shall be compacted to a minimum relative compaction of 90 percent as per ASTM Standard D1557-00 test method.

- Trench backfill underlying pavement shall be compacted to not less than 95 percent of the maximum dry density to a depth of at least one (1) foot below the pavement base.

- Soils obtained from the excavation that are free of organic and deleterious material may be used as backfill. Imported materials must be granular and approved by the project's geotechnical consultant.

- Rocks generated from the trench excavation not exceeding three (3) inches in the largest dimension may be used as backfill material. However, such material may not be placed within 12 inches of the top of the pipeline. No more than 30 percent of the backfill volume shall be larger than 3/4-inch in the largest dimension diameter, and rocks shall be well mixed with finer soil.

- Bedding material in the pipe zone areas should have a Sand Equivalent (SE) greater than or equal to 30, as determined by the ASTM Standard D2419-91 test method.

- Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to within ±2.0 percent of optimum moisture content, and then placed in horizontal layers. The thickness of uncompacted layers should not exceed

eight (8) inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

- The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

- The field density of the compacted soil shall be measured by the ASTM Standard D1556-00 or the ASTM Standard D2922-96 test methods or equivalent.

- The project's geotechnical consultant should perform observation and field tests during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive efforts shall be made with adjustment of the moisture content as necessary until the specified compaction is obtained.

- Wherever, in the opinion of the owner or its representatives, an unstable condition is being created, either by cutting or by filling, the work shall not proceed in that area until an investigation has been made and the excavation plan revised, if found necessary.

- Fill material shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

8.6 Dewatering

Control of groundwater during construction of the below grade parking structure will be required. The extent of the dewatering will be depending on the depth and area of excavation. In general, dewatering up to 5 feet below the bottom of the proposed over-excavation will be required.

The design, installation and operation of the dewatering system should be performed by the specialty contractor. It is recommended that a dewatering specialty contractor be retained to evaluate the approved dewatering system. If a permanent dewatering system is constructed in the design of the structure, the permanent system and the temporary construction related system can be coordinated and portions of the two systems combined if practical.

The design, installation, and operation of the temporary dewatering system should satisfy the following criteria:

- The dewatering system should be installed and operated for sufficient time period to the excavation reaching the level of the groundwater. The system can be designed to

either completely dewater the soil prior to excavation or to dewater the soil prior to excavation and pump the water out of the excavation with sumps located within the excavation.

- Design capacity of the dewatering system should be based upon pump tests performed by the designer.
- The system should maintain the groundwater levels low enough to prevent boiling of soil into the excavation.
- The contractor should be responsible for disposing the dewatering. He should be responsible for satisfying applicable codes and ordinances, including obtaining required discharge permits.
- The system should be operated continuously. Emergency power and backup pumps should be provided to ensure continual excavation dewatering
- The contractor should be responsible for monitoring the ground surface settlement resulting from the lowering of the groundwater within the soil strata. It is anticipated that maximum settlement will occur at the dewatering system and decrease gradually as the distance away from the excavation increases.

9.0 DESIGN RECOMMENDATIONS

The various design recommendations provided in this section are based on the assumption that in preparing the site, the above earthwork and grading recommendations will be implemented.

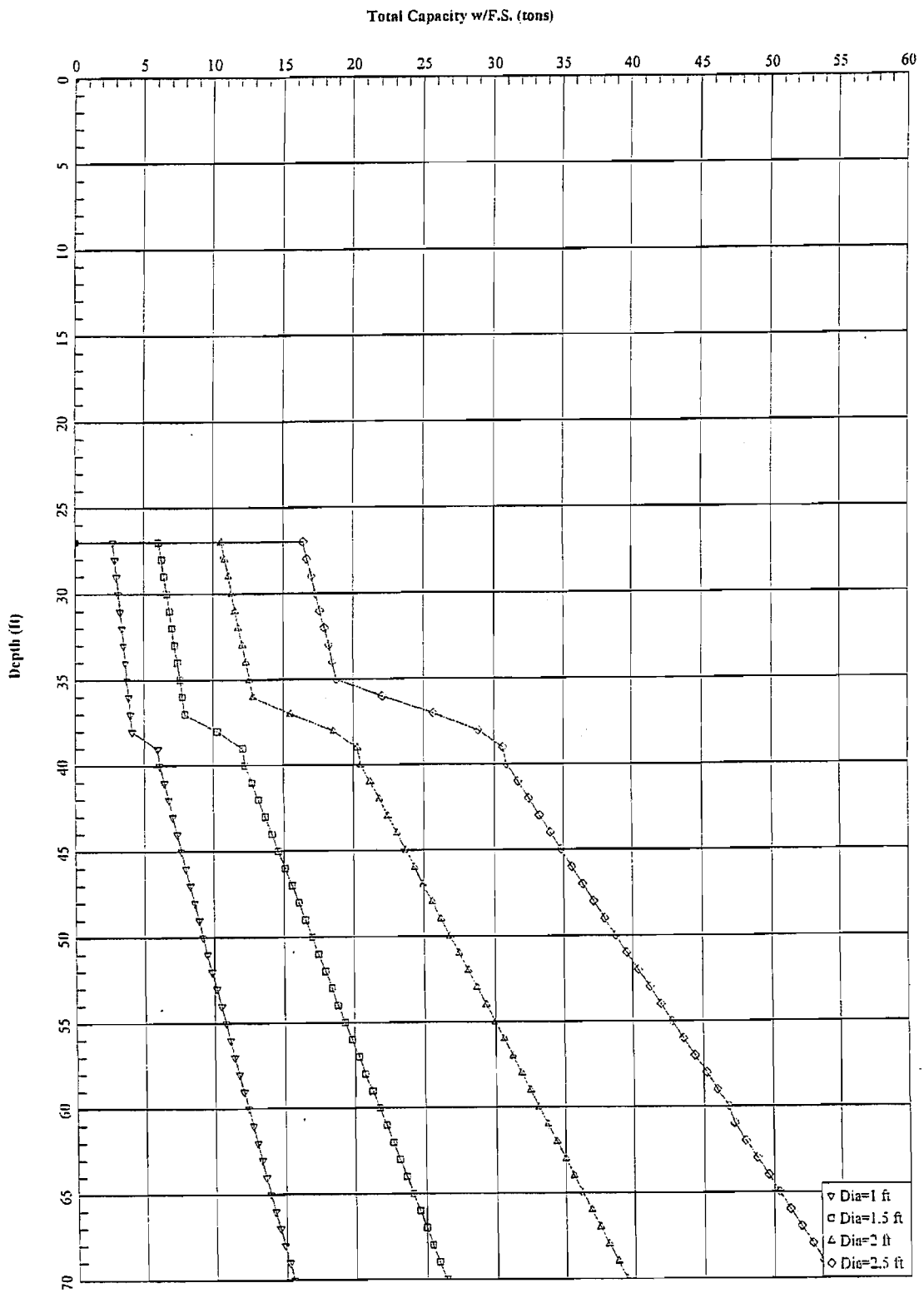
9.1 General Evaluation

Based on our field exploration, laboratory testing and analyses of subsurface conditions at the site, remedial grading or liquefaction potential mitigation measures will be required to prepare the site for support of the various structures.

9.2 Foundation Type and Bearing Pressures

For the moderately to highly loaded structures, the structures should be founded on either cast-in-place drilled or driven piles or shallow foundations on reinforced soil. Without soil improvement for liquefaction potential, the proposed structures will be supported on either drilled cast-in-place or driven piles.

For the lightly loaded small structures, the use of post-tensioned concrete slabs combined with grade beam or a mat foundation will reduce the potential for liquefaction settlement. Shallow foundation or mat foundation may be used with proper liquefaction potential mitigation measure.



Vertical Load Capacity (Driven Piles)

MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Figure No.
4a

Converse Consultants

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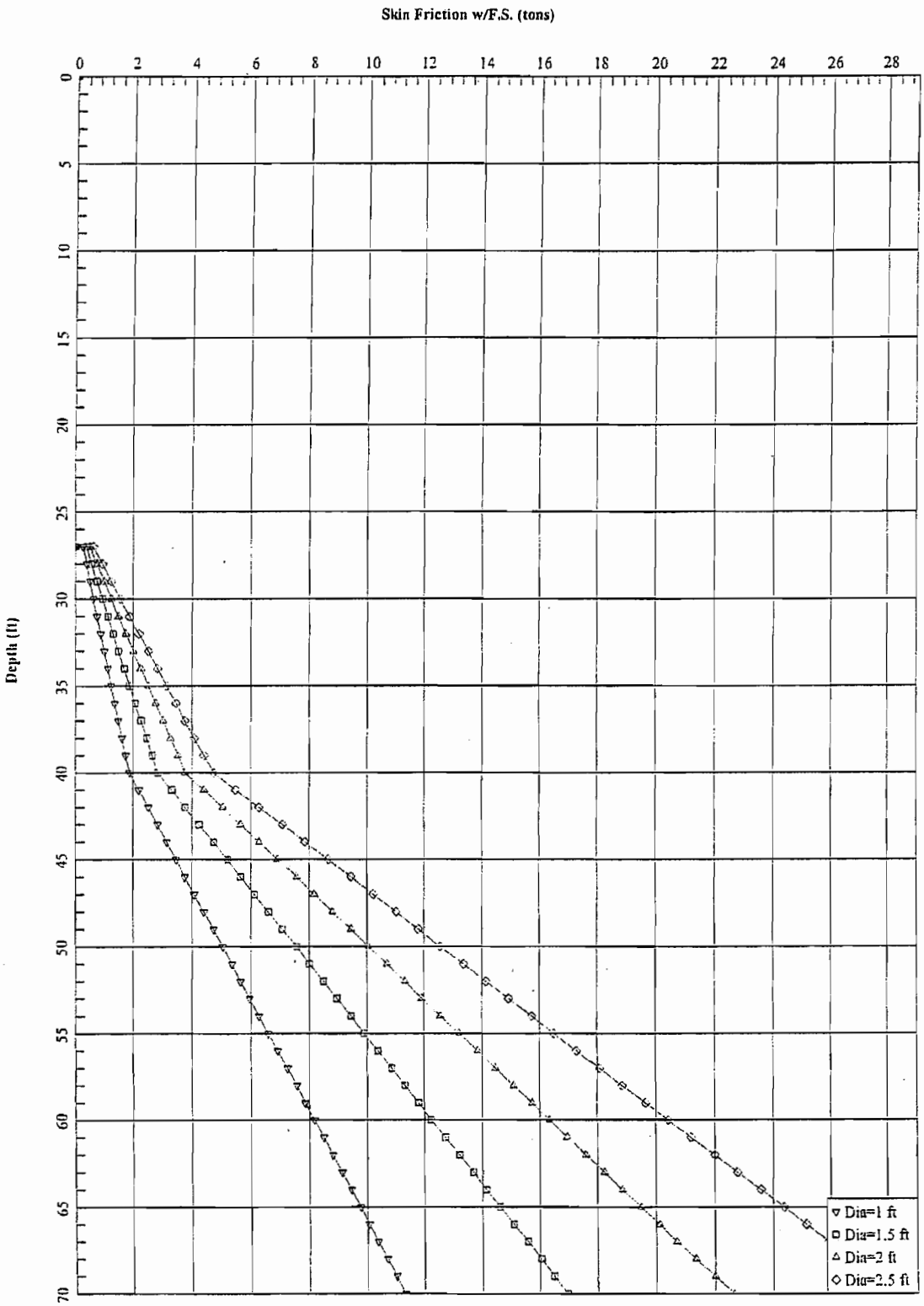

Vertical Load Capacity (Cast-in-place Piles)

MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Figure No.
4b

Converse Consultants

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Foundations should be supported on firm natural soils or on compacted fill. Design recommendations for various types of foundations is presented below.

9.2.1 Piles Foundations

Without soil improvement mitigation measures for liquefaction potential, the proposed structures will be supported on piles foundations. Pile design criteria are presented below.

Vertical Load Capacity

Downward capacity of driven and cast-in-drilled-hole piles may be obtained using the Figure No. 4a, *Friction Load Capacity (Driven Piles),* and Figure No. 4b; *Friction Pile Capacity (Cast-in-place Piles), respectively.* Pile capacities for the skin friction value are based upon geotechnical considerations only and actual pile capacities may be limited by structural considerations such as the strength and rigidity of the reinforced concrete pile as a structural element. In order to prevent liquefaction settlement, the friction for the soil within liquefied areas has been neglected. The pile length given in the curve is for the pile length from the bottom of the pile cap.

The minimum embedment of piles into native non-liquefied soil below the 40 feet deep should be 30 feet. In order to eliminate reductions in capacities due to group efficiency and problems in construction, the minimum pile spacing should be 3 diameters on center.

Short-term uplift capacities may be assumed to be equal to half the downward friction capacities.

Settlement of single piles designed and constructed in accordance with the recommendations presented herein is estimated to be on the order of 1 inch. Actual settlement would depend on the applied loads. Pile group settlement would depend on pile spacing, diameter, number of piles and/or the minimum dimensions of the pile group cap.

Vertical loads indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by 33 percent for a short duration of loading which will include the effect of wind or seismic forces. These recommended friction load capacity are obtained by applying a factor of safety of at least 2 against estimated ultimate pile capacities in skin friction.

Lateral Capacity

Analyses were performed to determine allowable lateral capacities for various diameter piles. The recommended allowable lateral pile capacities and related design parameters as presented in the Table No. 3, *Recommended Pile Design Parameters for Lateral Loads.*

Table No. 3, Recommended Pile Design Parameters for Lateral Loads

FIXED HEAD	Diameter			
	12-inch	18-inch	24-inch	30-inch
Allowable Lateral Load Capacity, P (kips)	14	16	18	24
Maximum Negative Moment (kip-ft)	6.31p	5.99p	5.99p	5.56p
Maximum Positive Moment (kip-ft)	1.89p	1.68p	1.68p	1.39p
Depth to Maximum Negative Moment (ft)	0.0	0.0	0.0	0.0
Depth to Maximum Positive Moment (ft)	15	15	15	16
First Point of Zero Lateral Displacement (ft)	20	22	23	25
Depths to Zero Moments (ft)	8.0 & 30.0	8.0 & 30.0	8.0 & 30.0	8.0 & 30.0

Recommended allowable lateral load capacities presented in Table No. 3, *Recommended Pile Design Parameters for Lateral Loads*, are based on 0.25-inch maximum horizontal deflection and included effects of pile group action and short-term loading such as wind or seismic forces. Lateral pile capacities are based on a 28-day concrete compressive strength of 5,000 pound-per-square-inch (psi)

Pile Construction

Pile drilling and concrete placement should be performed in accordance with the recommendations presented herein and in the Appendix D, *Guide Specifications for Drilled Pile Installation* and the Standards and Specifications of ADSC: An International Association of Foundation Drilling Contractors.

It should be noted that the soils encountered during this investigation are relatively sandy and groundwater was encountered at depths varying from 10 to 15 feet below the ground surface. As a result, caving of the sidewalls can be expected during the drilling and construction of the cast-in-drilled-hole piles.

Drilling of pile shafts should be observed by the Geotechnical Engineer to confirm that piles are extended to the proper depth and that material encountered is similar to that encountered in the borings drilled for this investigation. Pile lengths should be tabulated in the foundation plans based upon the embedment below the bottom of the pile cap or other point of reference that can be established in the field during construction.

9.2.2 Shallow Foundations

If the site soil will be improved in accordance with the recommendations presented in Section 5.4, *Liquefaction Evaluation and Mitigation Recommendations*, the structures may be supported by shallow foundations. Design parameters for shallow foundations are presented below.

Spread Footing

For shallow spread footings with grade beams founded on compacted structural backfill as indicated in Section 8.4, *Structural Backfill*, footings should be at least 24 inches wide and embedded at least 24 inches below the lowest adjacent final soil grade. The footing should be founded on at least 36 inches or half of the footing width of compacted soils, whichever is greater. An allowable net bearing capacity of 2,500 pounds per square foot (psf), plus 100 psf for each foot of width or depth may be used.

The allowable net bearing capacity is defined as the maximum allowable net bearing pressure on the ground. It is obtained by dividing the net ultimate bearing capacity by a safety factor. The ultimate bearing capacity is the bearing stress at which ground fails by shear or experiences a limiting amount of settlement at the foundation. The net ultimate bearing capacity is obtained by subtracting the total overburden pressure on a horizontal plane at the foundation level from the ultimate bearing capacity. The maximum allowable bearing capacity (net bearing capacity plus overburden pressure) should be limited to 3,500 psf.

In addition, overburden pressure of the soil can be added to the above net bearing capacity. Overburden pressure can be calculated by using a total unit weight of 110 pcf times the embedded depth above the ground water table. For example: a 2,500 psf overburden pressure can be added to the bearing capacity if the footing is founded at a depth of 20 feet below the lowest adjacent grade with the embedded depth above the ground water table. Also, 60 pcf should be used for the unit weight in calculating the overburden pressure below the ground water table. The maximum allowable bearing capacity (net bearing capacity + overburden pressure) should be limited to 6,000 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.

The groundwater level may raise to depth above the proposed bottom of foundations for some structures. Therefore, the uplift pressure equal to *62.4h* should be considered in design; where *h* is the height of water table above the foundation bottom.

Mat Foundations

For mat foundations placed on compacted structural backfill or firm native soils as indicated in Section 8.5, *Structural Backfill*, should be embedded at least four (4) feet below the lowest adjacent final soil grade. Greater embedment may be needed in order to resist lateral loads due to the wind and/or seismic forces. The following equation may be used to calculate k for use in mat foundation design:

$k= 200\ (B+1)/2B]^2$, k value including overburden pressure
k= modulus of subgrade reaction, kips per cubic foot
B= foundation width, feet

In addition, overburden pressure of the soil can be added to the above net bearing capacity. Overburden pressure can be calculated by using a total unit weight of 110 pcf times the embedded depth above the ground water table. For example: a 2,500 psf overburden pressure can be added to the bearing capacity if the footing is founded at a depth of 20 feet below the lowest adjacent grade with the embedded depth above the ground water table. Also, 60 pcf should be used for the unit weight in calculating the overburden pressure below the ground water table. The maximum allowable bearing capacity (net bearing capacity + overburden pressure) should be limited to 6,000 psf.

The groundwater level may raise to depth above the proposed bottom of foundations for some structures. Therefore, the uplift pressure equal to *62.4h* should be considered in design; where *h* is the height of water table above the foundation bottom.

9.3 Slabs-On-Grade

Design of the slab-on-grade supported by properly compacted soils may be based on a modulus of subgrade reaction of 200 pounds per square inch per inch (psi/in).

Structural design elements such as thickness, reinforcement, joint spacing, etc., for the slab-on-grade should be selected based on the analyses performed by the project structural engineer considering anticipated loading conditions and the modulus of subgrade reaction of the supporting soils as presented above.

Minimum reinforcement of 6" x 6"-#10/ #10, or equivalent, properly centered in the middle of the slab, is recommended. As indicated above, the structural design may require greater thickness and/or reinforcement.

Subgrade soils must be firm and non-yielding prior to placement of concrete.

In hot weather, the contractor should take appropriate curing precautions after placement of concrete to minimize cracking of the slabs. The potential for slab cracking may be lessened by the addition of fiber mesh to the concrete and/or control of water/cement ratio.

Joints for concrete slabs-on-grade must be carefully designed. Joint spacing is dependent upon slab thickness and concrete properties and should be selected by the structural engineer.

Concrete should be cured by protecting it against loss of moisture and rapid temperature change for at least seven days after placement. Moist curing, waterproof paper, white polyethylene sheeting, white liquid membrane compound, or a combination thereof, may be used after finishing operations have been completed. The edges of concrete slabs exposed after removal of forms should be immediately protected to provide continuous curing.

In areas where a moisture-sensitive floor covering (such as vinyl tile or carpet) is used, slabs should be protected by at least a six-mil-thick polyethylene vapor barrier between the slab and compacted subgrade. Where a vapor barrier is used, it should be protected with two inches of sand placed above the barrier, to reduce the potential for punctures and to aid concrete curing. Polyethylene sheets should be overlapped a minimum of six inches, and should be taped or otherwise sealed.

The above recommendations are based on the results of tests performed on representative site soils. If soils other than those presently encountered within the project site are placed as structural fill within the building pads, the modulus of subgrade reaction should be reevaluated. The final slab design should be based on this value of the modulus of subgrade reaction.

9.4 Settlement

The settlement of structures supported on spread footing and mat foundations founded on compacted fill or firm native soils will depend on the actual foundation dimensions and the imposed vertical load. Based on the maximum allowable bearing pressures, settlement of less than one inch should be expected. Differential settlement within the foundation systems would depend on the distance and the difference in the applied loads. For similarly loaded points, the differential settlement may be expected to be less than one half the total settlement. The total settlement for piles estimated to be less than one inch.

9.5 Site Drainage

Adequate positive drainage should be provided away from the structures to prevent ponding and to reduce percolation of water into structural backfill. A desirable slope for surface drainage is two (2) to four (4) percent in landscaped areas and one (1) to two (2)

percent in paved areas. Planters and landscaped areas adjacent to the building perimeter should be designed to minimize water infiltration into the subgrade soils.

9.6 Lateral Earth Pressures and Resistance to Lateral Loads

The following subsections outline subterranean or retaining walls design. Lateral earth pressure and resistance to lateral loads are estimated by using on-site native soils or imported material with a minimum total unit weight of 110 pounds per cubic foot (pcf), cohesion of 50 pounds per square foot (psf), and an internal friction angle of 22 degrees.

9.6.1 Active Earth Pressure

Permanent subterranean or retaining walls system should be designed to resist lateral earth pressures as illustrated in Figure No. 5, *Recommended Lateral Earth Pressure on Permanent Retaining Wall.* These pressures assume a level ground surface behind the wall for a distance greater than the wall height.

9.6.2 Passive Earth Pressure

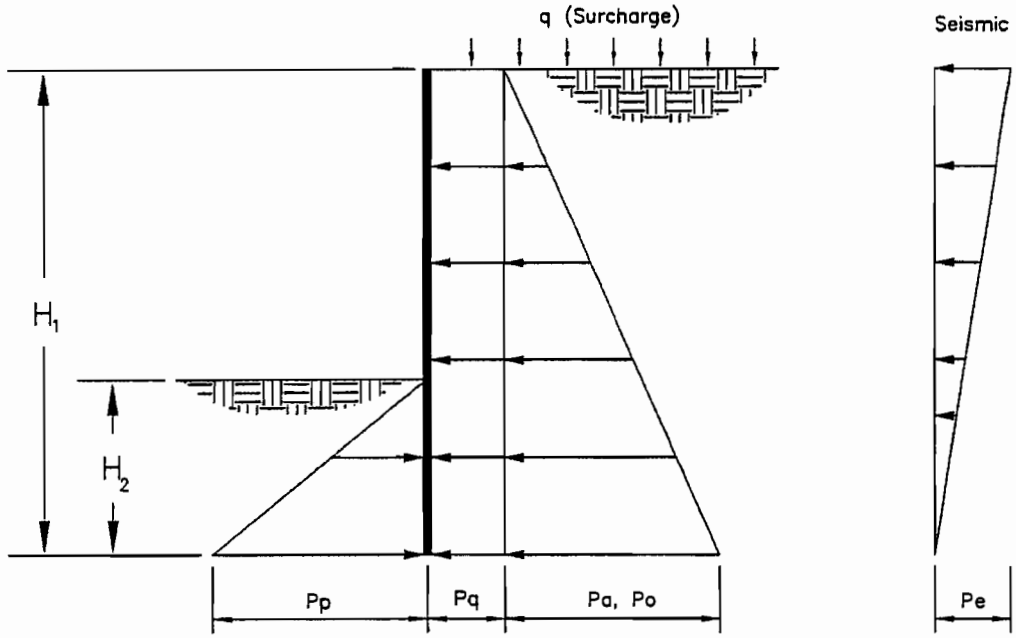
Lateral loads can be resisted by an allowable passive earth pressure of 250 pounds per foot of depth for dense natural soil or compacted fill applied against below-grade wall elements. The allowable passive earth pressure indicated above is obtained by applying a factor of safety of 1.5 to the ultimate passive earth pressure. Due to the low overburden stress of the soil at shallow depth, the upper one foot of passive resistance should be neglected unless the soil is confined by pavement or slab. The maximum passive pressure should not exceed 2,500 psf. In addition, a friction coefficient of 0.35 between the concrete and dense natural soil or compacted fill can be used in combination with passive earth pressures to resist lateral loads. The coefficient of friction should be applied to net normal dead loads only.

9.7 Groundwater Control

The two-story subterranean parking structures that extend below the groundwater level should be designed to resist hydrostatic pressure or be provided with a permanent dewatering system.

In the absence of a permanent dewatering system the exterior walls and floor slabs should be designed to resist the lateral and uplift pressures from the groundwater. Based upon review of the groundwater information reported in the referenced geotechnical reports, the historical groundwater information, and groundwater data obtained as part of this investigation, we recommend that the 6 feet below the ground surface to be used as the design depth to groundwater in the calculation of hydrostatic pressures.

PERMANENT RETAINING WALLS



$P = Pq + Pa$	(350 psf minimum if less than 350 psf)
$= 0.5q + 40\ H_1$	– active earth pressure (Cantilever walls)
$= 0.5q + 60\ H_1$	– active earth pressure (Restrained walls)
$Pp = 250\ H_2 \leq 3500$ psf	– passive earth pressure
$Pe = 15\ H_1$	– seismic earth pressure
$\mu = 0.35$	– allowable friction coefficient

Notes:

1. All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf).
2. Pp, Pa, and Po are the passive, active, and at–rest earth pressures, respectively; Pe is the incremental seismic earth pressure; Pq is the incremental surcharge earth pressure; and μ is the allowable friction coefficient, applied to dead normal loads acting on non–pile supported elements.
3. For restrained walls (not free to rotate), use at–rest (Po) earth pressure; increase Pe by 30 percent.
4. Base friction coefficient (μ) and Pp include a safety factor of 1.5.
5. Neglect the upper 1 foot for passive pressure unless the surface is confined by a pavement or slab.
6. Surcharge load only applys the upper 10 feet.
7. Pe calculated by using a maximum probable ground acceleration (10% in 50 years)
8. Drainage system should be provided for the retaining wall.
9. For traffic surcharge, assume a 100–psf uniform pressure along the top 10 feet.
10. Earth pressure assume no hydrostatic pressure. If hydrostatic pressures are allowed to build up, the incremental earth pressure below the groundwater level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.

RECOMMENDED LATERAL EARTH PRESSURE ON PERMANENT RETAINING WALL

MIXED–USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04–31–118–01

Figure No.
5

Converse Consultants

H:\DATA\CADWORK\04–31–118–01e.dwg

The alternative to design of the exterior walls and on-grade floor for the full hydrostatic pressure based upon current depth to groundwater, a permanent dewatering system can be included in the structural design to either eliminate groundwater contact with the structure or to partially lower the groundwater in order to reduce hydrostatic pressures on the walls and floor slabs in contact with the groundwater. Design of a dewatering system to lower the groundwater completely below the bottom of the parking structure or to partially lower the groundwater to reduce hydrostatic pressures, should be performed by dewatering contractor.

9.8 Construction Considerations

Temporary slopes may be used during excavations where not constrained by adjacent utilities and structures. Where space is limited due to adjacent facilities and buried utilities to be salvaged and protected, shoring may be required.

9.8.1 Temporary Excavation

Stability of temporary excavations is a function of several factors, including the time period that the excavation is left open, moisture condition, soil type and consistency, and the contractor's operations. As a general guideline, the sides of temporary excavations deeper than five (5) feet should be supported or sloped in accordance with Table No. 4, *Slope - Temporary Excavation.*

Table No. 4, Slope - Temporary Excavation

Maximum Depth of Excavation (feet)	Maximum Slope Ratio (horizontal:vertical)
0 – 5	0.5:1
5 – 10	1:1
10 – 15	1.5:1
15 – 40	2:1

For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor as necessary to protect the workers in the excavation. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Equipment that may result in surcharge loads should not be placed within five (5) feet of the trench edge. The above maximum slopes are based on a maximum height of stockpiled soils of six (6) feet adjacent

to the trench. Cuts that are proposed within five (5) feet of light standards, other utilities or pavement should be provided with temporary shoring.

The proposed excavation should not cause loss of bearing and/or lateral supports of the existing structures.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1993, and the Construction Safety Act should be met.

Personnel from a qualified geotechnical firm should inspect the soils exposed in the cut slopes during excavation so that modification of slopes can be made if variations in the soil conditions occur.

Permanent cut and fill slopes should be planned no steeper than 2:1 (horizontal:vertical).

9.8.2 Temporary Shoring

Earth materials encountered in our borings generally consisted of sands with varying amounts of silt. Due to the sandy nature of the subsurface soils, caving should be expected during installation of soldier piles and tiebacks.

Cantilevered Shoring

Temporary shoring will be required to support of construction excavations. A soldier-pile shoring system may be used to maintain temporary support of vertical wall excavations. Due to the sandy nature of the soils encountered during this investigation, caving during the drilling of soldier-pile borings should be expected. A soldier-pile system will also most likely require continuous lagging to control caving and sloughing in the excavation between soldier piles. Shoring design must consider the support of adjacent underground utilities and/or structures, and should consider the effects of shoring deflection on supported improvements.

Temporary cantilever shoring should be designed to resist a lateral earth pressure equivalent to a fluid density of 40 pounds-per-cubic-foot (pcf). This equivalent fluid pressure is valid only for shoring retaining level ground. Temporary cantilevered shoring retaining slope ground with an inclination of 2 to 1 (horizontal to vertical) or flatter, should be designed to resist a lateral earth pressure equivalent to a fluid density of 55 pounds-per-cubic-foot (pcf). These values for active earth pressure are considered actual earth pressure with no increase for factors of safety. The shoring design engineer in designing the shoring system should add an appropriate factor of safety.

Surcharge pressures should be added to the above earth pressures for surcharges within a distance from the top of the shoring less than or equal to the shoring height. A surcharge

coefficient of 30 percent of any uniform vertical surcharge should be added as a horizontal shoring pressure for cantilever shoring.

Lateral resistance for soldier piles may be assumed to be provided by passive pressure below the bottom of excavations. The allowable passive pressure for soldier piles spaced at least 3 diameters on center may be taken as 600 psf on the pile per foot of depth, measured below the bottom of excavation. Closer spaced soldier piles should be designed using a passive resistance of 300 psf. The allowable maximum passive resistance should not exceed 9,000 psf. It should be noted that the above values for passive earth pressure given for the design of soldier piles have been adjusted for potential arching between piles and no additional increases for arching should be assumed.

Caving soils should be anticipated between the piles. To limit local sloughing, caving soils can be supported by continuous lagging or guniting. All lumber to be left in the ground should be treated in accordance with Section 204-2 of the "Standard Specifications for Public Works Construction" (2000 Edition, Green Book).

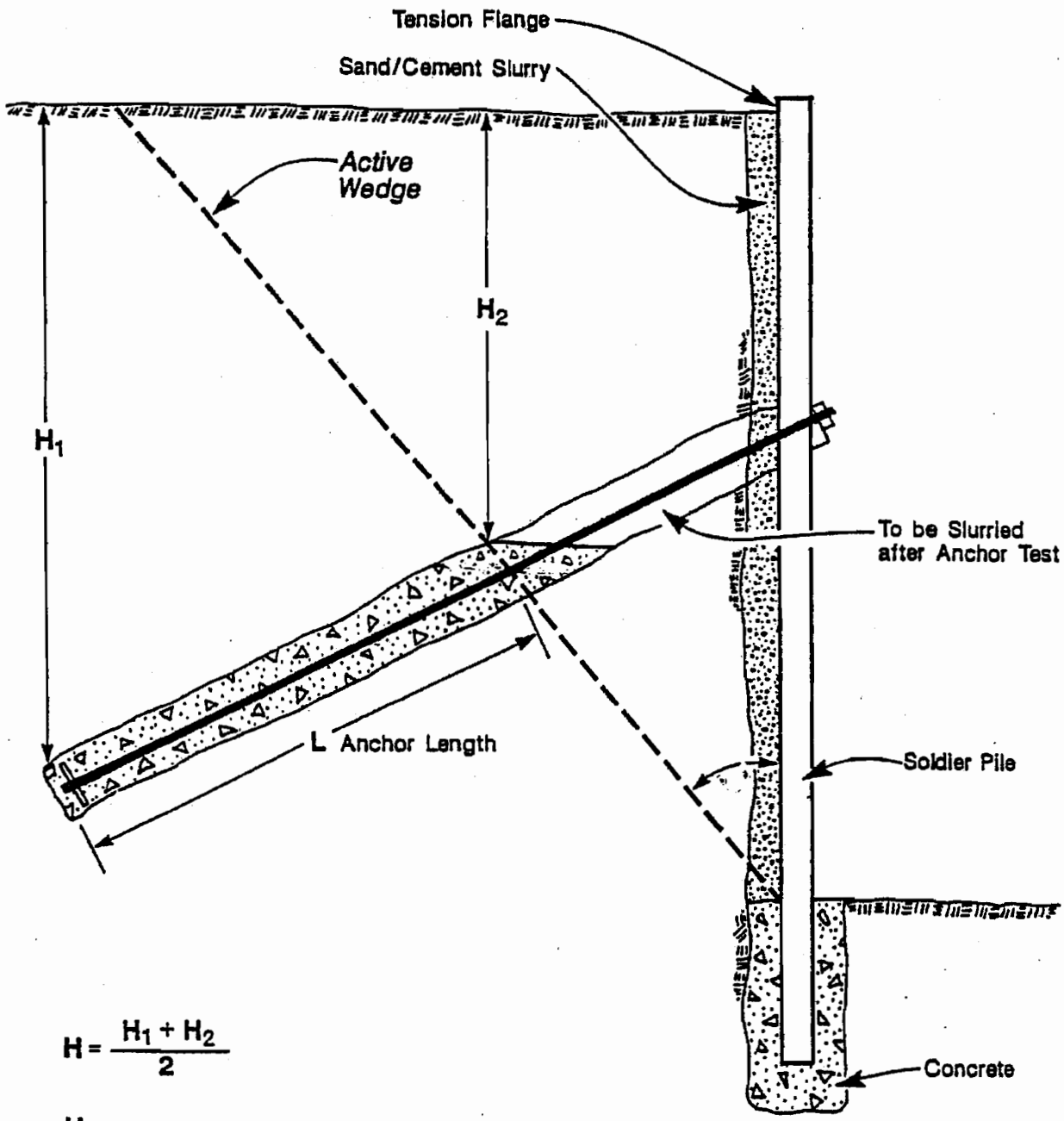
It is recommended that Converse review plans and specifications for proposed shoring and that a Converse representative observe the installation of shoring. A licensed surveyor should be retained to establish monuments on shoring and the surrounding ground prior to excavation. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project Structural (shoring) Engineer and Converse for review and evaluation. Adjacent buildings should be photo-documented prior to construction.

Braced (Tied-Back) Shoring

A tied-back soldier-pile shoring system may be used to maintain temporary support of deep vertical wall excavations. Braced or tied-back shoring, retaining a level ground surface, should be designed for a uniform pressure of 26H psf, where H is the height of the retained cut in feet. Surcharge pressures should be added to this earth pressure for surcharges within a distance from the top of the shoring less than or equal to the shoring height. A surcharge coefficient of 40 percent of any uniform vertical surcharge should be added as a horizontal shoring pressure for braced shoring. Braced or tied-back shoring, retaining a sloping ground surface with a inclination of 2 to 1 (horizontal to vertical) or flatter, should be designed for a uniform pressure of 36H psf, where H is the height of the retained cut at the back of the shoring in feet. These values for earth pressure are considered actual earth pressure with no increase for factors of safety. The shoring design engineer in designing the shoring system should add an appropriate factor of safety.

Tie-backs

Tied-back shoring will consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied back with drilled in friction anchors, as depicted schematically on Figure



$$H = \frac{H_1 + H_2}{2}$$

H = Average Depth to Tie-Back

SCHEMATIC TIE-BACK DESIGN

Converse Consultants	MIXED-USED COMMUNITY LONG BEACH, CALIFORNIA FOR: LENNAR COMMUNITIES	Project No. 04-31-118-01	Figure No. 6

No. 6, *Schematic Tie-back Design.*

For design of tied-back shoring, it should be assumed that the potential wedge of failure is determined by a plane at 30 degrees from the vertical, through the bottom of the excavation. Tieback anchors may be installed at angles of 15 to 40 degrees below a horizontal plane. Tieback installation and testing guidelines and procedures are presented in Appendix E, *Guide Specifications for Installation and Acceptance of Tieback Anchors.* Soil friction values, for estimating the allowable capacity of drilled friction anchors with grout or concrete applied under low pressure, may be computed using the following equation:

$q = 40H$; $q \leq 600$ *pounds-per-square-foot (psf)*
where:
H = average depth of anchor below ground surface, shown on Figure No. 6,
q = anchor surface area resistance, in psf (excluding tip),

Post-grouted tiebacks with grout applied at high pressure should be designed in accordance with the criteria specified in the Caltrans "Trenching and Shoring Manual".

Only the frictional resistance developed beyond the assumed failure plane should be included in the tieback design for resisting lateral loads.

9.9 Asphalt Concrete Pavement

Preliminary structural pavement sections were calculated based on an R-value of 41. To determine the final structural pavement sections, R-value tests should be performed from soils collected from near finish grade within the proposed streets after grading. An analysis was performed in accordance with the method suggested in the Caltrans Highway Design Manual, to determine required flexible pavement structural sections.

Asphalt concrete pavement sections corresponding to Traffic Indices (TIs) ranging from 5.0 to 9.0 and an R-value of 41 are presented for preliminary design. Results of these analyses are presented in Table No. 5 *Recommended Pavement Sections.* Analysis was based on Caltrans' design procedure for flexible pavement structural sections without the recommended safety factor of 0.20 feet, when evaluating the required Gravel Equivalent (GE) of Asphalt Concrete (AC).

Table No. 5, Recommended Pavement Sections

R-value	Traffic Index (T.I.)	Pavement Section	
		Asphalt Concrete (inches)	Aggregate Base (inches)
41	5.5	3.0"	5.5"
	6.5	3.5"	7.0"
	7.5	4.5"	8.0"
	9.0	5.5"	9.5"

Prior to placement of aggregate base, at least the upper 12 inches of subgrade soils should be scarified, moisture-conditioned, if necessary, and recompacted to at least 95 percent relative compaction as defined by ASTM Standard D1557-00 test method.

Base materials should conform to Section 200-2.2, "Crushed Aggregate Base," of the current Standard Specifications for Public Works Construction (SSPWC, 2003 edition) and should be placed in accordance with Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC and should be placed in accordance with Section 302.5 of the SSPWC.

10.0 PLAN REVIEW

This report has been prepared to provide preliminary geotechnical information pertaining to design and construction of proposed developments. It is recommended that the geotechnical consultant be provided the opportunity to review the grading plan, the final design drawings, and specifications to determine if the recommendations of this report have been properly implemented.

11.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Preliminary design recommendations given in this report are based on the assumption that the proposed structures will be placed on native suitable soil or properly compacted fill. All excavations and pile installations should be observed by a qualified geotechnical consultant prior to placement of the structures to verify that the foundation is founded on satisfactory materials and that excavations are free of loose and disturbed soils. All backfill should be placed and compacted under observation and testing of a geotechnical consultant.

12.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice within our profession. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field and laboratory investigations, combined with an interpolation of soil conditions between and beyond boring locations. If conditions encountered during grading appear to be different from those shown by the borings, this office should be notified.

Design recommendations given in this report are based on the assumption that the earthwork and construction recommendations contained in this report are implemented. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

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APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance of the property and a subsurface exploration program consisting of drilling five (5) borings (BH-1 through BH-5), and four (4) Cone Penetration Testing (CPT-1 through CPT-4). During the site reconnaissance, the surface conditions were noted and the locations of the borings were determined. The exploratory borings and CPTs were located using existing topography and boundary features as a guide. Description of the two methods of subsurface exploration is presented below.

Boring

The borings were drilled with a truck mounted, 8-inch diameter hollow stem auger equipped for soil sampling. Soils were continuously logged and classified in the field by visual examination in accordance with the Unified Soil Classification System. The field descriptions have been modified where appropriate to reflect laboratory test results.

Relatively undisturbed ring and bulk samples of the subsurface soils were obtained at frequent intervals in the borings. The undisturbed samples were obtained with a California Modified Sampler (2.4-inch inside diameter and 3-inch outside diameter) lined with thin sample rings. The soil was retained in brass rings (2.4 inches in diameter and one inch in height). The lower portion of the sample was retained and carefully sealed in waterproof plastic containers for shipment to our laboratory. Bulk soil samples were collected in plastic bags and brought to our laboratory.

Standard Penetration Tests (SPTs) were performed at selected depths in all five (5) borings using a standard (1.4-inch inside diameter and 2.0-inch outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for each 6 inches of a total of one and half foot of sampler penetration is shown on the Logs of Borings in the "blows" column. The standard penetration tests were performed in accordance with the ASTM Standard D1586-84 test method.

A key to soil symbols and terminology used in the boring logs is included as Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For Logs of Borings, see Drawings No. A-2 through A-6, *Logs of Borings.*

CPT

The purpose of the CPTs was to obtain a continuous profile of the subsurface conditions and use them for evaluation of the liquefaction potential of the proposed site. The tests were performed by sub-consultant, Furgo Geosciences, Inc., Santa Fe Springs, California, 90670. Results of Cone Penetration Testing (CPT) are included at the end of this appendix.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

- STANDARD PENETRATION TEST — Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method
- DRIVE SAMPLE — 2.42" I.D. sampler.
- DRIVE SAMPLE — No recovery
- BULK SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION

Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h

STRENGTH

Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

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Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-1

Log of Boring No. BH-1

Dates Drilled: 3/2/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 15

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		4" ASPHALT CONCRETE						ma,se
		FILL (Af): **SANDY SILT (ML):** fine-grained sand, trace clay, brown.						
		ALLUVIUM (Qal): **SILTY SAND (SM):** fine-grained, gray.			3,4,5	24	98	
5					1,5,11	27	97	ds
10		**SANDY SILT (ML):** fine-grained sand, slight odor, gray to dark gray.			3,7,9	21	98	
15					2,3,3	36	82	wa (56%)
20		**SILTY SAND (SM):** fine-grained, trace clay, gray.			2.3.5			wa (45%)
25					3,5,13	32	89	
30		**SAND WITH SILT (SP-SM):** fine-grained, gray.			2,6,11			wa (11%)

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Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-2a

Log of Boring No. BH-1

Dates Drilled: 3/2/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 15

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND WITH SILT (SP-SM): fine-grained, gray.	■		5,8,11	21	107	
40		**SILTY SAND (SM):** fine to medium-grained, yellowish gray.	☒		4,4,3			wa (19%)
45		-fine grained	■		11,20,34	24	102	
50		**SAND WITH SILT (SP-SM):** fine-grained, gray.	☒		4,6,6			wa (6%)
55			■		5,14,21	24	99	
60		**SAND (SP):** fine to medium-grained, gray.	☒		6,12,16			wa (4%)
		End of boring at 61.5 feet. Groundwater encountered at 15' bgs. Boring backfilled with soil cuttings and surface patched with asphalt on 3-2-04.						

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Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-2b

Log of Boring No. BH-2

Dates Drilled: 3/2/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 15

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		2" ASPHALT CONCRETE						max
		FILL (Af): **SILTY SAND (SM):** fine-grained, greenish brown.						
		ALLUVIUM (Qal): **SILTY SAND (SM):** fine-grained, light gray.	■		4,5,11	16	97	
5			■		5,8,14	26	97	
10		-gray	■		5,6,5	29	92	
15		**CLAYEY SAND (SC):** fine-grained, gray.	■		2,3,2	28	82	
20		**SANDY SILT (ML):** fine-grained sand, gray.	☒		3,2,2			wa (53%)
25			■		5,8,11	29	96	
30		**SILT (ML):** trace of fine-grained sand, gray.	☒		3,3,6			wa (86%)

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Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-3a

Log of Boring No. BH-2

Dates Drilled: 3/2/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 15

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SILT (ML): trace fine-grained sand, gray.	■		4,7,8	31	97	
40		-yellowish brown	☒		2,2,3			wa (92%)
45		**SILTY SAND (SM):** fine-grained, gray.	■		6,12,19	26	95	ds
50		**SAND (SP):** fine to medium-grained, gray.	☒		7,19,15			wa (5%)
55		-trace fine gravel	■		12,25/6"	20	97	
60		**SAND WITH SILT (SP-SM):** fine-grained, gray.	☒		3,8,16			wa (12%)

End of boring at 61.5 feet.
Groundwater encountered at 15' bgs.
Boring backfilled with soil cuttings and surface patched with asphalt on 3-2-04.

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-3b

Log of Boring No. BH-3

Dates Drilled: 3/3/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 15

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" ASPHALT CONCRETE OVER 2" BASE						
		ALLUVIUM (Qal):						
		SILTY SAND (SM): fine-grained, gray brown.						
			■		2,4,6	21	103	
5		-increasing silt content	■		4,5,5	25	97	
10		-trace clay	■		1,2,3	39	82	c
15		**SILT (ML):** trace fine-grained sand, gray.	■		2,2,3	49	74	
20		-little clay	☒		1,2,1			wa (90%)
25		**SILTY SAND (SM):** fine-grained, gray.	■		3,6,9	33	90	ds
30		-trace shells	☒		3,6,8			wa (25%)

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-4a

Log of Boring No. BH-3

Dates Drilled: 3/3/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 15

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SILTY SAND (SM): fine-grained, gray.	■		8,14,20	24	98	
40		**SILT (ML):** some, fine-grained sand, trace clay, gray.	☒		3,4,5			wa (93%)
45		**SAND (SP):** fine-grained, gray.	■		6,18,22	26	97	
50		-fine-to medium-grained, trace gravel up to 1/4" in maximum dimension.	☒		4,7,10			wa (5%)
55		**SAND WITH SILT (SP-SM):** fine-grained, gray.	■		6,8,13	24		dist.
60		-trace gravel up to 1/3" in maximum dimension.	☒		6,10,15			wa (8%)
65			□		10,23,31			

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-4b

Log of Boring No. BH-3

Dates Drilled: 3/3/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 15

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND WITH SILT (SP-SM): fine-grained, gray.	X		7,10,16			
75		-trace gravel up to 1/2" in maximum dimension.			10,18,26			
80			X		8,20,28			wa (6%)
		End of boring at 81.5 feet. Groundwater encountered at 15' bgs during drilling. Boring backfilled with soil cuttings and surface patched with asphalt on 3-3-04.						

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-4c

Log of Boring No. BH-4

Dates Drilled: 3/2/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 10

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		2.5" ASPHALT CONCRETE						ei,r ca,er
		FILL (Af): **SILTY SAND (SM):** fine-grained, brown.						
		ALLUVIUM (Qal): **SILTY SAND (SM):** fine-grained, trace clay, gray.	■		6,5,4	23	100	
5		-trace dark gray organic, few shells, slight odor	■		4,6,6	25	94	c
10		**CLAY (CL):** trace dark gray organic, and fine-grained sand, gray.	■		5,5,6	62	66	pi
15		**SILTY SAND (SM):** fine-grained, gray to brown.	■		3,4,6	30	91	
20		**SILT (ML):** some fine-grained sand, trace clay, gray.	☒		5,6,7			wa (89%)
25		**SILTY SAND (SM):** fine-grained, gray.	■		5,6,14	29	92	
30		**SANDY SILT (ML):** fine-grained sand, dark gray.	☒		3,5,4			wa (63%)

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-5a

Log of Boring No. BH-4

Dates Drilled: 3/2/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 10

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SANDY SILT (ML): fine-grained sand, gray.	■		3,5,7	37	87	ds
40		-trace clay	☒		2,4,12			wa (68%)
45		**SILTY SAND (SM):** fine to medium-grained, gray with light gray.	■		8,22,30	23	103	
50		**SAND WITH SILT (SP-SM):** fine to medium-grained, gray.	☒		3,4,6			wa (9%)
55			☐		3,7,9			
60		**SAND (SP):** medium grained, gray.	☒		6,12,18			wa (5%)
		End of boring at 61.5 feet. Groundwater encountered at 10' bgs. Boring backfilled with soil cuttings and surface patched with asphalt on 3-2-04.						

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-5b

Log of Boring No. BH-5

Dates Drilled: 3/3/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 12

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" ASPHALT CONCRETE OVER 3" BASE						
		ALLUVIUM (Qal): **SILTY SAND (SM):** fine-grained, few clay, gray.						
			■		8,6,9	29	92	
5			■		2,5,14	21	103	
10		**CLAY (CL):** trace fine grained sand, some root, slight odor, gray.			5,5,3			
15		**SILTY SAND (SM):** fine-grained, gray.	■		6,10,8	23		dis.
20		-root	☒		4,4,5			wa (39%)
25			■		5,9,9	29	101	
30		-increasing sand content	☒		3,6,8			wa (30%)

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-6a

Log of Boring No. BH-5

Dates Drilled: 3/3/2004 Logged by: CBOO Checked By: KF

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 10 Depth to Water (ft): 12

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SANDY SILT (ML): trace fine-grained sand, gray.	■		5,5,9	27	95	
40		**SILT (ML):** trace fine-grained sand, some clay, gray.	☒		4,4,4			wa (98%)
45		**SAND WITH SILT (SP-SM):** fine-grained, gray.	■		7,27,28	24	99	
50		-fine to medium-grained	☒		2,2,4			wa (11%)
55		**SILTY SAND (SM):** fine-grained, gray.	■		8,9,13	21	107	
60		**SAND WITH SILT (SP-SM):** fine to medium-grained, gray.	☒		7,12,14			wa (12%)

End of boring at 61.5 feet.
Groundwater encountered at 12' bgs during drilling and stabilized at 13' after 15 minutes.
Boring backfilled with soil cuttings and surface patched with asphalt on 3-3-04.

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
A-6b

FUGRO GEOSCIENCES, INC.

13049 East Florence Avenue
Santa Fe Springs, CA 90670
Phone : 562-903-0055
Fax: 562-903-9005

March 8, 2004
Report Number 0303-0945

Converse Consultants
222 E. Huntington Drive
Suite 211
Monrovia, California 9106

Attention: Mr. Keyvan Fotoohi

REPORT FOR
CONE PENETRATION TESTING
AND RELATED SERVICES
LONG BEACH, CALIFORNIA

Dear Mr. Fotoohi:

Please find enclosed herewith the final results of the cone penetration tests conducted at the above referenced location.

For your information, the soil stratigraphy was identified using Campanella and Robertson's Simplified Soil Behavior Chart. Please note that because of the empirical nature of the soil behavior chart, the soil identification should be verified locally.

Fugro Geosciences, Inc. appreciates the opportunity to be of service to your organization. If you should have any questions, or if we can be of further assistance, please do not hesitate to contact us. We look forward to working with you in the future.

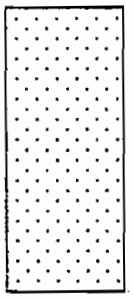
Very truly yours,
FUGRO GEOSCIENCES, INC.

Recep Yilmaz
President

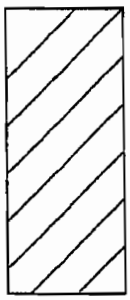
RY/jm

1 Diskette Enclosed

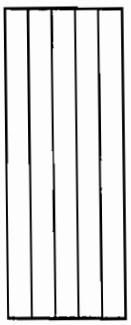
KEY TO SOIL BEHAVIOR TYPE



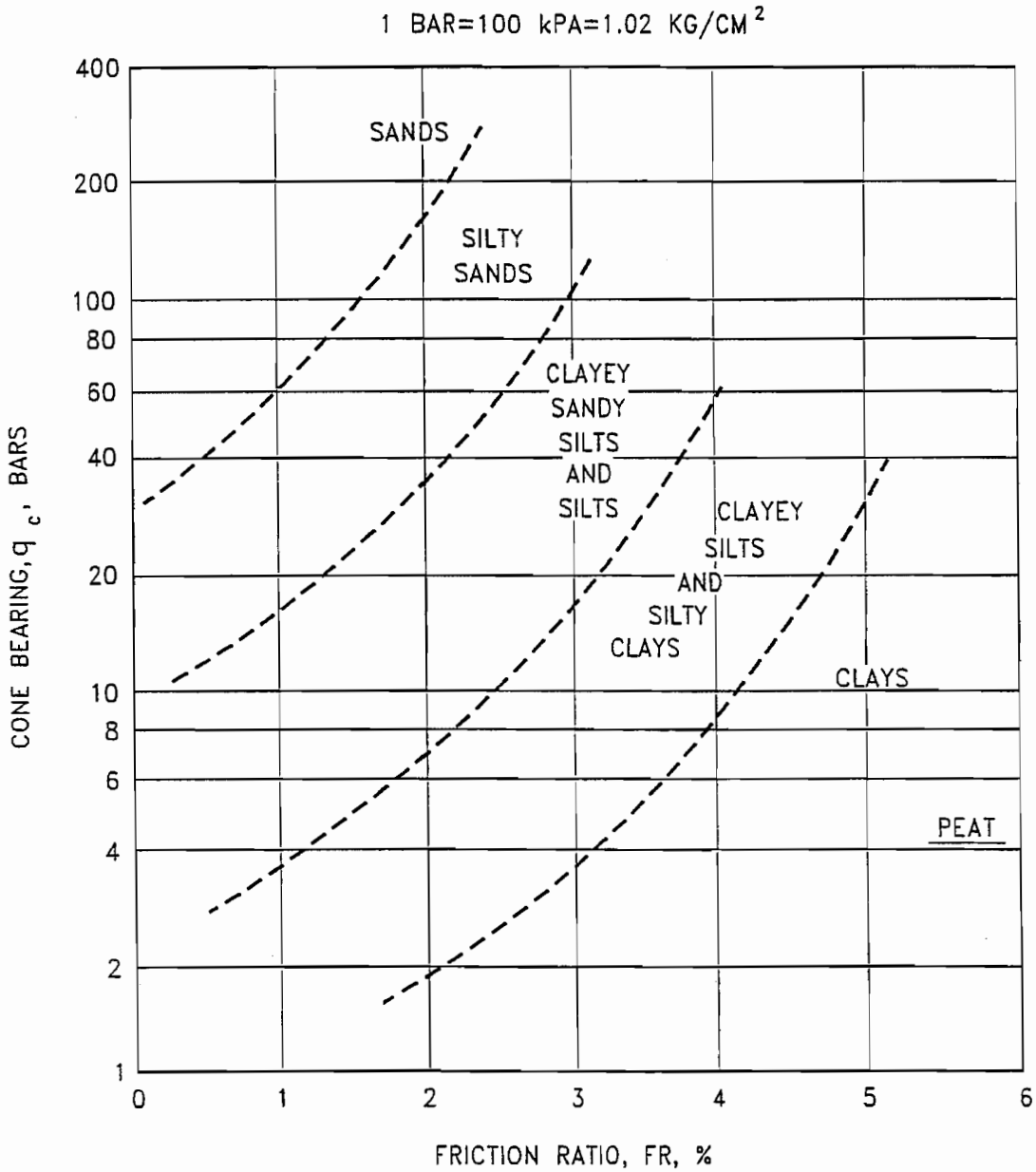
SAND AND SANDY SOIL



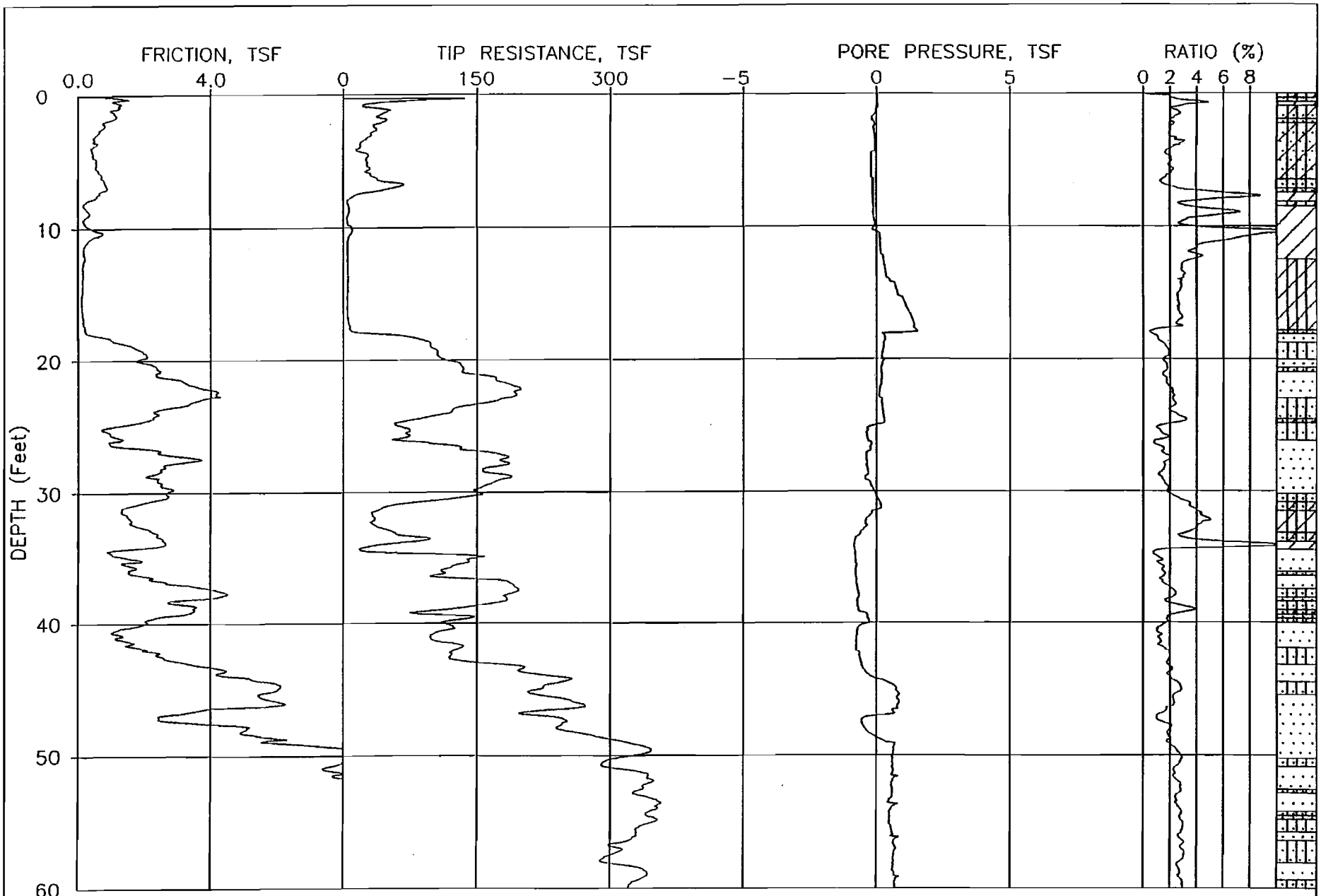
CLAY AND CLAYEY SOIL



SILT AND SILTY SOIL



MODIFIED CAMPANELLA AND ROBERTSON SOIL BEHAVIOR CHART (1983)



JOB NUMBER: 03-0945

CPT NUMBER: C01

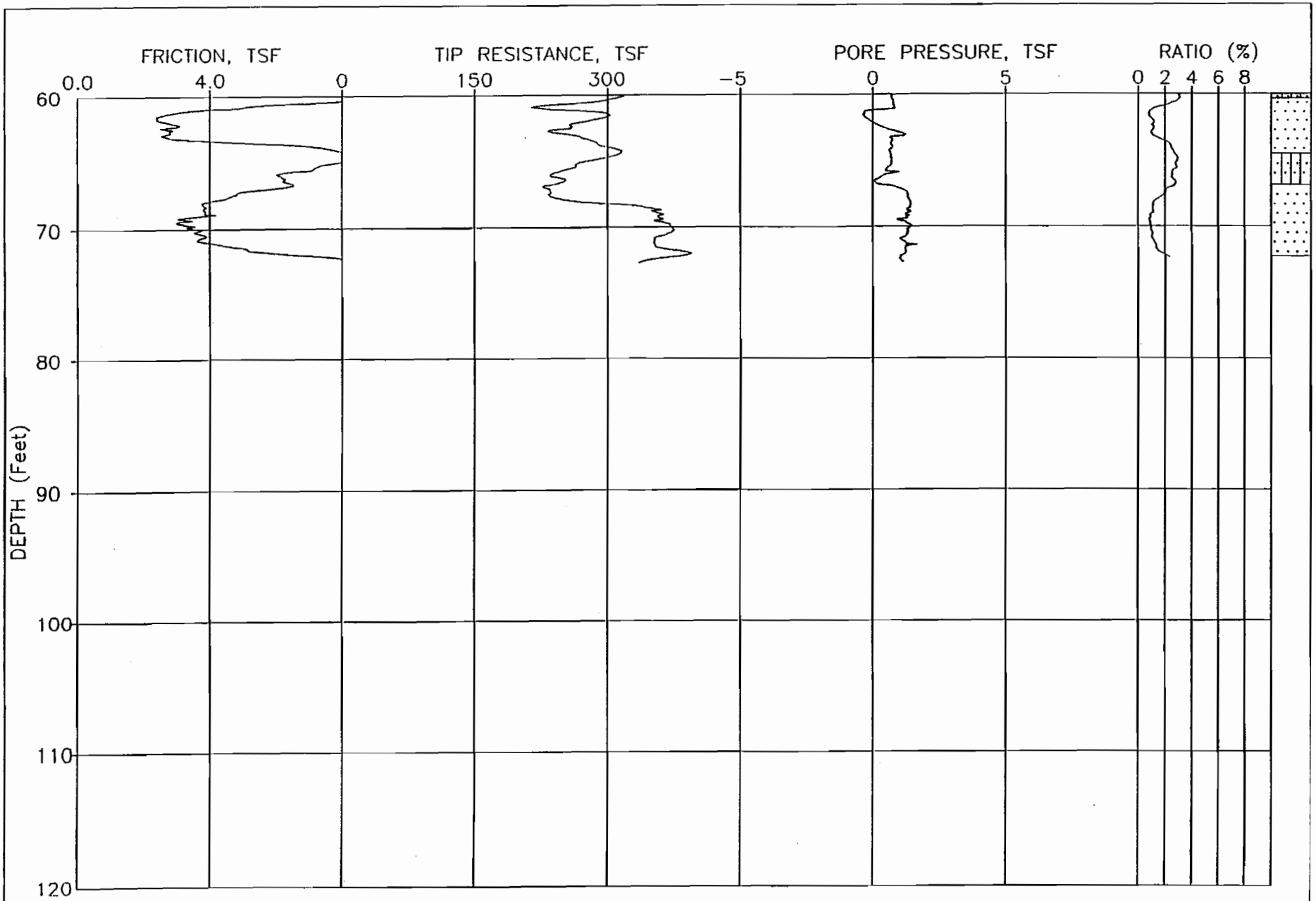
DATE: 03-02-2004

ELEVATION: 0.00

CONE NUMBER: F7.5CKEW1170

PLATE: 1 OF 2

FUGRO GEOSCIENCES,INC



JOB NUMBER: 03-0945

CPT NUMBER: C01

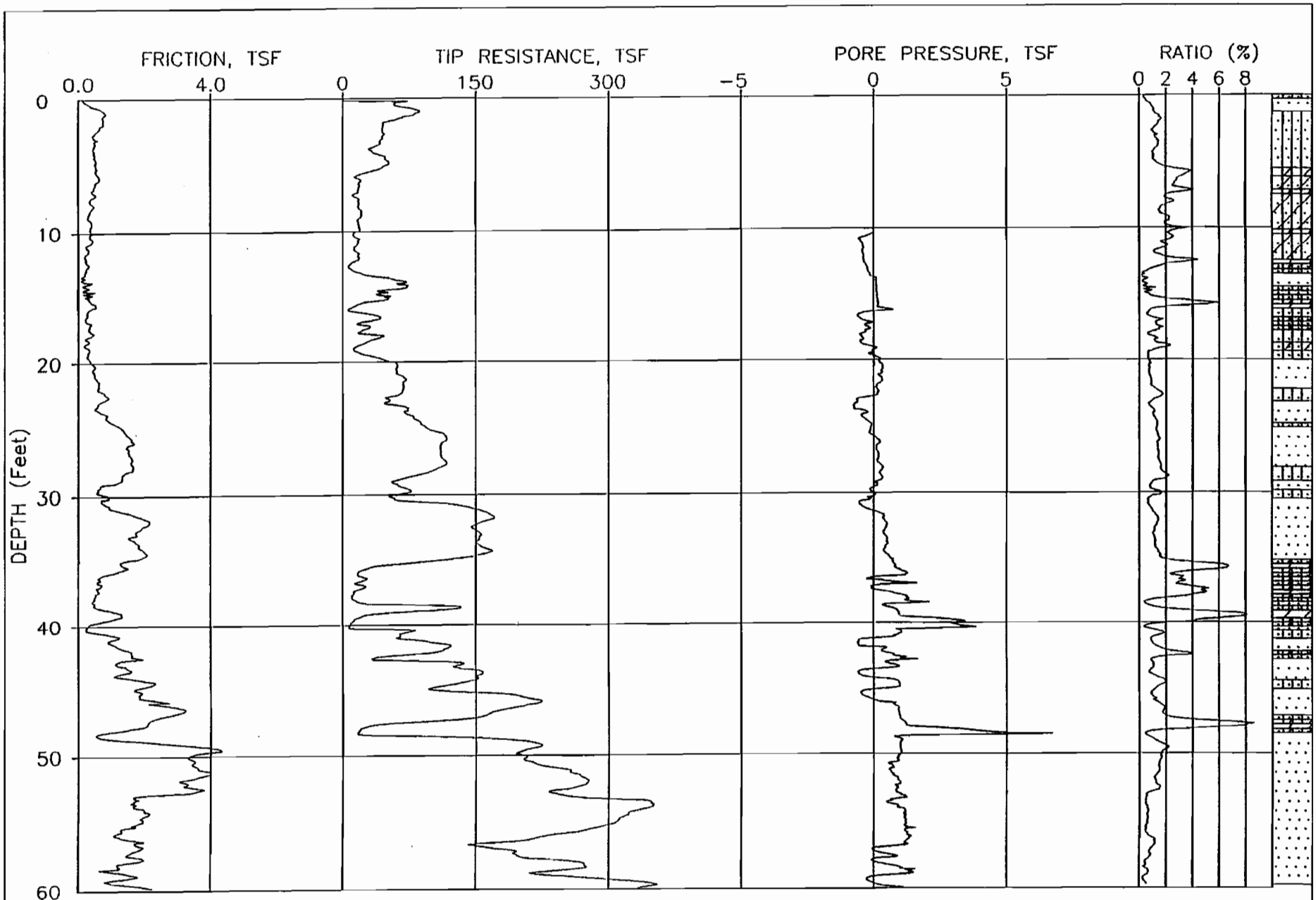
DATE: 03-02-2004

ELEVATION: 0.00

CONE NUMBER: F7.5CKEW1170

PLATE: 2 OF 2

FUGRO GEOSCIENCES,INC



JOB NUMBER: 03-0945

ELEVATION: 0.00

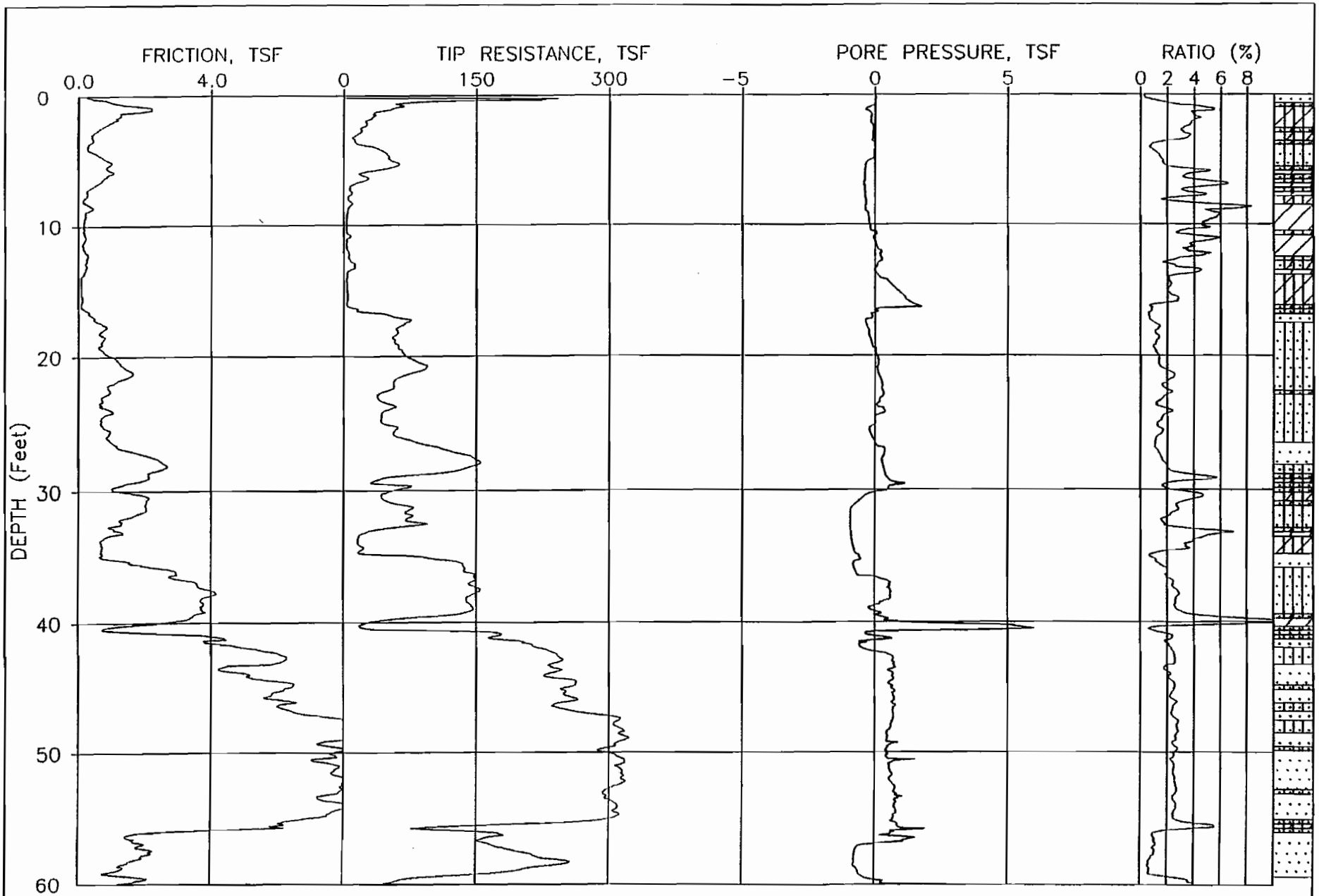
CPT NUMBER: C02

CONE NUMBER: F7.5CKEW1170

DATE: 03-02-2004

PLATE: 1 OF 1

FUGRO GEOSCIENCES,INC



JOB NUMBER: 03-0945

CPT NUMBER: C03

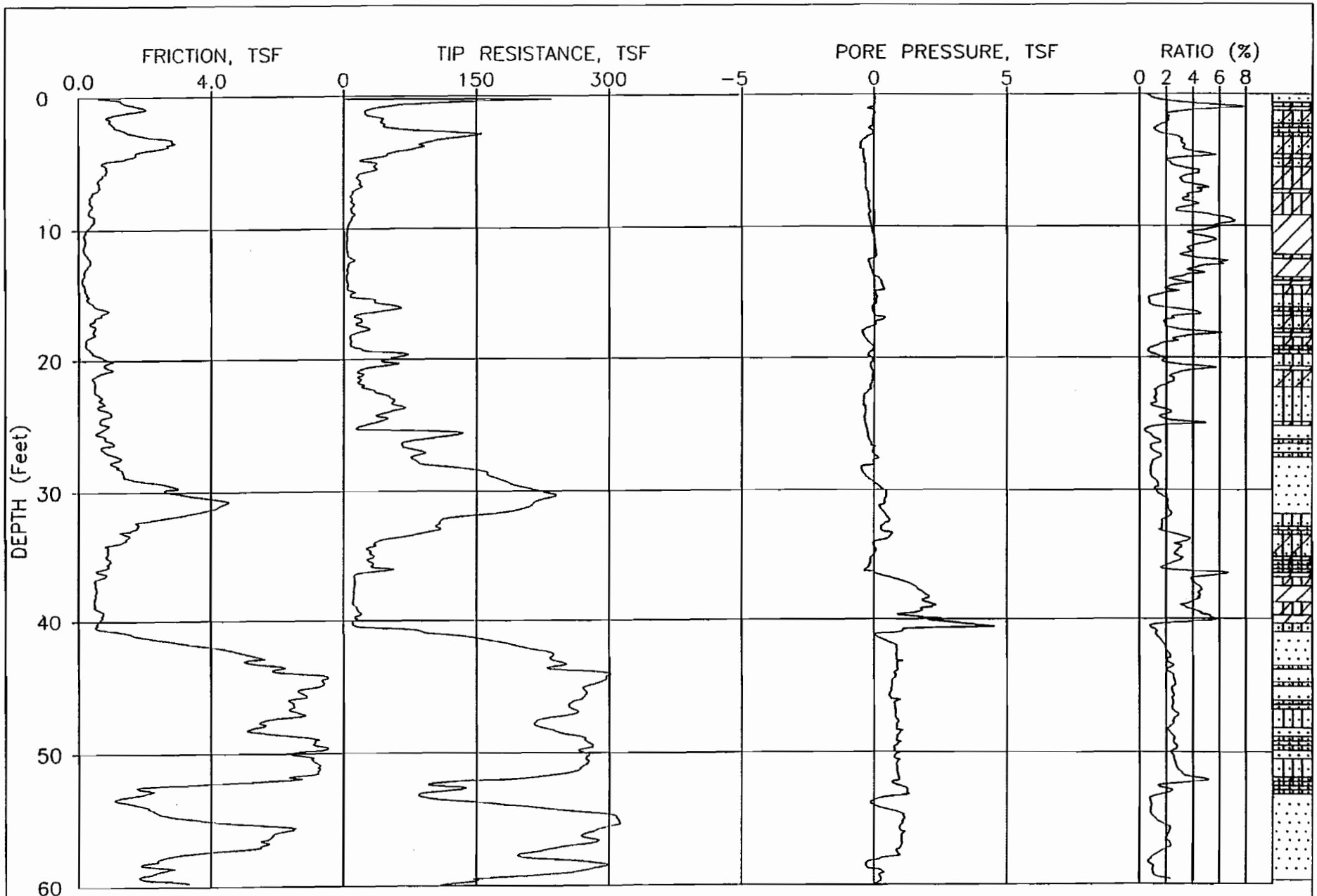
DATE: 03-02-2004

ELEVATION: 0.00

CONE NUMBER: F7.5CKEW1170

PLATE: 1 OF 1

FUGRO GEOSCIENCES,INC



JOB NUMBER: 03-0945

CPT NUMBER: C04

DATE: 03-02-2004

ELEVATION: 0.00

CONE NUMBER: F7.5CKEW1170

PLATE: 1 OF 1

FUGRO GEOSCIENCES,INC

APPENDIX B

LABORATORY TESTING RESULTS

APPENDIX B

LABORATORY TESTING PROGRAM

Tests were conducted in our geotechnical laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical parameters required for the design and construction of the project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

Grain-Size Distribution

The grain-size analysis covers the quantitative determination of the distribution of particle sizes in soils. The particle size distribution is used to aid in classification of the soil. Tests were performed on representative soil samples in general accordance with the ASTM Standard D422-63 test method. For test results, see Drawing No. B-1, *Grain Size Distribution Results.*

Soils Finer Than The #200 Sieve

Twenty seven (27) selected samples were tested in accordance with the ASTM Standard D1140-92 test method to determine the amounts of materials finer than U.S. Standard Sieve No. 200. This information was used to aid in classification of the soil as well as the liquefaction analysis. This information is summarized in Table No. B-1, *Percent Finer Than #200 Sieve Test Results.*

Table No. B-1, Percent Finer Than #200 Sieve Test Results

Boring No.	Depth (ft)	Soil Classification	Percent Finer Than No. 200 Sieve (%)
BH-1	15	Sandy Silt (ML)	56
BH-1	20	Silty Sand (SM)	45
BH-1	30	Sand with Silt (SP-SM)	11
BH-1	40	Silty Sand (SM)	19
BH-1	50	Sand with Silt (SP-SM)	6
BH-1	60	Sand (SP)	4
BH-2	20	Sandy Silt (ML)	53
BH-2	30	Silt (ML)	86
BH-2	40	Silt (ML)	92
BH-2	50	Sand (SP)	5
BH-2	60	Sand with Silt (SP-SM)	12
BH-3	20	Silt (ML)	90
BH-3	30	Silty Sand (SM)	25

Boring No.	Depth (ft)	Soil Classification	Percent Finer Than No. 200 Sieve (%)
BH-3	40	Silt (ML)	93
BH-3	50	Sand (SP)	5
BH-3	60	Sand with Silt (SP-SM)	8
BH-3	80	Sand with Silt (SP-SM)	6
BH-4	20	Silt (ML)	89
BH-4	30	Sandy Silt (ML)	63
BH-4	40	Sandy Silt (ML)	68
BH-4	50	Sand with Silt (SP-SM)	9
BH-4	60	Sand (SP)	5
BH-5	20	Silty Sand (SM)	39
BH-5	30	Silty Sand (SM)	30
BH-5	40	Silt (ML)	98
BH-5	50	Sand with Silt (SP-SM)	11
BH-5	60	Sand with Silt (SP-SM)	12

Atterberg Limits

Atterberg Limits test was performed on one (1) representative sample to assist the classification of the soils according to ASTM Standard D4318 test method. The test result is presented in the following table.

Table No. B-2, Atterberg Limits Test Results

Boring No.	Depth (ft)	Soil Classification	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)
BH-4	10	CLAY (CL)	39	19	20

Laboratory Maximum Density Test

One (1) representative bulk sample was tested in the laboratory to determine the maximum dry density and optimum moisture content. The test was conducted in accordance with the ASTM Standard D1557-00 laboratory procedure. The test results are presented in Drawing No. B-2, *Moisture-Density Relationship Results*.

Direct Shear Tests

Four (4) direct shear tests were performed on relatively undisturbed soil samples. Three samples for each test contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. Each sample was then sheared at a constant strain rate. Shear deformation was recorded until a maximum of about 0.25 inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and

plotted to determine the shear strength parameters. For test data, including sample density and moisture content see Drawings. Nos. B-3 through D-6, *Direct Shear Test Results,* and in the following table.

Table No. B-3, Summary of Direct Shear Test Results

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-1	5	Silty Sand (SM)	30	100
BH-2	45	Silty Sand (SM)	30	100
BH-3	25	Silty Sand (SM)	32	0
BH-4	35	Sandy Silt (ML)	31	100

Consolidation Tests

Data obtained from consolidation test performed on two (2) relatively undisturbed soil samples were used to evaluate the settlement characteristics of the on-site soils under load. Preparation for this test involved trimming the sample, placing it in a one-inch-high brass ring, and loading it into the test apparatus, which contained porous stones to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state of equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load.

The representative samples were tested at both field moisture and submerged conditions. For test results, including sample density and moisture content, see Drawings Nos. B-7 and B-8, *Consolidation Test Results.*

Expansion Index Tests

One (1) representative bulk sample was tested for expansion index to evaluate the expansion potential of material encountered at the site. The test was conducted in accordance with California Building Code (CBC) Standard. For test results see the following table.

Table No. B-4, Summary of Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index
BH-4	0 – 5	Silty Sand (SM)	26

Sand Equivalent

One (1) representative bulk sample was tested for sand equivalent. The test was conducted in accordance with California Building Code (CBC) Standard. For test results see the following table.

Table No. B-5, Summary of Sand Equivalent Test Results

Boring No.	Depth (feet)	Soil Description	Sand Equivalent
BH-1	0 – 3	Sandy Silt (ML)	3

R-value Test

One (1) representative bulk soil sample was tested for resistance value (R-value) in accordance with State of California Standard Method 301-G. This test is designed to provide a relative measure of soil strength for use in pavement design. For the test result see the following table.

Table No. B-6, R-value Test Results

Boring No.	Depth (feet)	Soil Description	R-value
BH-4	0 – 5	Silty Sand (SM)	41

Chemical Test

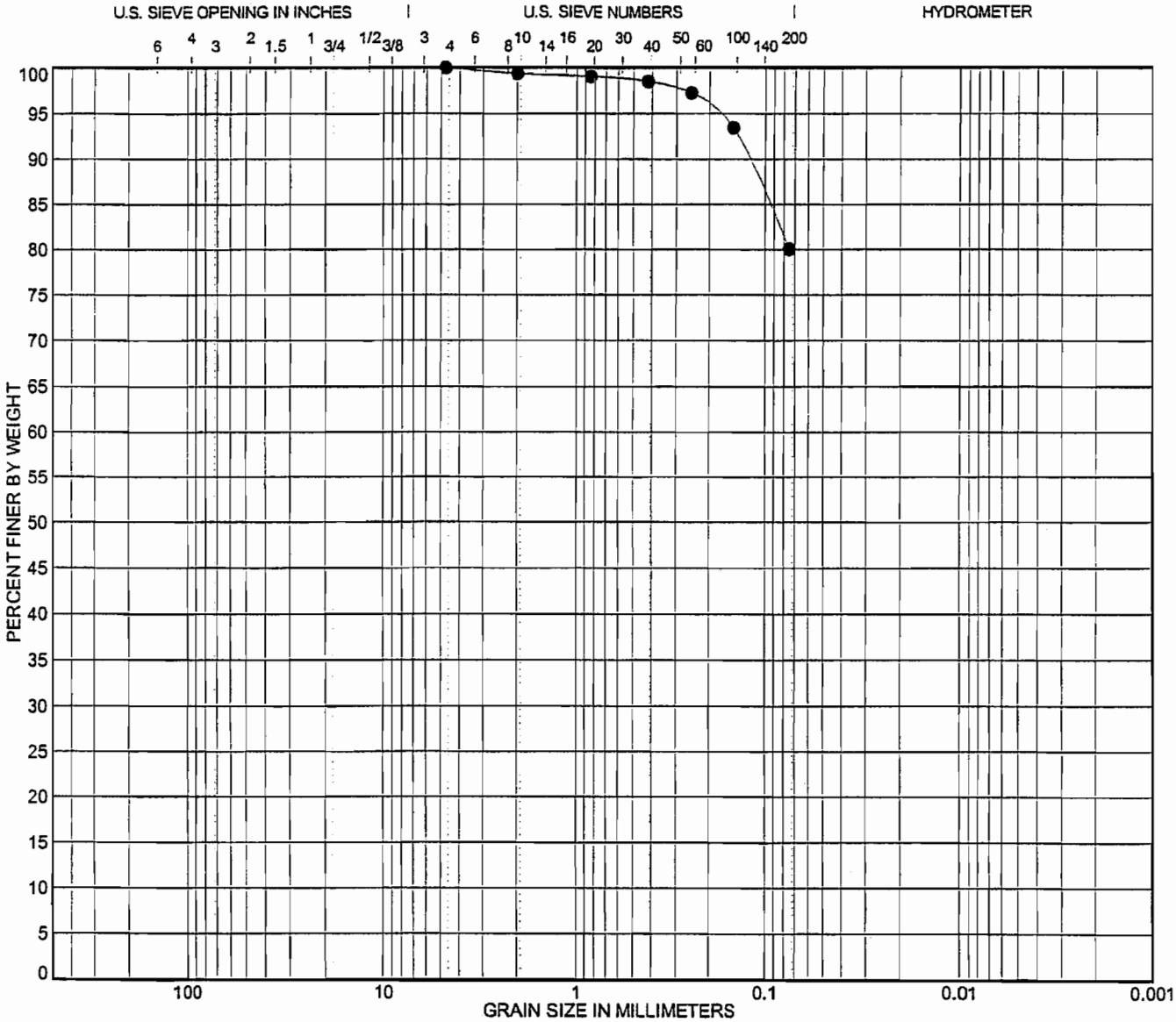
One (1) representative sample was tested for corrosivity potential evaluation. Minimum electrical resistivity, pH, chloride content, and soluble sulfate were determined using California Tests 532, 643, 422, and 417, respectively. For the test result see the following table.

Table No. B-7, Soil Corrosivity Test Results

Boring No./Depth	pH	Chloride (ppm)	Sulfate (% by weight)	Min. Saturated Resistivity (ohm-cm)
BH-4/ 0-5	7.42	110	0.024	875

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-1**	**0-3**	**SANDY SILT (ML)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-1**	**0-3**	**4.75**				**0.0**	**20.0**	**80.0**	

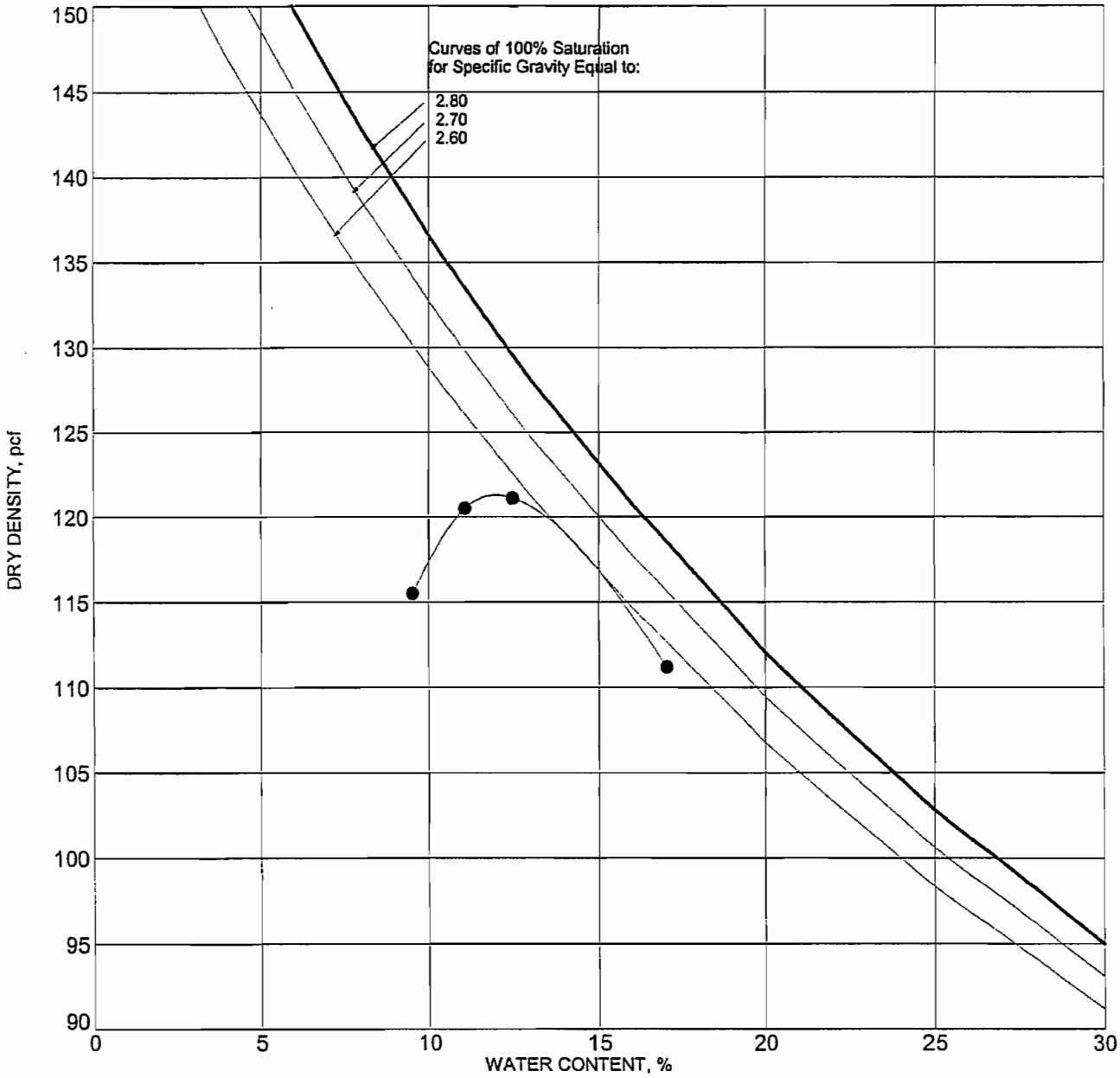
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-2	0-5	SILTY SAND (SM)	D1557 Method A	12.1	121.3

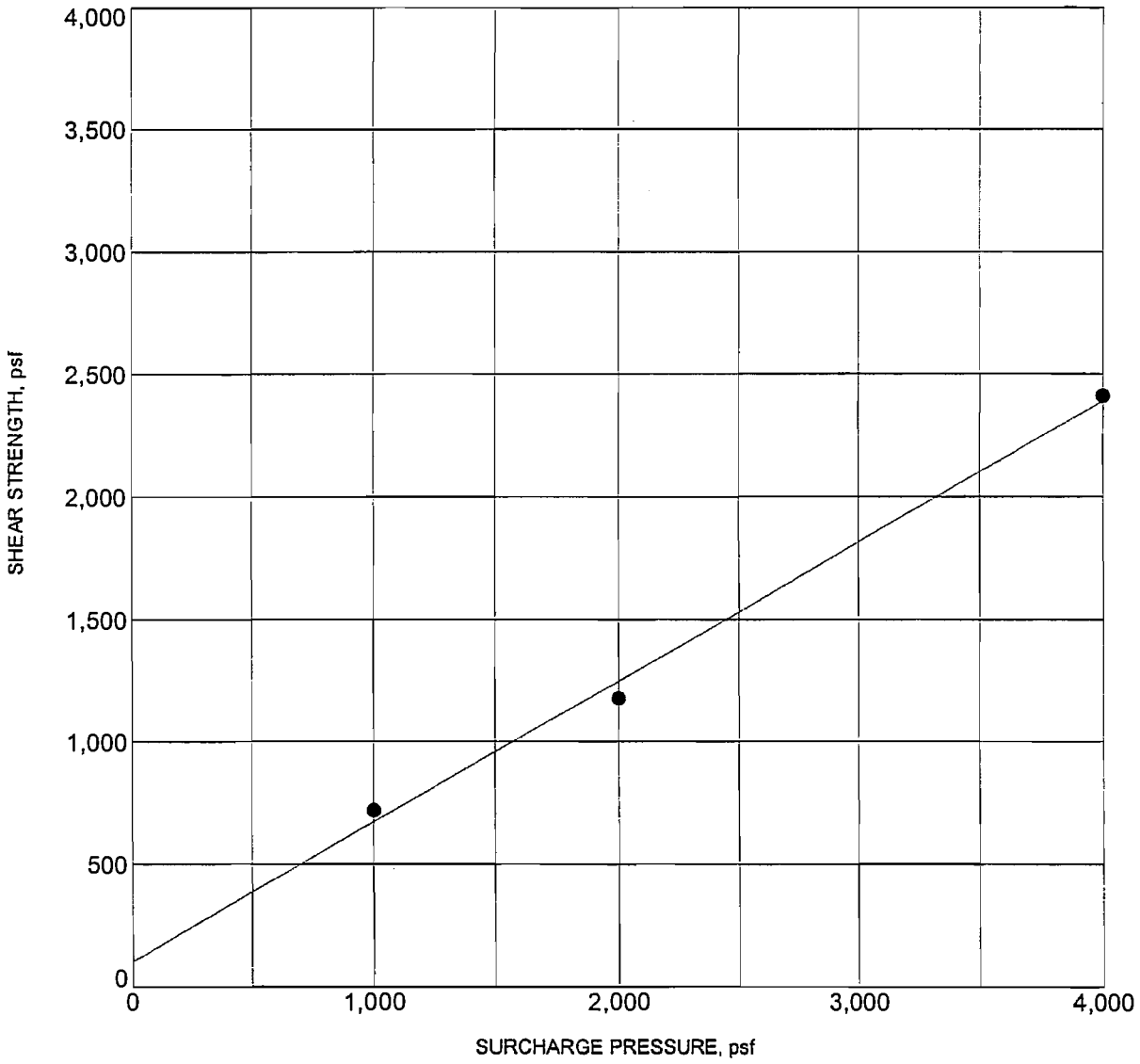
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
B-2



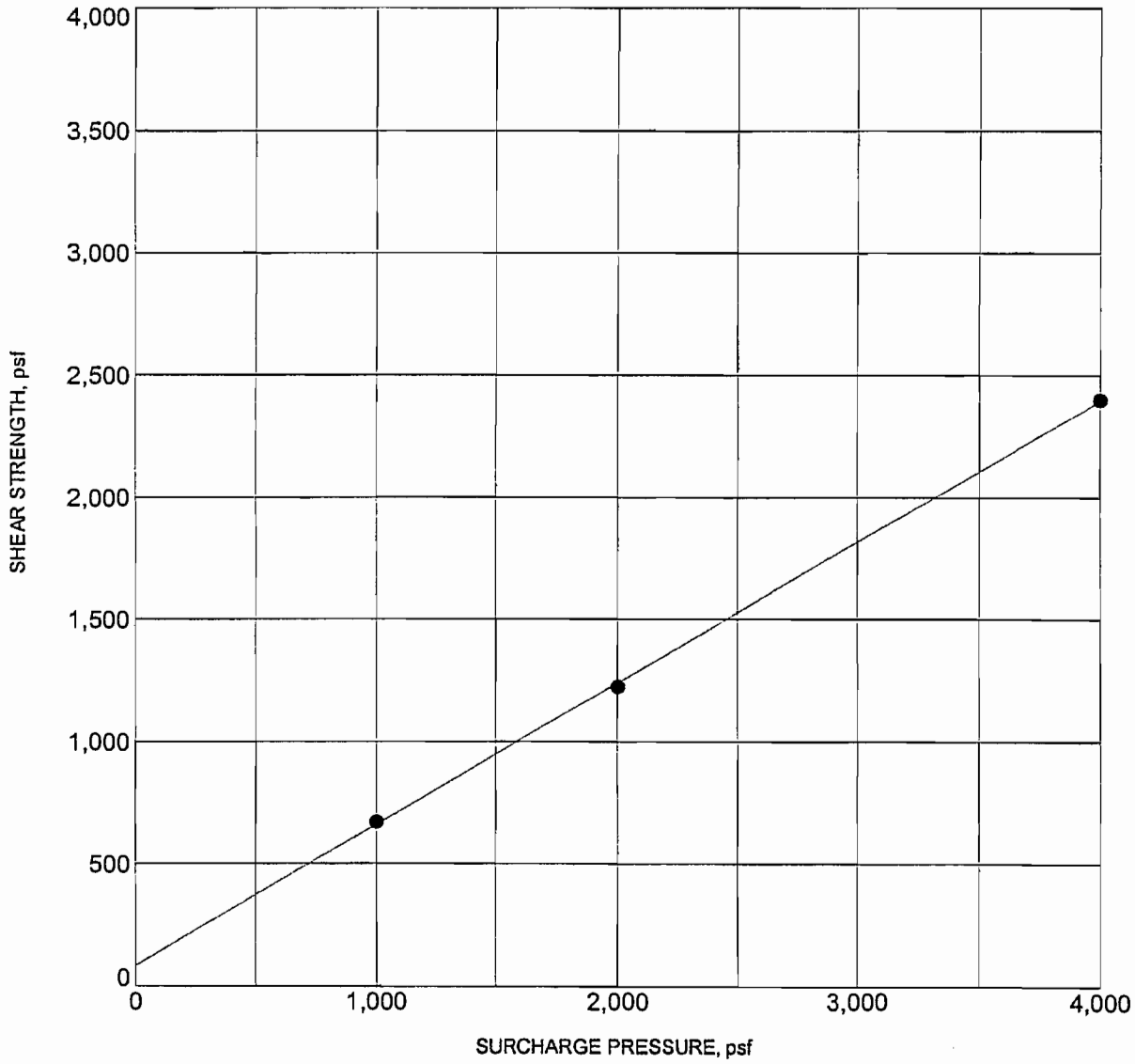
BORING NO. :	BH-1	DEPTH (ft) :	5
DESCRIPTION :	SILTY SAND (SM)		
COHESION (psf) :	100	FRICTION ANGLE (degrees):	30
MOISTURE CONTENT (%) :	27	DRY DENSITY (pcf) :	97

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
MIXED-USE COMMUNITY LONG BEACH, CALIFORNIA FOR: LENNAR COMMUNITIES	04-31-118-01	B-3



BORING NO.	:	BH-2	DEPTH (ft)	:	45
DESCRIPTION	:	SILTY SAND (SM)			
COHESION (psf)	:	100	FRICTION ANGLE (degrees):		30
MOISTURE CONTENT (%)	:	26	DRY DENSITY (pcf)	:	95

NOTE: Ultimate Strength.

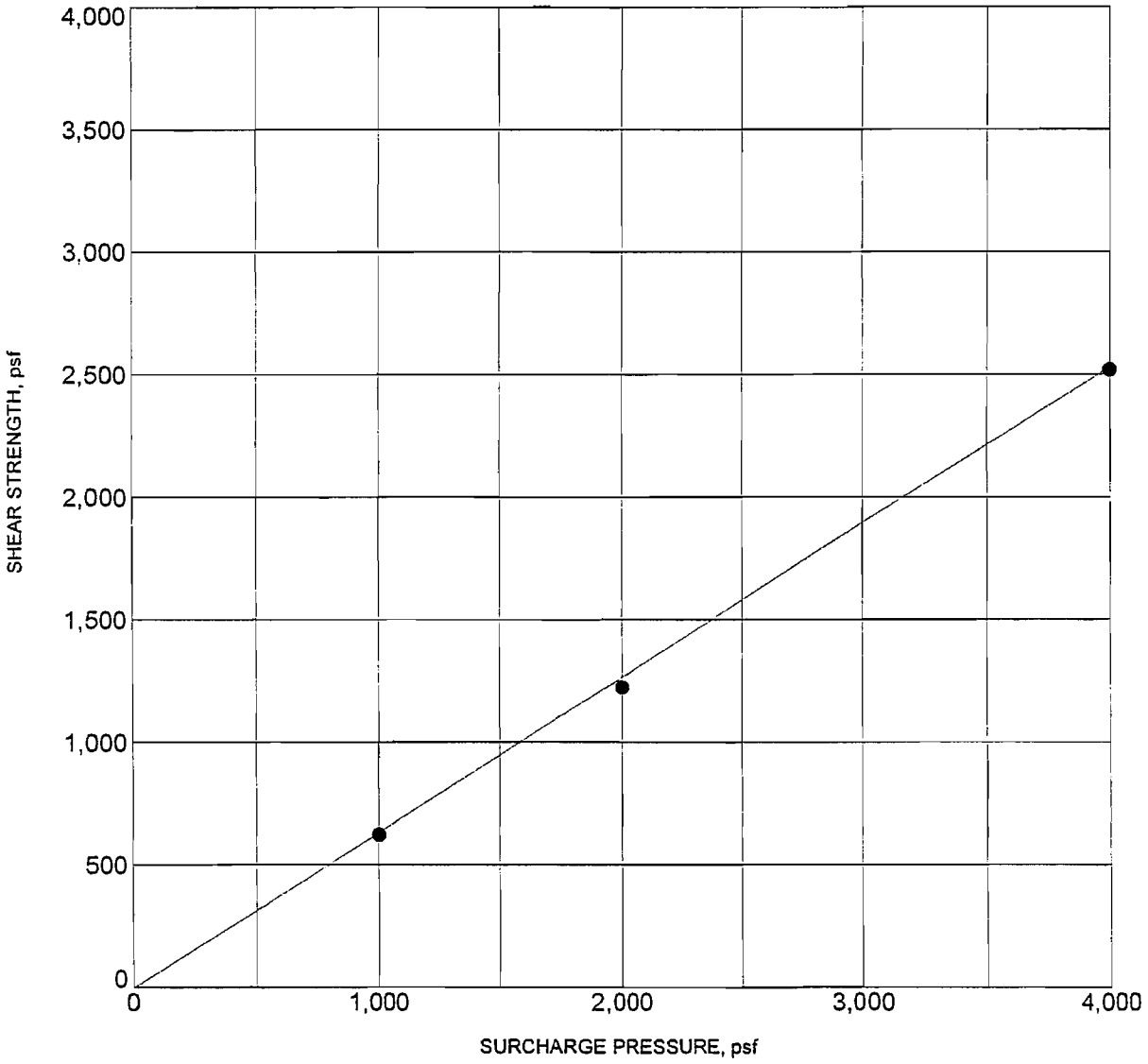
DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
B-4



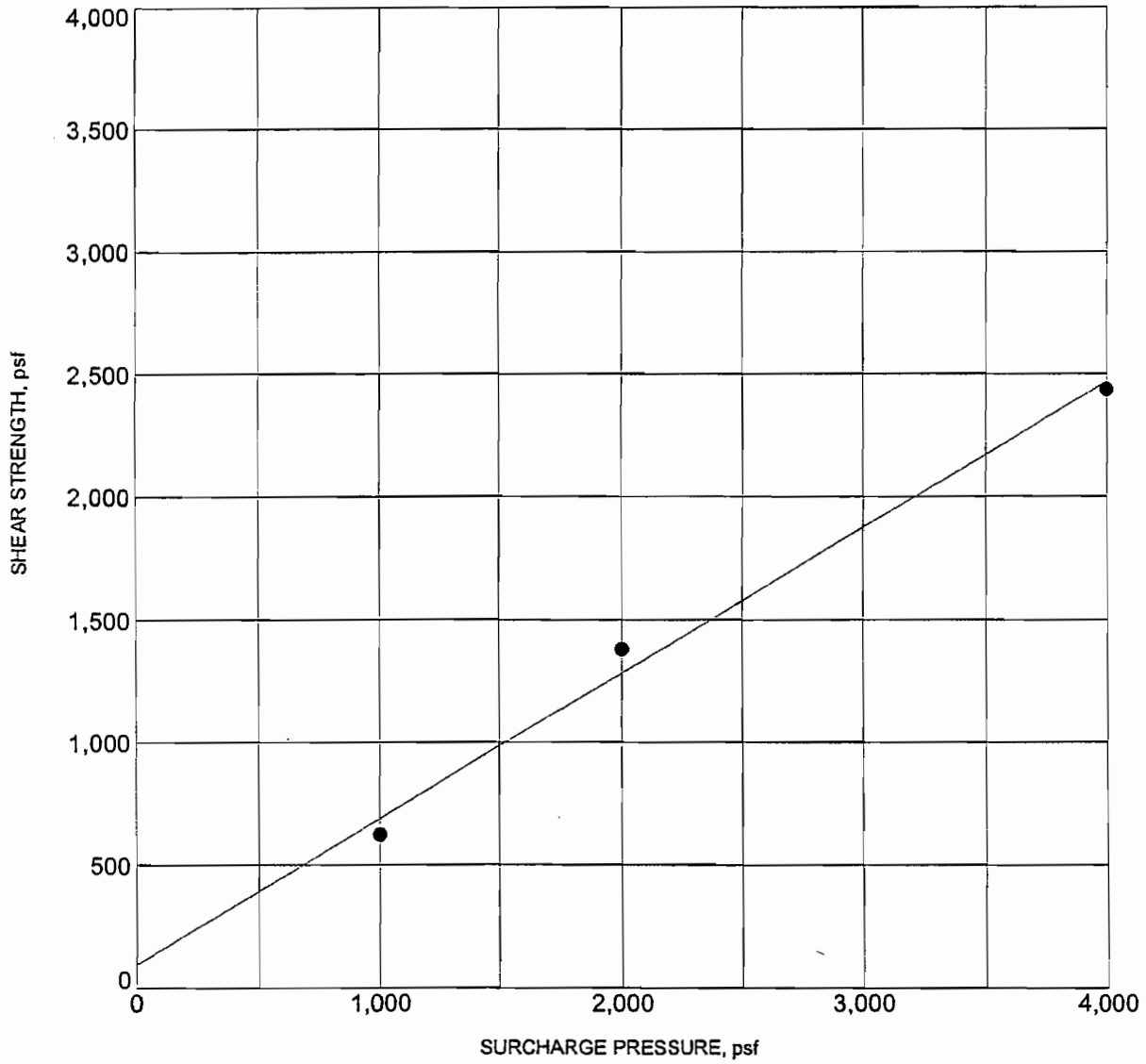
BORING NO. :	BH-3	DEPTH (ft) :	25
DESCRIPTION :	SILTY SAND (SM)		
COHESION (psf) :	0	FRICTION ANGLE (degrees):	32
MOISTURE CONTENT (%) :	33	DRY DENSITY (pcf) :	90

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
MIXED-USE COMMUNITY LONG BEACH, CALIFORNIA FOR: LENNAR COMMUNITIES	04-31-118-01	B-5



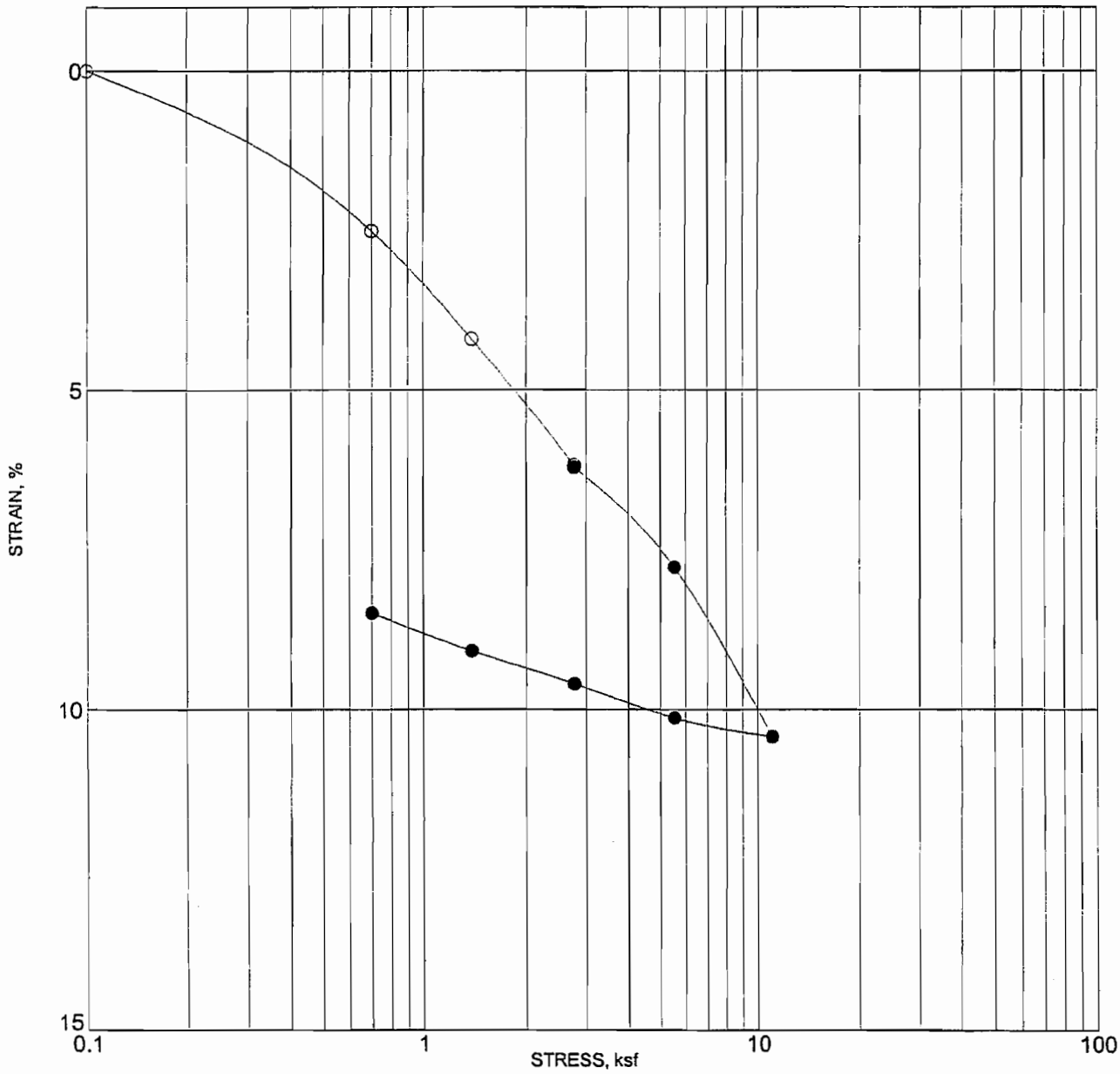
BORING NO.	:	BH-4	DEPTH (ft)	:	35
DESCRIPTION	:	SANDY SILT (ML)			
COHESION (psf)	:	100	FRICTION ANGLE (degrees):		31
MOISTURE CONTENT (%)	:	37	DRY DENSITY (pcf)	:	87

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
MIXED-USE COMMUNITY LONG BEACH, CALIFORNIA FOR: LENNAR COMMUNITIES	04-31-118-01	B-6



BORING NO. :	BH-3	DEPTH (ft) :	10
DESCRIPTION :	SILTY SAND (SM)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	38.8	82.1	98	1.085
FINAL	34.8	89.1	100	0.921

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

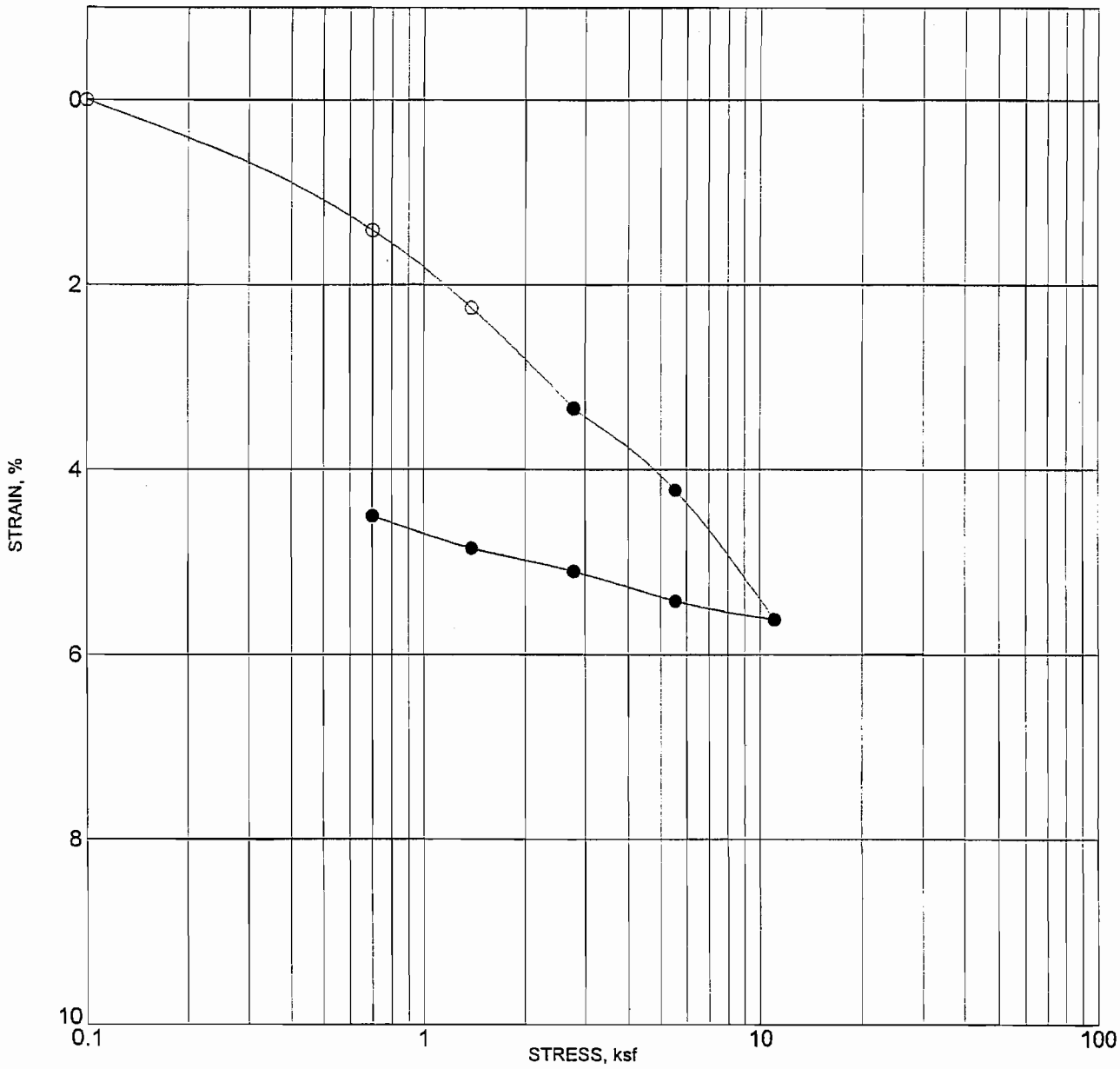
CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
B-7



BORING NO. :	BH-4	DEPTH (ft) :	5
DESCRIPTION :	SILTY SAND (SM)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	25.4	93.9	89	0.755
FINAL	26	98.2	100	0.680

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
MIXED-USE COMMUNITY
LONG BEACH, CALIFORNIA
FOR: LENNAR COMMUNITIES

Project No.
04-31-118-01

Drawing No.
B-8

SUMMARY OF CORROSION TEST RESULTS

PROJECT NAME: Lennar Com/Long Beach

EGL JOB NO.: 04-118-003

PROJECT NO.: 04-31118-01

CLIENT: Converse Consultants

DATE: 03-11-04

SUMMARIZED BY: VW

TEST PIT NO	SAMPLE NO	DEPTH (ft)	pH CALTRANS 643	CHLORIDE CONTENT CALTRANS 422 (ppm)	SULFATE CONTENT CALTRANS 417 (% by weight)	MINIMUM RESISTIVITY CALTRANS 532 (ohm-cm)
BH-4	N/A	0-5	7.42	110	0.024	875

APPENDIX C

LIQUEFACTION ANALYSIS

APPENDIX C

LIQUEFACTION-ANALYSIS

The subsurface data obtained from exploratory borings and CPTs were used to evaluate the liquefaction potential of the subject site. The Logs of Borings and the CPTs sounding results are presented in Appendix A, *Field Exploration.* Due to the proposed two-story subterranean garage, the liquefaction analysis was conducted for both the site soil below the ground surface before the excavation and the site soil below the proposed garage floor after the excavation. For the liquefaction analysis, it was assumed that the garage floor is located at 25 feet below existing ground surface.

Liquefaction evaluation was performed utilizing SPT as well as CPT data. Liquefaction analyses at the location of the exploratory borings were performed in accordance with the method suggested by Seed et. al (1985) and methods published by Southern California Earthquake Center (March 1999). The earthquake magnitude of M 6.8 and peak ground acceleration of 0.49g, where g is the acceleration due to gravity was selected for this analysis.

The site soil below the groundwater table has SPT blow counts ranging from 3 to 50 blows per foot with silt and clay content ranging from 4 to 98 percent. These soils may be susceptible to liquefaction under earthquake ground shaking. Due to the limitation of SPT sampling performed by hollow-stem auger drill rig and five (5) interval sampling frequency, liquefaction potential of site soils between 10 to 40 feet below the existing ground surface could not be established with certainty.

Liquefaction analyses at the locations of the CPT soundings were performed utilizing the CPT Liquefaction–Potential Relationship suggested by Seed et. al (1985) and guidelines published by Southern California Earthquake Center (March 1999). In performing this analysis, the measured cone tip resistance was corrected for overburden pressure in accordance with the method suggested by Seed et. al (1985). The Critical Penetration Resistance for a given depth is defined as the threshold value of the corrected tip resistance above which liquefaction is not likely to occur.

Silty sand and sand layers with corrected tip resistance values less than the Critical Penetration Resistance are likely to liquefy in the event of a significant seismic event associated with the nearby faults.

Site soils with slightly higher values of corrected tip resistance than the Critical Penetration Resistance may be marginally liquefiable. In general, however, soils showing higher corrected tip resistance than the Critical Penetration Resistance is not considered to be susceptible to liquefaction.

Subsurface soils below the designed groundwater table at depths of 10 to 40 feet have a Corrected Tip Resistance (Qc1n) less than Critical Penetration resistance (QPR). Therefore, it is our opinion that the CPT data, in conjunction with SPT data, will provide more reliable liquefaction analyses. At localized area at depths between 10 to 40 feet below the existing ground surface, site soils are marginally susceptible to liquefaction under earthquake ground shaking.

Seismically-Induced Ground Settlement

Tokimatsu and Seed (1987) present a simplified method for the evaluation of settlement in sands due to earthquake loading.

Analysis for seismically-induced settlement for the proposed additions was performed utilizing SPT as well as CPT data.

This analysis was performed by utilizing the *equivalent* corrected SPT blow counts, $(N_1)_{60}$ corresponding to the measured CPT tip resistance as interpreted by Furgo Geosciences, Inc., and included in Appendix A, *Field Exploration*. These *equivalent* corrected SPT $(N_1)_{60}$ reported by Furgo Geosciences, Inc., agrees well with our comparison of the corrected SPT blow counts and the CPT tip resistances for the locations where SPT and CPT tests were performed at adjacent locations.

Before the excavation, the total seismically induced settlement ranges from 6.5 inches to 8.3 inches, and the differential settlement considered to be half of the total settlement.

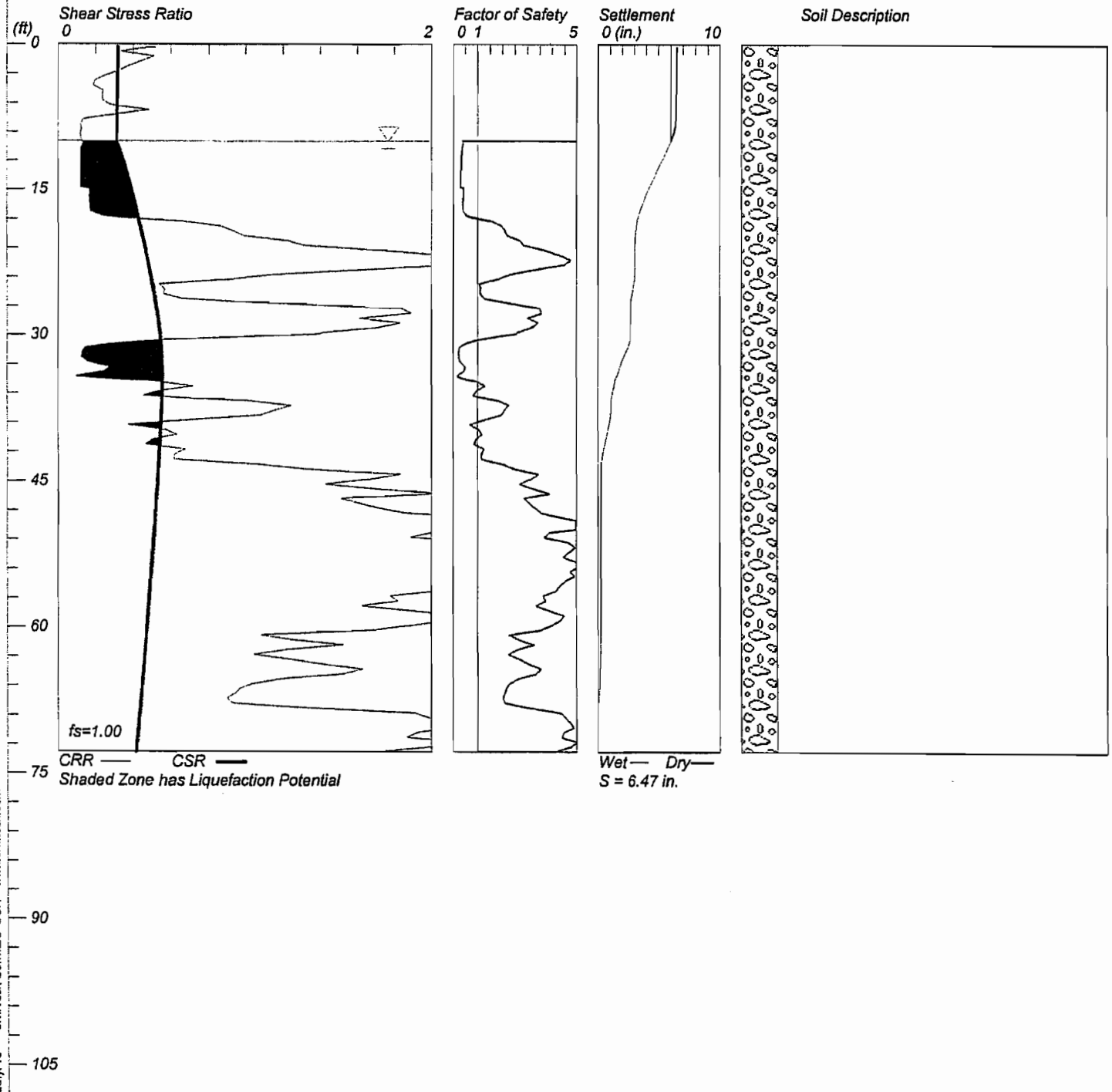
Considering 25 feet over excavation, the total seismically induced settlement ranges from 2.6 inches to 5.7 inches, and the differential settlement can be assume to be half of the total settlement.

LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-1 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

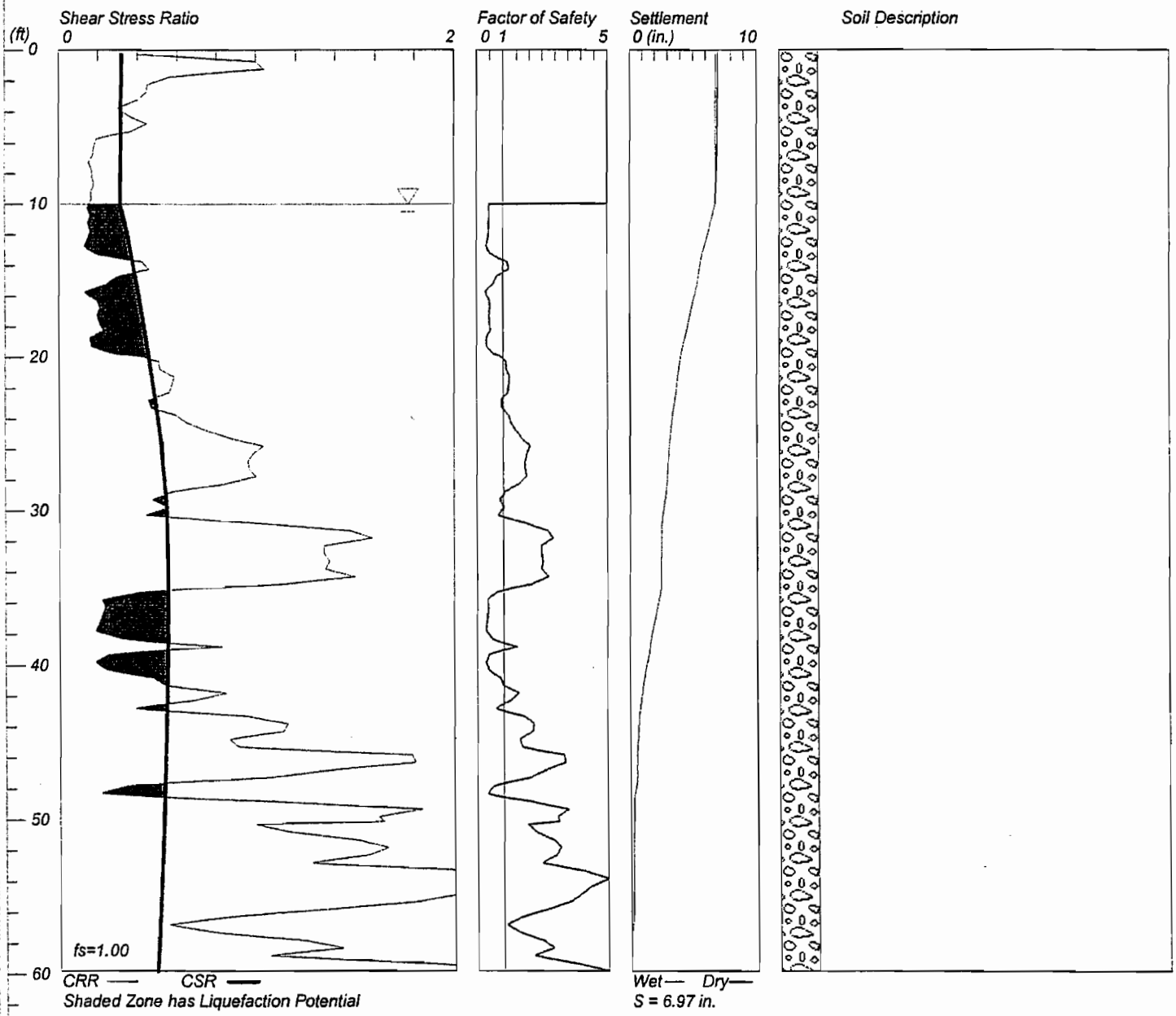


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-2 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

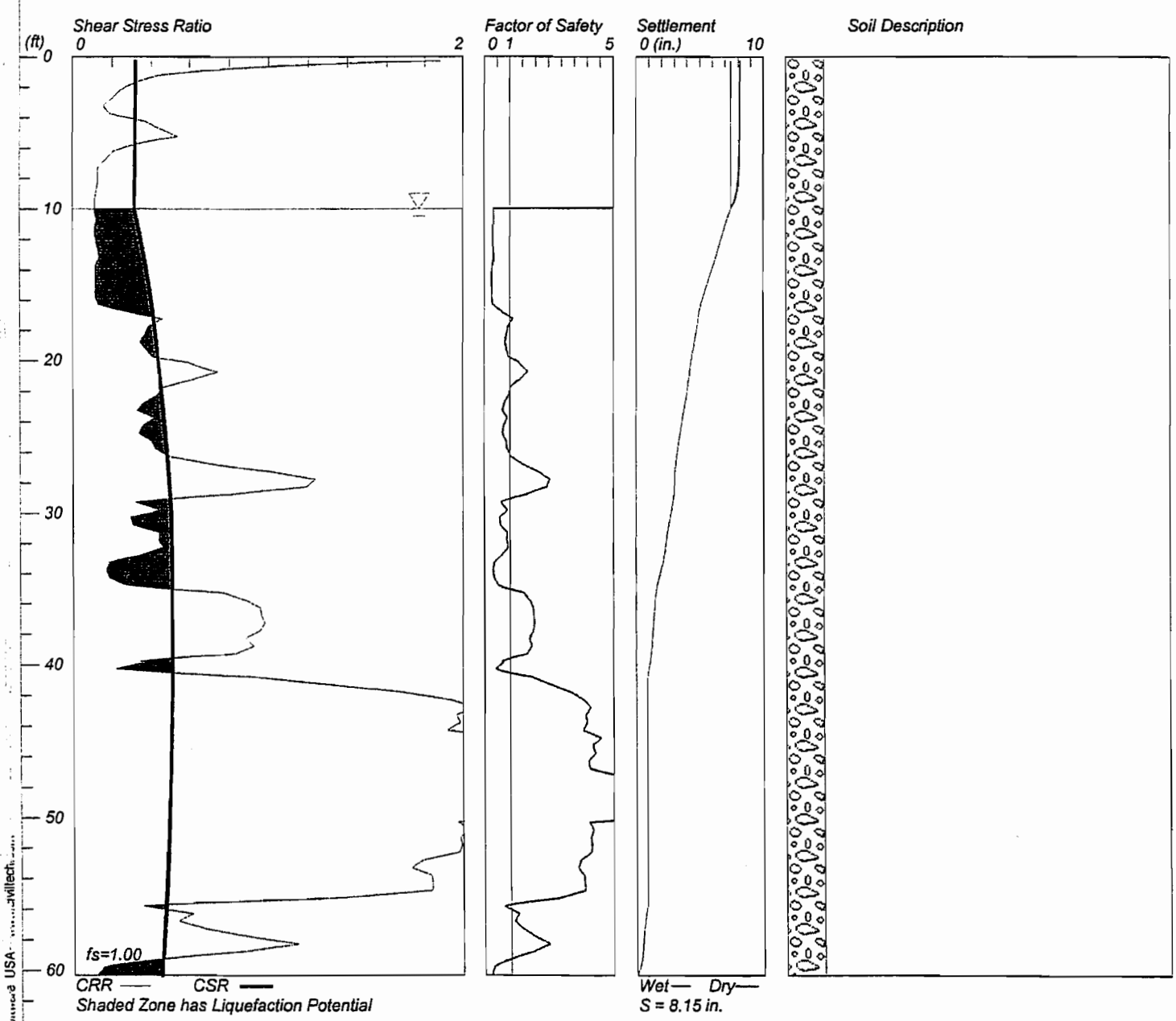


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-3 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

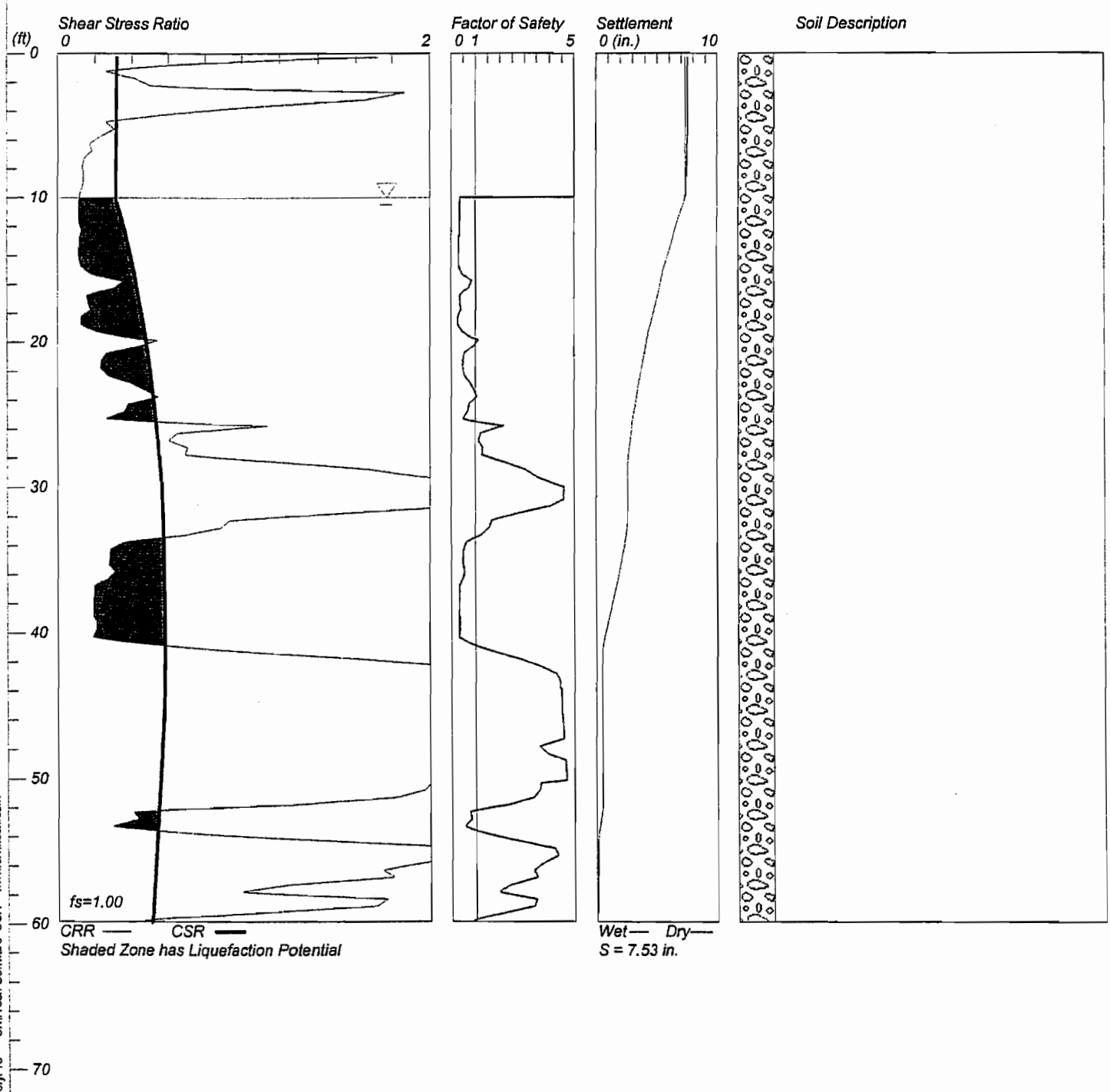


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-4 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

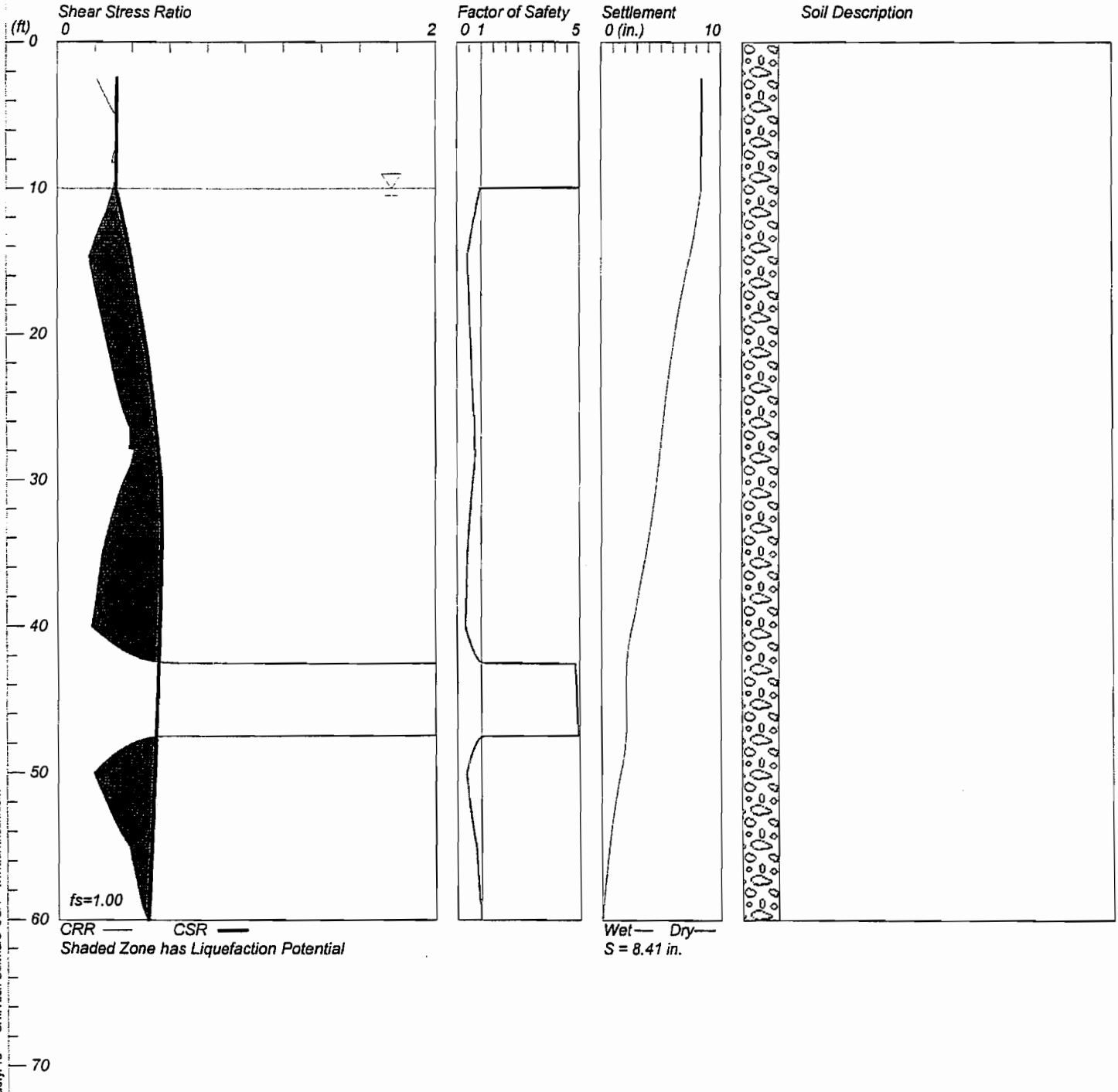


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-1 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

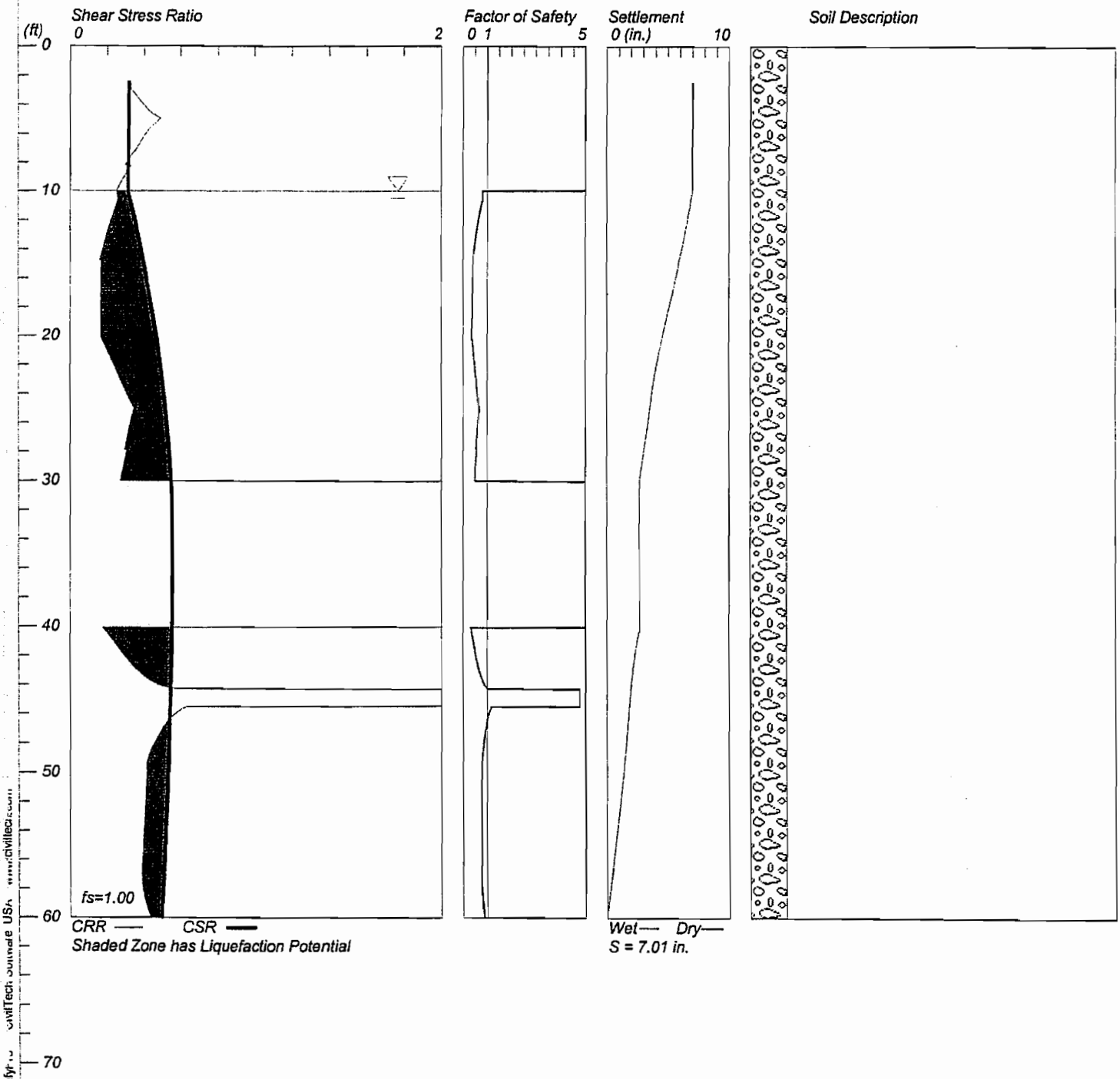


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-2 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

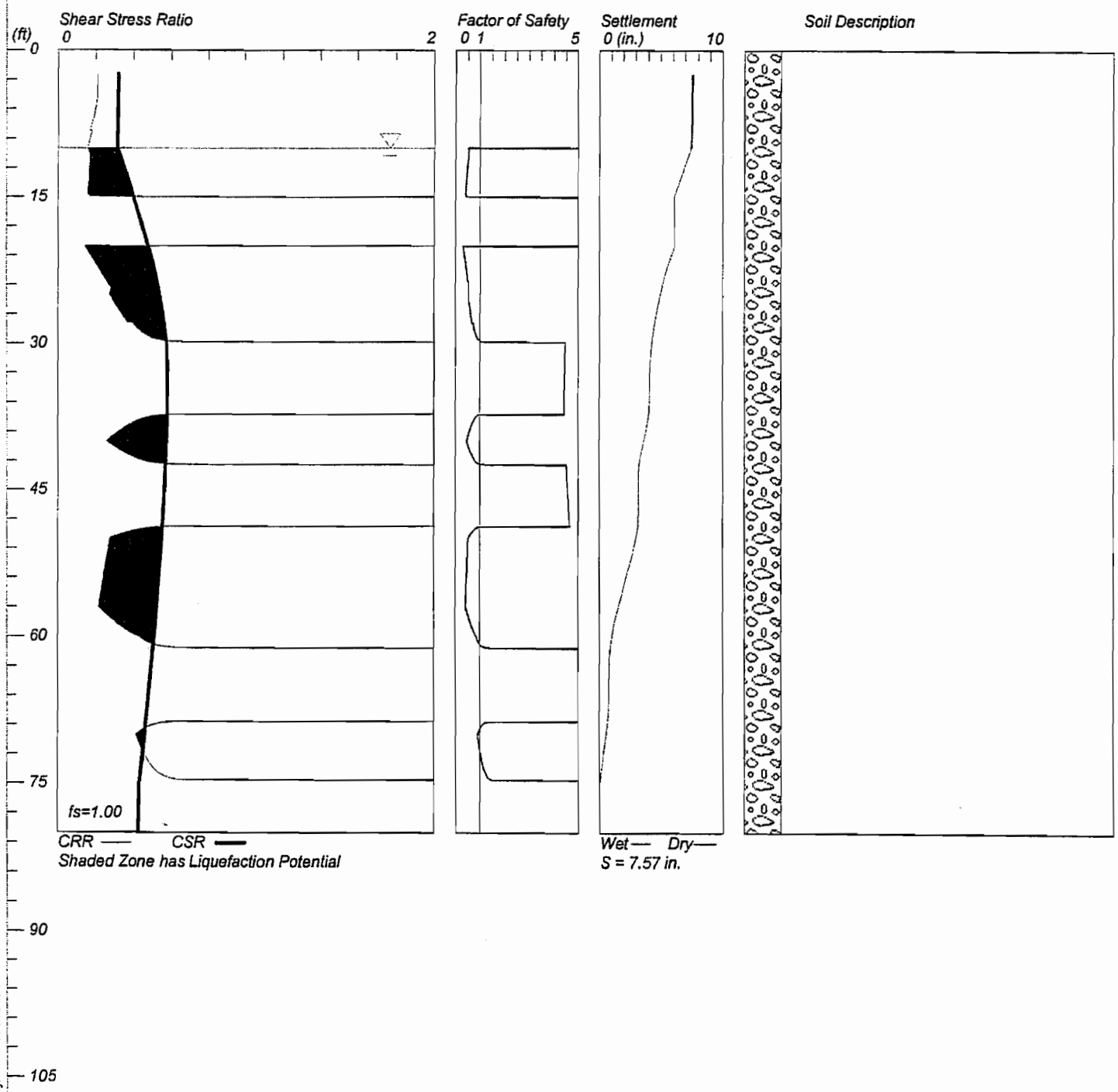


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-3 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g



LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-4 Water Depth=10 ft

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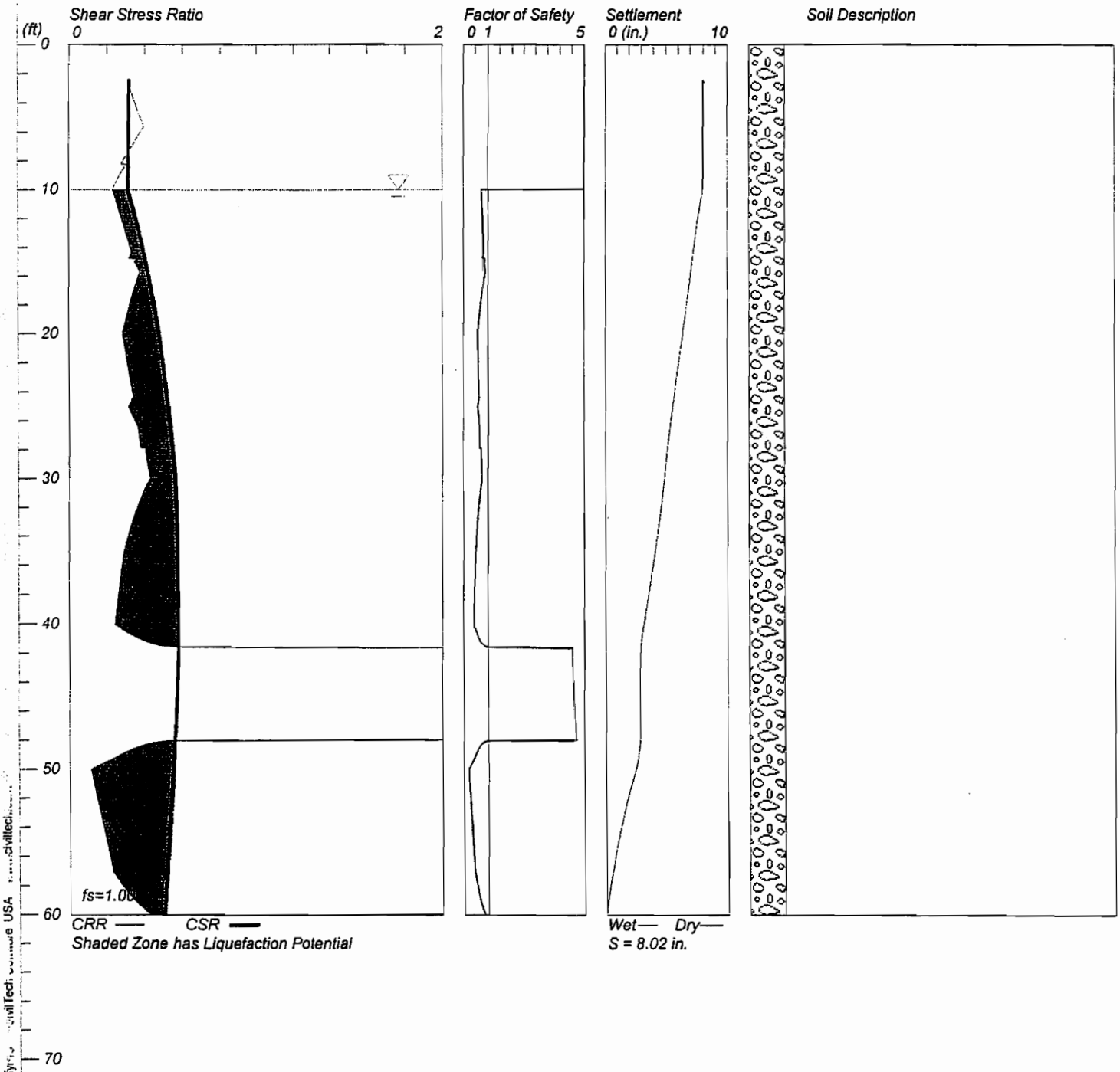
Acceleration=0.49g

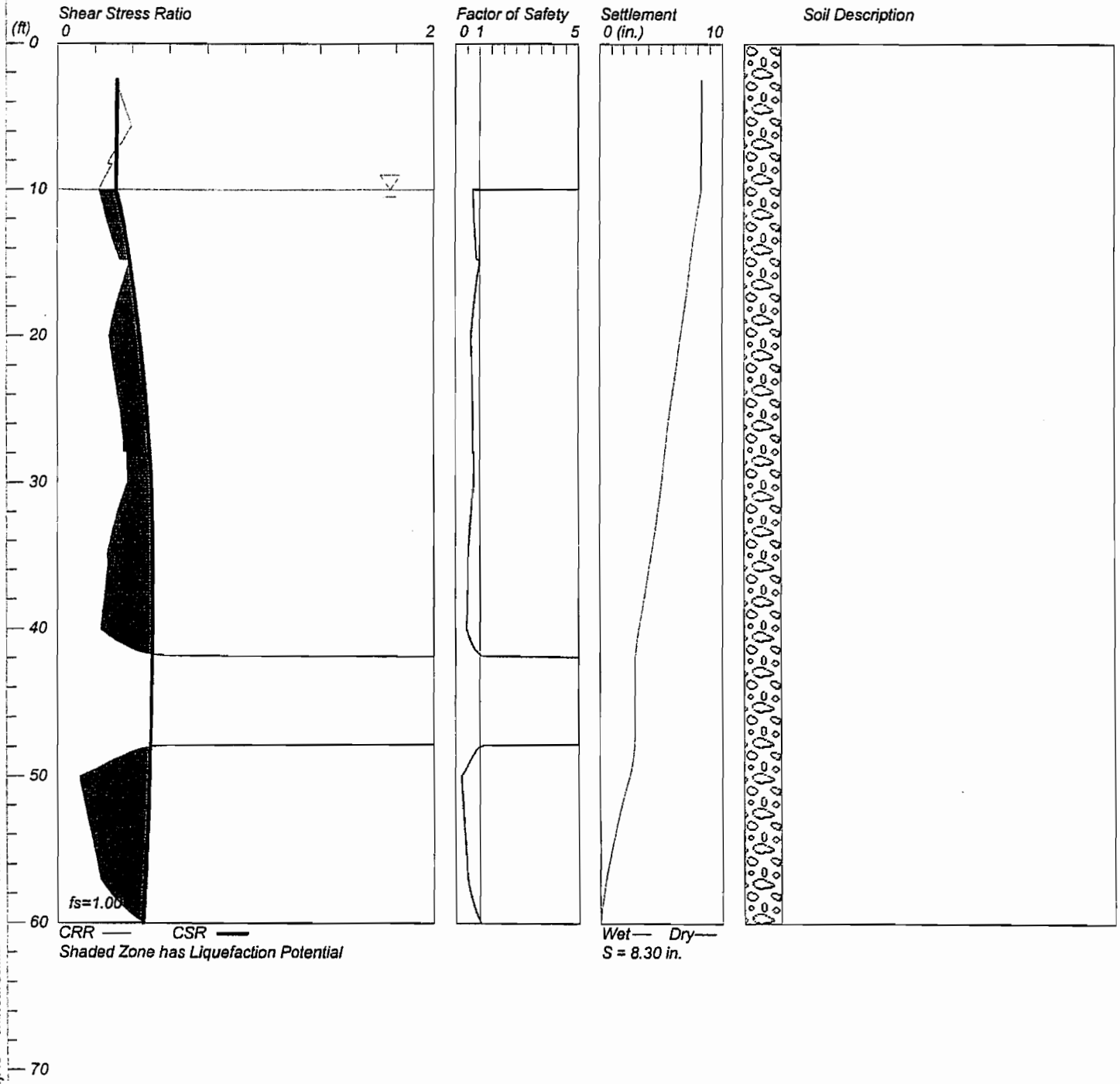


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-5 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

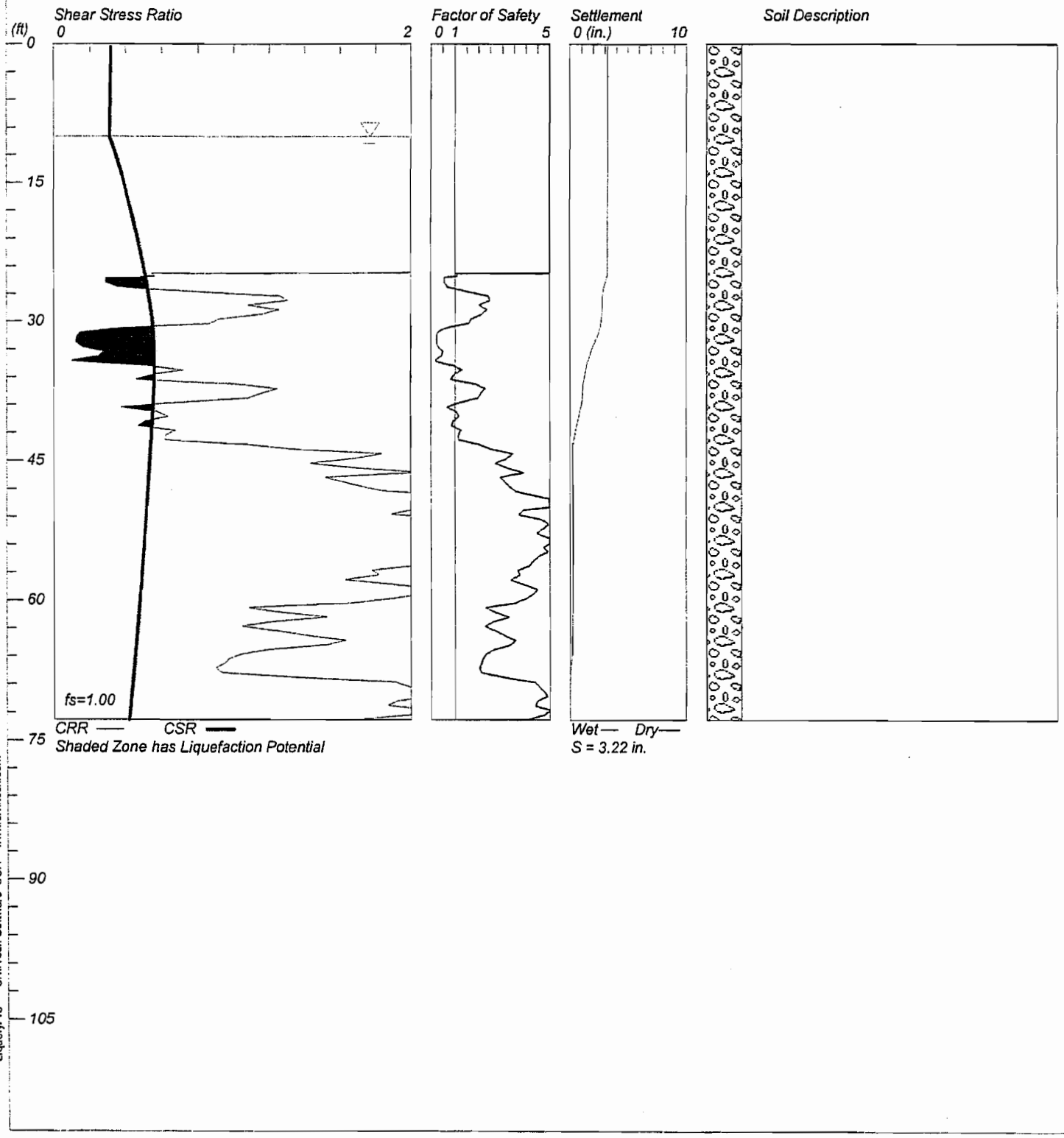


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-1 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g



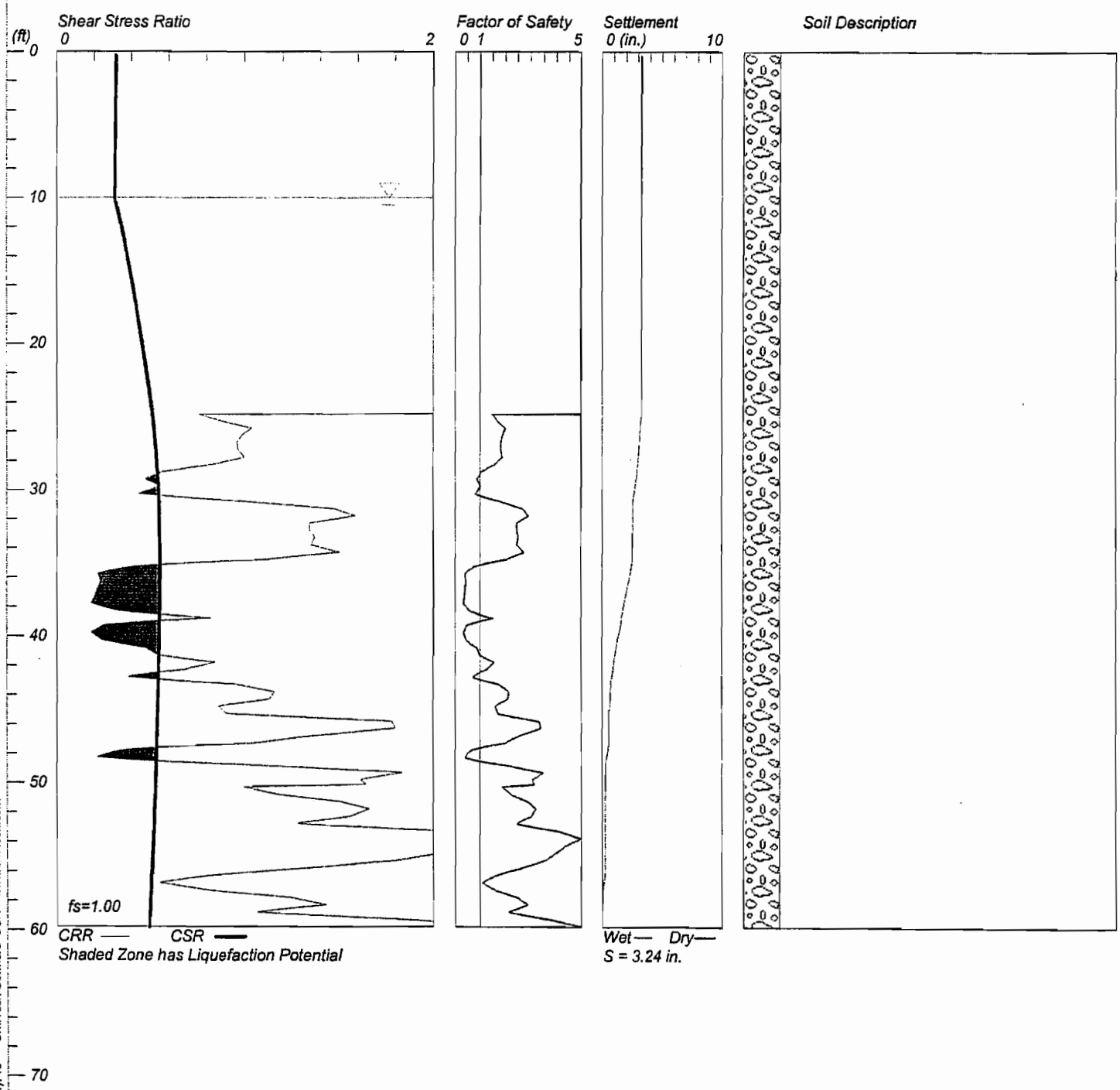
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-2 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

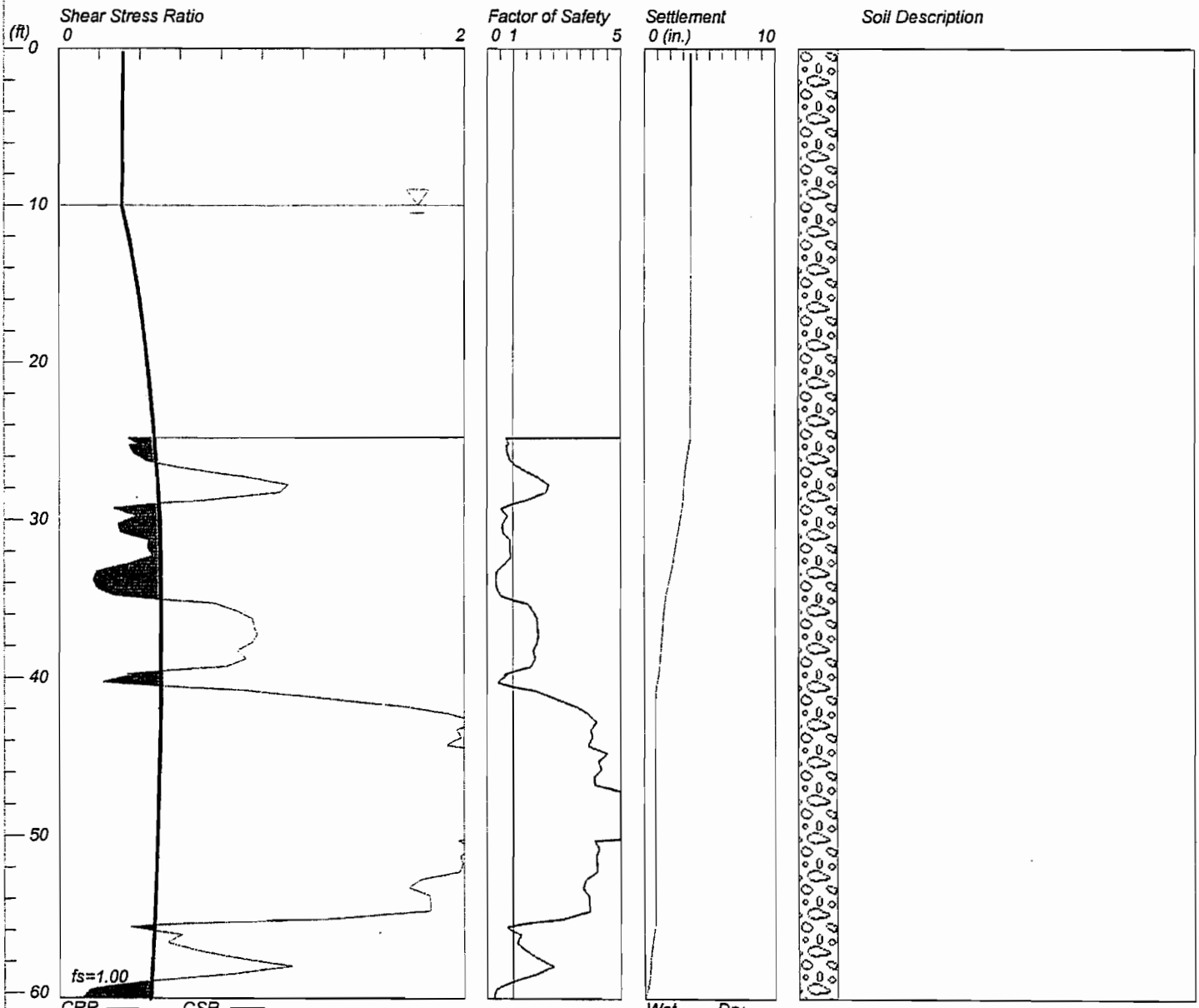


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-3 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

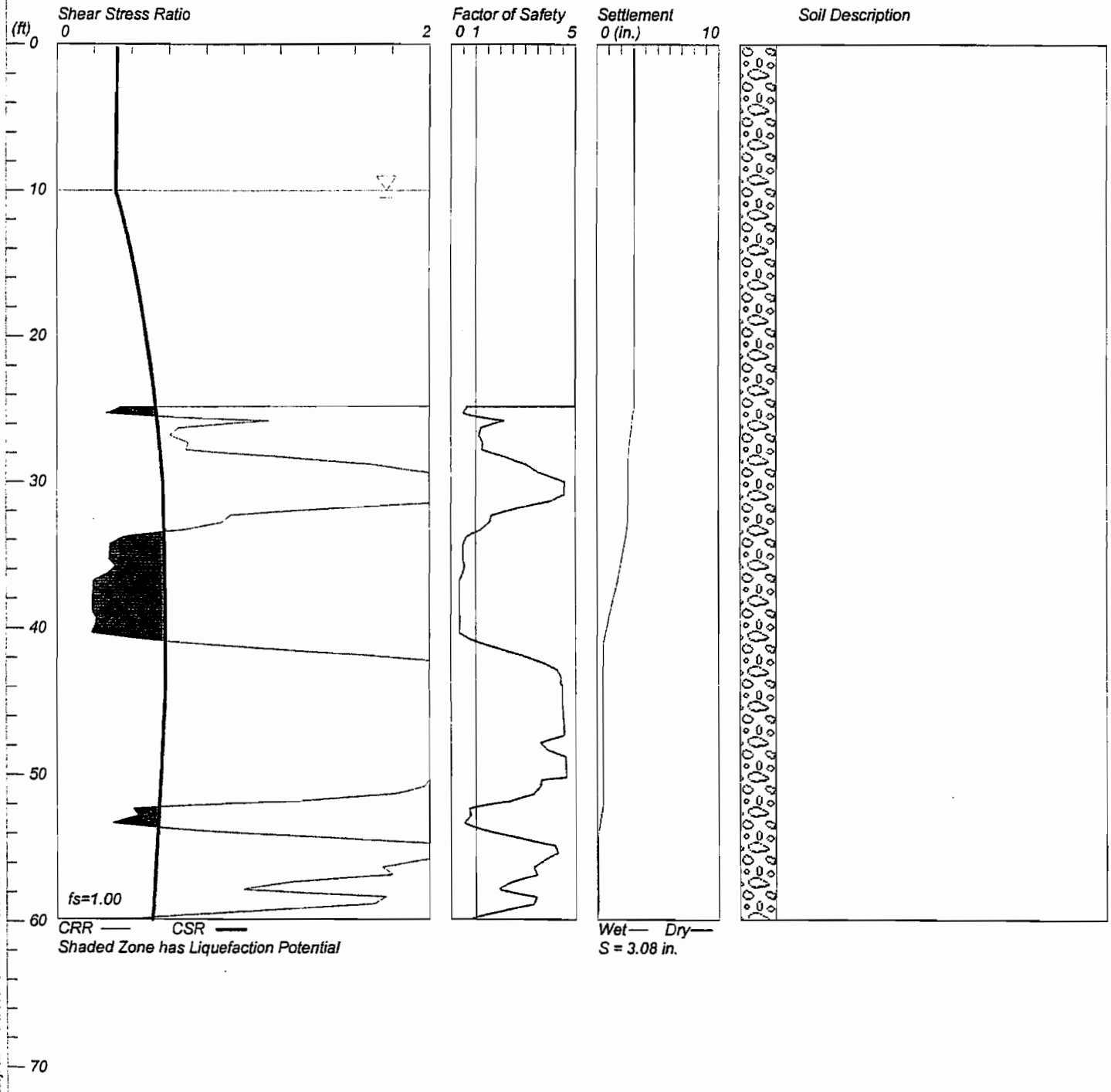


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=CPT-4 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g



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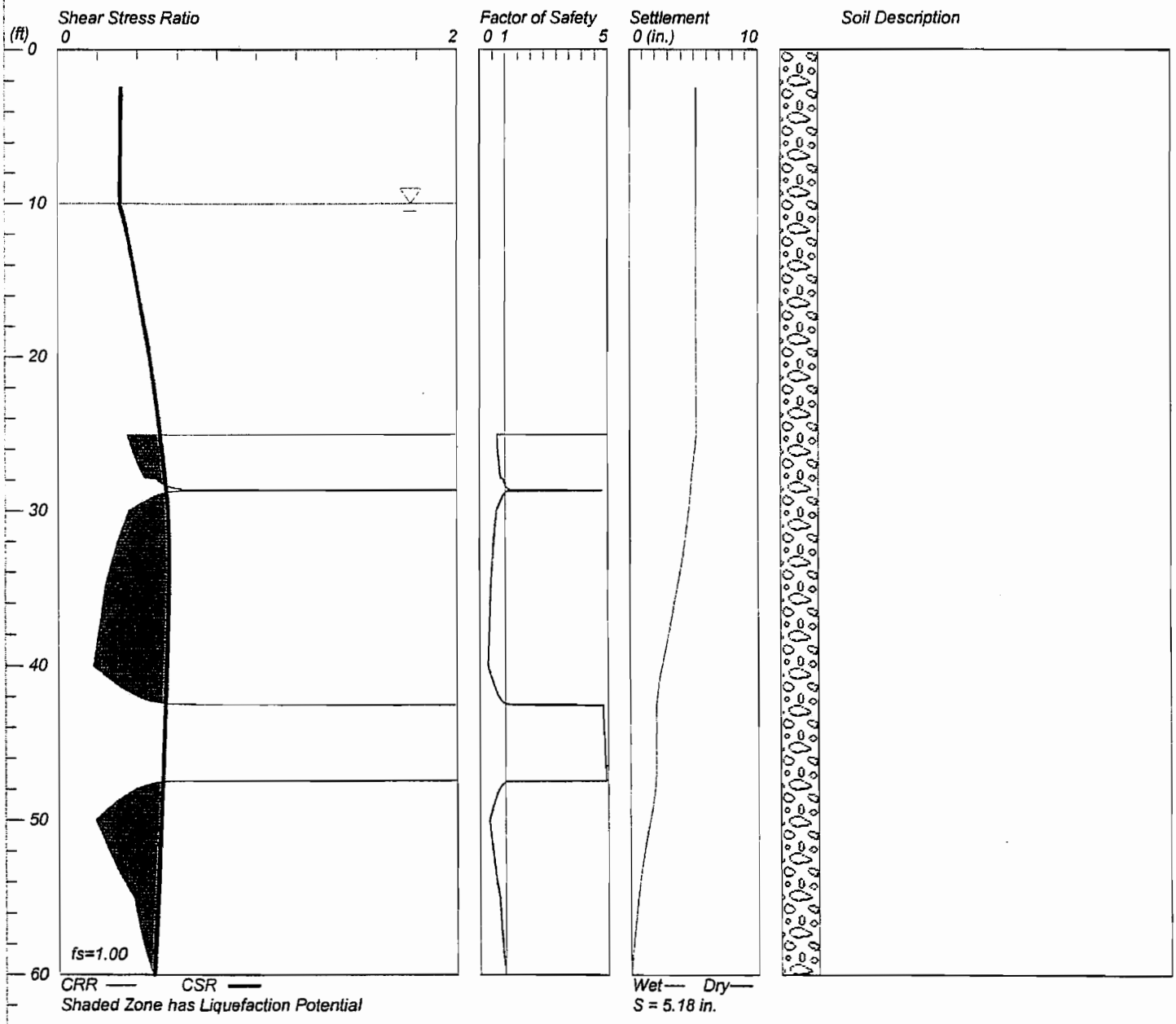
LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-1 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g



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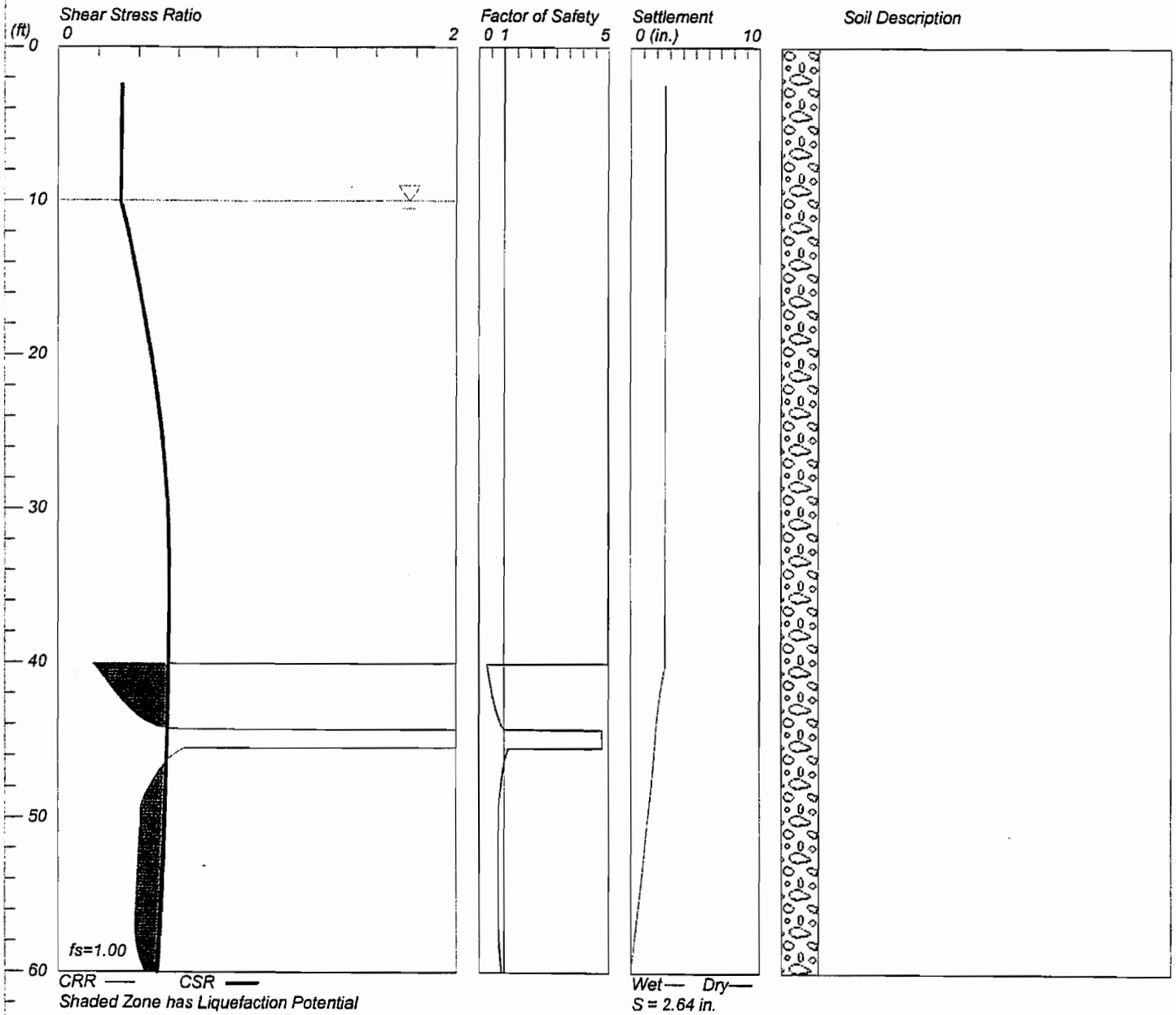
LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-2 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g



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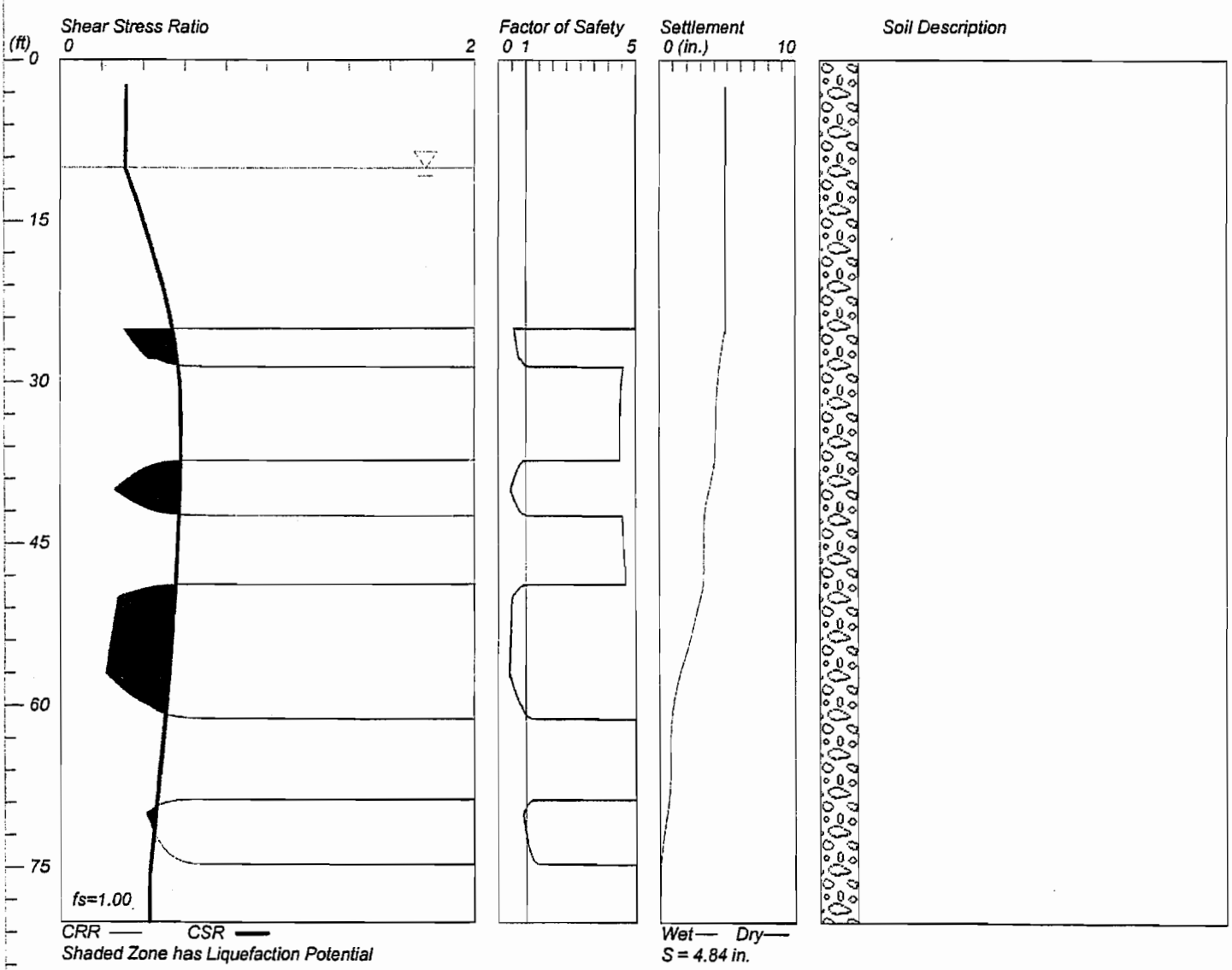
LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-3 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g



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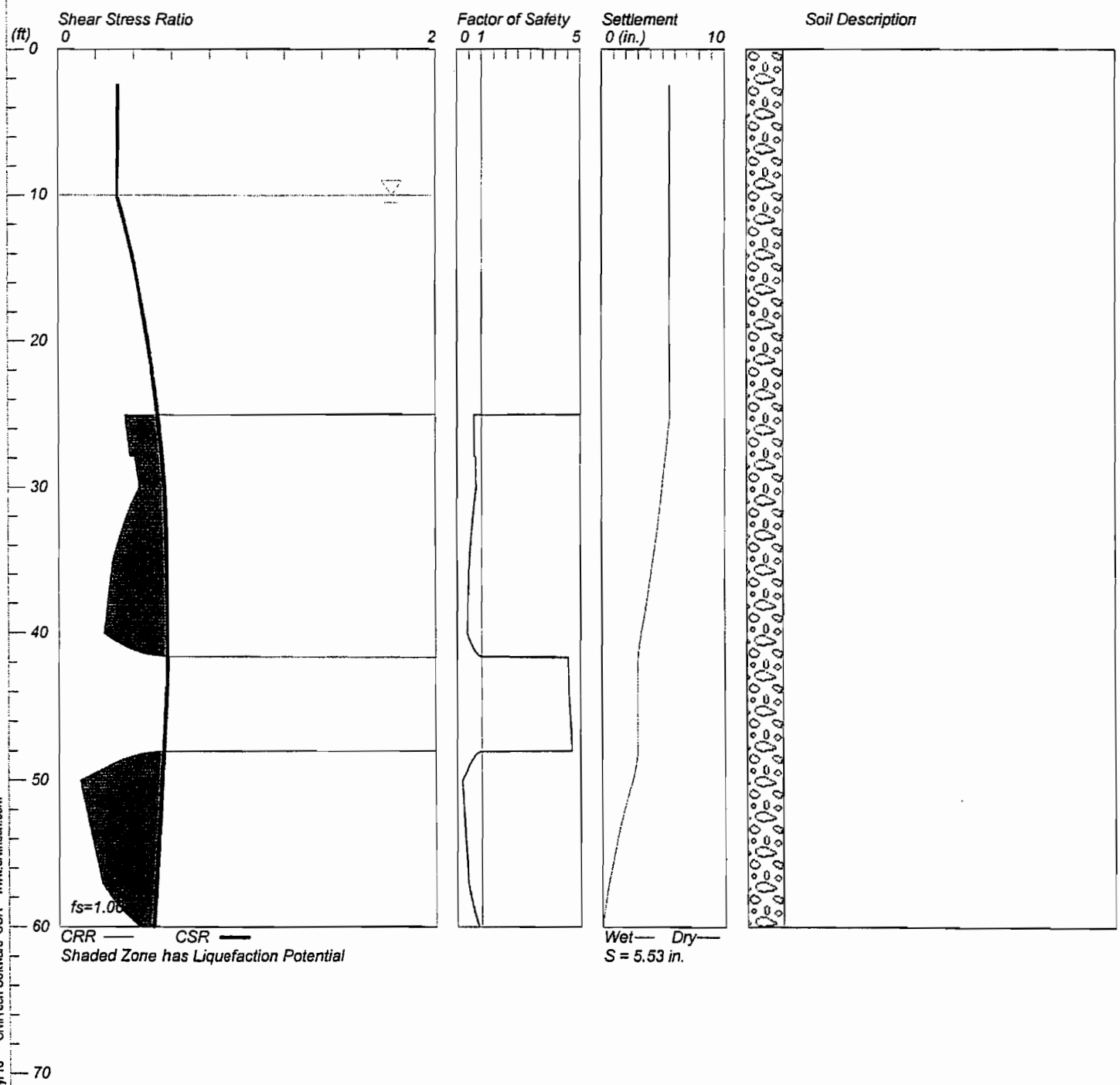
LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-4 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g

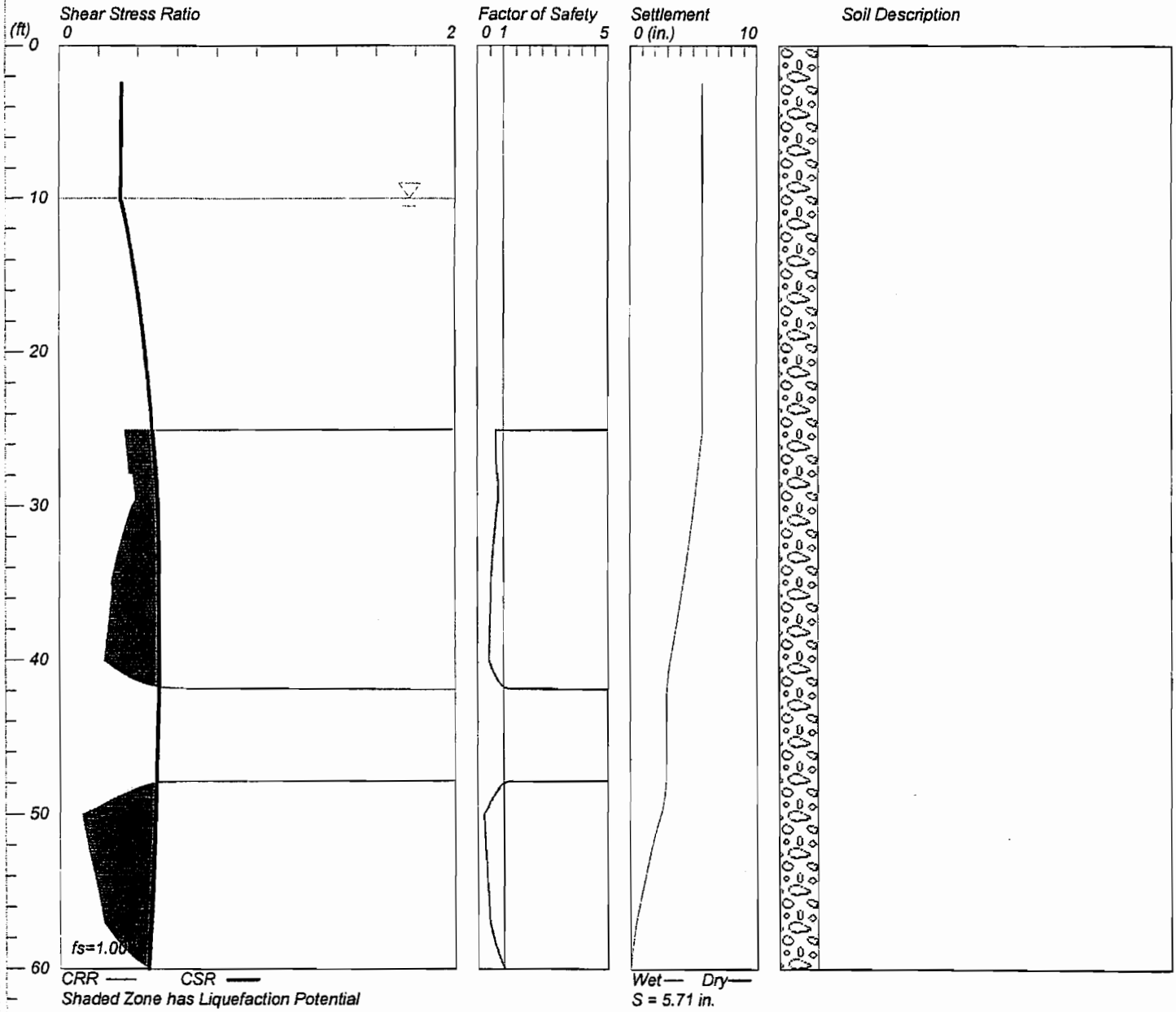


LIQUEFACTION ANALYSIS

LC/Long Beach/ Mixed Use

Hole No.=BH-5 Water Depth=10 ft

Magnitude=6.8
Acceleration=0.49g



LiquefyPro CivilTech Software USA www.civiltech.com

APPENDIX D

GUIDE SPECIFICATIONS FOR DRILLED/DRIVEN PILE INSTALLATION

APPENDIX D

GUIDE SPECIFICATIONS FOR DRILLED/DRIVEN PILE INSTALLATION

It should be the responsibility of the contractor to select proper construction equipment and method to correctly install the piles based on his own interpretation of the information presented in this report. The following recommendations are provided as a guide for preparing plans and specifications and for quality control:

Drilled Piles

- Prior to starting any foundation work, staking should be checked by the project Civil/Structural Engineer. Variations in the alignment from the vertical greater than ¼-inch per foot of length should not be permitted. Any pile installed having a center more than three (3) inches off plan centerline will require structural analysis.

- Some variations in the final pile tip elevations should be expected. The actual tip elevation should be determined by the project geotechnical engineer during excavation based on observation of the actual field conditions.

- Based on the subsurface soil characteristics, caving during excavations may occur within the sandy material layer. Casing, or other methods approved by the project geotechnical consultant, should be used to support the sides of the pile excavation. Casing should be used at the discretion of the contractor. Casing should be advanced as drilling proceeds by drilling with a flight or bucket auger smaller in diameter than the inside of the casing. Occasional hammering may be required to advance the casing with the excavation. Casing should be pulled as the concrete is being poured, while always maintaining a head of concrete inside the casing. Drilling fluids should not be used to support the sides of the excavation without prior approval by the project geotechnical consultants. The contractor should have equipment on-site with sufficient pulling capacity to pull the casing at the proper time. The casing should have outside diameter not less than the specified diameter of the pile.

- In the event that the pile excavation becomes bell-shaped and cannot be advanced due to severe caving, the caved region may be filled with a sand and Portland Cement slurry. Drilling may continue when the slurry has reached its initial set. In this case, it may be prudent to utilize casing or other special methods to facilitate continued drilling after the slurry has set. Sufficient space should be provided in the pier-reinforcing cage during fabrication to allow insertion of a concrete pump pipe or tremie tube for concrete placement.

- The bottoms of the excavations should be cleaned of any loose cuttings before placing concrete. All applicable state and federal OSHA safety regulations must be satisfied during construction.

- The reinforcing bars in the piles should have a minimum concrete cover of 3 inches. Sufficient space should be provided in the reinforcing cage to allow insertion of a concrete tremie tube for concrete placement.

- The reinforcing cage must be carefully placed in uncased holes to prevent gouging of the sides. This will cause loose material to fall into the hole. The cage of reinforcing steel should be placed to the depth required by the plans, and adequately supported at the top.

- Pile shafts spaced closer than five (5) diameters center-to-center shall be drilled and filled with concrete alternatively, allowing at least 12 hours after concrete placement in one shaft before drilling of an adjacent shaft.

- All piles should be concreted immediately after drilling and clean out. Concrete should be placed through a tremie to prevent segregation and unnecessary splashing on the reinforcing steel. The concrete should be directed towards the center of the pile. Free fall of concrete should not exceed three (3) feet.

- The concrete should be flowable, non-segregating concrete with slump near the maximum allowable to obtain satisfactory consolidation without vibration, and to facilitate filling of all voids outside the casing. Concrete should not exhibit rapid slump loss. The recommended slump for uncased drilled piles is 5-in. +/-1 in. When casing is withdrawn, the minimum slump should be 6.0 in and specially designed concrete with retard to prevent arching of concrete during casing withdrawal, or setting of the concrete until after the casing is withdrawn, should be used.

- Casing should be pulled as the concrete is being poured, while always maintaining a head of concrete inside the casing. The bottom of the casing should be maintained not more than five (5) feet nor less than one (1) foot below the top of the concrete during withdrawal and placing operations.

- Place concrete in pile in one continuous operation, Care should be taken to ensure that the concrete in the hole is dense and homogeneous. After the hole has been filled with concrete, the top 10 feet or the length of the reinforcing, whichever is greater should be vibrated.

- In the event that any pile excavation becomes bell-shaped and cannot be advanced due to severe caving, the caved region may be filled with a sand and Portland Cement slurry.

Drilling may continue when the slurry has hardened. In this case, it may be prudent to utilize casing or other special methods to facilitate continued drilling after the slurry has set.

- Drilled pile installation shall be performed under continuous observation by the project geotechnical consultant to confirm that the subsurface soils are similar to the soils encountered during our field investigation, which have formed the basis of our pier design recommendations. Further, the soils consultant should confirm that the dimensions of the installed piers are at least as large as those indicated on the foundation plan, and that pier installation has been performed as specified in this report. The contractor shall provide access and necessary facilities, including droplights, at his expense, to accommodate pier observations.

- Drilled pile installation shall be performed such that compliance with all safety rules and requirements is achieved. Drilling equipment, casing, reinforcement, and other items required for installation shall be kept at a safe distance from all overhead power lines and utilities.

Driven Pile

- Selection of a suitable pile-driving hammer depends on the overall soil-pile hammer-cushion system. We estimate that a pile-driving hammer with rated maximum energies between 50,000 and 60,000 foot-pounds of energy per blow will be appropriate. Pile driving in the sandy layers will encounter difficulty. Wave equation analyses should be performed to verify the adequacy of the proposed driving equipment.

- A pile driving indicator program is recommended to be carried out to finalize pile length, potential pre-drilling methods, and correlate blowcount to the results of the wave-equation analyses.

- Predrill can be used through localized hard layers to help improve driveability into bearing strata and to help to reduce downdrag on piles driven through fill. Maximum auger size should not exceed 85 percent of minimum pile dimension for predrill hard layers. Predrill in bearing strata is not recommended. Depths of predrill can be determined based on the observation made during the indicator pile program.

APPENDIX E

GUIDE SPECIFICATIONS FOR INSTALLATION AND ACCEPTANCE OF TIEBACK ANCHORS

APPENDIX E

GUIDE SPECIFICATIONS FOR INSTALLATION AND ACCEPTANCE OF TIEBACK ANCHORS

Installation

1. Tie-back installation shall be performed during continuous observation by Converse Consultants (Converse) to confirm that the recommended earth materials are penetrated, that the dimensions of the installed anchors are at least as large as that indicated on the shoring plan, and that anchor installation has been performed as specified. The Contractor shall provide access and necessary facilities, including lighting, at their expense, to accommodate observations.

2. All anchors shall be installed at the specified locations, to the required depth, and at the specified angle of inclination. A tolerance of 3° will be permitted on the required angle of inclination.

3. After drilling, all holes shall be cleaned of loose soils. Concrete shall be placed by pumping from the tip of the anchor to the active wedge. Concrete placement shall begin within four hours after completion of drilling. The portion of the anchor within the active wedge shall be backfilled with sand-cement slurry after the anchor has been tested as specified below. However, if excessive caving occurs, the active wedge portion of the excavation can be filled with slurry as the casing is pulled. A zone of soft soil shall (in this case) be placed between the anchor and slurry (before testing).

4. If a hollow-stem auger or casing is used due to caving, concrete shall be placed by pumping as the auger or casing is withdrawn while always maintaining a head of concrete inside the casing or auger.

5. Concrete placement shall be continuous without interruption, and at such a rate that fresh concrete will not be deposited on concrete hardened sufficiently to form seams and planes of weakness.

6. Any anchor deemed by the Owner or Converse to be defective shall be replaced with substitute anchor(s) as directed by the Owner or Shoring Designer. The cost of installation of such substitute anchors shall be borne by the Contractor. Costs associated with analysis and design of substitute anchor(s) shall also be borne by the Contractor.

Acceptance Criteria

1. Actual capacities of anchors shall be determined by testing designated Test Anchors and all Production Anchors. Testing of anchors will enable evaluation of the applicability of design values for the chosen method of tieback construction.

2. All anchors shall be check-tested to at least 150% of the designed working load in accordance with the following procedures:

   a. Test load anchors to 150% of the design-working load, incrementally noting loads, tendon extensions and soldier pile deflections. Hold load for 15 minutes. After pulling slack, the anchor movement shall not exceed 0.10 inch during the 15-minute load period. If the deflection is acceptable, reduce load to 100% of the design load and lock off.

   b. Where an anchor shows excessive movement for additional 15-minute intervals, the load should be reduced until the rate of movement is 0.10 inch per 15 minutes or less. The load at which acceptable movement is attained should be divided by 1.5 to establish the working load of the anchor and additional measures taken to carry the required load.

3. Converse shall designate at least 5% of all proposed anchors as 200% Test Anchors. Additional anchor steel reinforcement will likely be required for the 200 percent load test anchors, and should be appropriately considered prior to anchor installation. Half of the 200% Test Anchors shall be tested for 30 minutes. The remaining Test Anchors shall be tested for a 24-hour period. Test anchors shall be tested in the following manner:

   a. For the 30-minute test anchors, incrementally load the anchors to 200% of the design-working load noting loads, tendon/bar extensions and soldier pile deflections. Hold load for 30 minutes. Anchor movement shall not exceed 0.3 inch during the 30-minute load period. If the deflection is acceptable, reduce load to design load and lock off; otherwise, reduce the test load by 50% and repeat this step.

   b. For 24-hour test anchors, incrementally load to 200% and hold for 24 hours; check load after 24 hours. If a pre-stress loss of 8% or less is recorded, restore load to 100% of working load and lock off. If loss of pre-stress exceeds 8%, restore load to 150% of working load and hold for an additional 24 hours. Check load after second 24-hour hold and, if loss of pre-stress is less than 8%; restore to 100% and lock off as before.

Where an anchor shows a continuous loss of pre-stress during a subsequent 24-hour period, the test load shall continue to be reduced by 50% until loss of pre-stress is negligible. Then the test load shall be divided by 1.5 to establish the working load of that anchor and additional measures taken to carry the required shoring load.

5. Any anchor pulled more than 12 inches shall not be used.

6. Immediately after testing, the active wedge portion of tieback excavations should be filled with slurry.