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Geotechnical Engineering and Percolation Testing Report

Bob Cole Conservatory of Music
California State University Long Beach
Long Beach, California

Prepared for:
California State University Long Beach
1250 Bellflower Boulevard, MS #5805
Long Beach, CA 90840

June 7, 2011
Project No.: 110403.1

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June 7, 2011
Project No. 110403.1

Mr. Enrique Robles
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1250 Bellflower Boulevard, MS #5805
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Subject: Geotechnical Engineering and Percolation Testing Report
Bob Cole Conservatory of Music
California State University Long Beach
Long Beach, California

Dear Mr. Robles:

In accordance with your request and authorization, we performed a geotechnical investigation for the above-referenced project in Long Beach, California. Our findings and recommendations are presented in this report. The purpose of this investigation has been to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for the design and construction of the proposed project.

Please note that the recommendations presented within the report are based on assumptions stated herein. Should conditions encountered during development differ from those assumed in our analyses, or should the proposed development change, our recommendations may need to be modified accordingly.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
QUALITY ASSURANCE INSPECTIONS

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1. INTRODUCTION

This report presents the results of our geotechnical evaluation and percolation testing performed for the Bob Cole Conservatory of Music plaza renovation on the California State University campus in the city of Long Beach, California (Figure 1). The purpose of this study has been to evaluate the subsurface conditions at the site and to provide geotechnical recommendations relative to design and construction of the proposed project.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject project site is located on the California State University Long campus in Long Beach, California. The site is located within the University Music Center bounded by Carpenter Performing Arts Center to the north, athletic fields to the south, parking lot 12 to the east, and Walter Pyramid to the west.

The proposed project consists of the renovation of the existing University Music Building. The renovation includes an entry plaza with sidewalks, pedestrian ramps, retaining walls and a single-story music pavilion. The approximate limits of the proposed project are shown on Figure 2, Boring Location Map. Anticipated structural loads for foundation design were not available at the time of this report and preliminary foundation plans have not been made available for our review. Upon availability, the actual planned bearing pressures with associated footing sizes should be reviewed by QAI to ensure conformance with foundation recommendations and geotechnical engineering assumptions used in this report. For the recommendations provided in this report, we assumed that the proposed improvements will be supported on shallow spread footings with maximum column loads of 25 kips and maximum perimeter foundation loads of 4 kips per lineal foot.

3. SCOPE OF SERVICES

Our scope of services for this project consisted of the following:

- We performed a subsurface evaluation, including the excavation, logging, and sampling of five exploratory borings. The borings were advanced to depths of approximately 5 to 26½ feet below the existing grades. Samples of earth materials were obtained from the borings and transported to our in-house laboratory for observation and testing.

- We performed percolation testing in three locations to evaluate the feasibility of using a storm water infiltration system at the site surface.

- We performed laboratory testing on selected samples of earth materials in order to evaluate the geotechnical engineering properties of the on-site soils. Laboratory tests included in-situ moisture content and dry density, sieve, direct shear, Atterberg limits, expansion index, corrosivity index, and consolidation testing.

- We compiled and performed engineering analyses of the data collected from our subsurface evaluation, percolation testing, and laboratory testing. Specifically, our analyses included the following:

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials;
- Evaluation of geologic hazards, including site seismicity, liquefaction and seismic settlement potential;
- Evaluation of seismic design parameters in accordance with the 2010 California Building Code for use in structural design;
- Evaluation of current and historical groundwater conditions at the site and potential impact on design and construction;
- Evaluation of project feasibility and suitability of on-site soils for foundation support;
- Preparation of recommendations for site grading and subgrade preparation for sidewalks and foundations;
- Evaluation of the potential for on-site earth materials to corrode buried metal and concrete;
- Evaluation of expansion potential and recommendations to mitigate the impact of expansive soil, if necessary;
- Evaluation of percolation rates (minutes/inch); and
- Evaluation of foundation design parameters including allowable bearing capacity for shallow foundations, estimated settlement, lateral earth pressures, and lateral resistance.

- We prepared this report that presents the work performed, the data collected, and geotechnical recommendations for the design and construction of the proposed parking lots.

4. FIELD EXPLORATION AND LABORATORY TESTING

The subsurface conditions were evaluated by advancing five exploratory borings at various locations across the site. The approximate locations of the exploratory borings are shown in Figure 2. The boring logs describing the subsurface materials encountered are presented in Appendix A.

The borings were advanced to depths of approximately 5 to 26½ feet below the existing grades using hand tools and a rubber-track-mounted limited access drill rig with an 8-inch-diameter hollow stem auger. Driven samples of the soil were obtained using a standard penetration test (SPT) sampler and a modified California split spoon sampler. The samplers were driven using a 140-pound hammer falling approximately 30 inches on a rope-and-cathead pulley system. The blow counts were recorded, and the materials encountered in the borings were logged by our staff engineer in the field. The recorded blow counts are included on the boring logs in Appendix A. Upon completion of drilling, the borings were backfilled by the drilling subcontractor using cuttings derived from the excavations.

Percolation testing was performed at three locations. The approximate locations of the percolation tests are shown in Figure 2. The results from the percolation testing are included in Section 6.6. Upon completion of excavating and percolation testing, the borings were backfilled using the cuttings.

Laboratory tests were performed on selected samples obtained from the borings to aid in the soil classification and to evaluate the engineering properties of the soils. The following tests were performed in general accordance with ASTM standards:

- Laboratory determination of in-situ moisture and dry density;
- Atterberg limits;
- Sieve analysis;
- Direct shear;
- Consolidation
- Expansion index; and
- Corrosivity.

The moisture content and density data are presented on the boring logs in Appendix A and are summarized in Appendix B, Table B-1. Details of the laboratory testing program and all of the laboratory test results are included in Appendix B.

5. ENGINEERING SEISMOLOGY AND DESIGN

The southern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are generally capable of generating ground shaking of engineering significance to the proposed construction. The project area is located in the general proximity of several active and potentially active faults. Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years).

5.1. Active Faulting

Official Maps of Earthquake Fault Zones were reviewed to evaluate the location of the project site relative to known active fault zones. Alquist-Priolo Earthquake Fault Zones (known as Special Studies Zones prior to 1994) have been established in accordance with the Alquist-Priolo Special Studies Zones Act of 1972, as amended. The act directs the State Geologist to delineate the regulatory zones that encompass the surface traces of active faults that exhibit a potential for future surface fault rupture. The purpose of the act is to regulate development near active faults in order to mitigate the hazard of surface fault rupture.

The project site is located on the Los Alamitos Quadrangle Special Studies Zones map. This map indicates that the site is not located within any mapped Earthquake Fault Zone. The closest mapped Earthquake Fault Zone to the site is located approximately 1.3 miles southwest of the site. Accoridng to the CSU Seismic Requirements Manual (2001), there are no active faults that traverse the CSU Long Beach campus.

Neither our field observations nor our review of published geologic literature indicated the presence of any active faults passing through the site or projecting toward the site. The United States Geological Survey queries the presence of a fault northwest-trending fault near the northeastern boundary of the project site. Based on our evaluation of the data, it is our opinion that the likelihood of fault rupture occurring at the site during the design life of the proposed improvements is low.

5.2. Historical Seismicity

In the absence of fault rupture, the greatest seismic hazard likely to affect the site is seismic shaking due to one or more earthquakes generated on nearby or distant active faults. The approximate locations of major faults in the region and their geographic relationship to the site are shown on Figure 4, Fault Location Map. The epicentral locations of selected historic earthquakes in southern California have been plotted by the California Division of Mines and Geology (Toppozada and others, 2000). A reproduction of this map is presented as Figure 5, Historical Seismicity, 1800-1999.

5.3. Geotechnical Parameters for Earthquake Design

The seismic design of the project may be performed using criteria presented in the CSU Seismic Requirement Manual dated January 6, 2011. The seismic design parameters for BSE-2 (MCE) hazard level are listed below in Table 1. The seismic design parameters for BSE-1 (DE) hazard level are listed below in Table 2.

Table 1
CSU Seismic Design Parameters for BSE-2 Hazard Level

CSU Seismic Design Factor	Value
0.2-Second CSU Long Beach Spectral Acceleration Parameter, $S_{BSE\text{-}2\text{-}.2S}$	1.66
1-Second CSU Long Beach Spectral Acceleration Parameter, $S_{BSE\text{-}2\text{-}1S}$	0.62
Site Class	D
Short Period Site Coefficient, F_a	0.88
1-Second Period Site Coefficient, F_v	1.39
Short Period Adjusted Spectral Response Acceleration Parameter, $S_{BSE\text{-}2S}$	1.46
1-Second Period Adjusted Spectral Response Acceleration Parameter, $S_{BSE\text{-}21}$	0.86

Table 2
CSU Seismic Design Parameters for BSE-1 Hazard Level

CSU Seismic Design Factor	Value
0.2-Second CSU Long Beach Spectral Acceleration Parameter, $S_{BSE\text{-}1\text{-}.2S}$	1.10
1-Second CSU Long Beach Spectral Acceleration Parameter, $S_{BSE\text{-}1\text{-}1S}$	0.42
Site Class	D
Short Period Site Coefficient, F_a	1.06
1-Second Period Site Coefficient, F_v	1.58
Short Period Adjusted Spectral Response Acceleration Parameter, $S_{BSE\text{-}1S}$	1.17
1-Second Period Adjusted Spectral Response Acceleration Parameter, $S_{BSE\text{-}11}$	0.66

5.1. Liquefaction and Seismic Settlement Potential

Liquefaction occurs when the pore pressures generated within a soil mass approach the effective overburden pressure. Liquefaction of soils may be caused by cyclic loading such as those imposed by ground shaking during earthquakes. The increase in pore pressure results in a loss of strength, and the soil then can undergo both horizontal and vertical movements, depending on the site conditions. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity. Liquefaction is generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion.

The site is located within a state-designated Seismic Hazard Zone for liquefaction potential. Groundwater was encountered at approximately 12 feet below the existing surface during our subsurface exploration. The reported historical high depth to groundwater is less than 10 feet below the surface of the site. The soil encountered during our investigation consists of medium stiff silts and clays. Preliminary screening of the on-site fine grained material indicates the soil to be non-liquefiable.

6. DESIGN RECOMMENDATIONS

Based on the results of the field exploration and engineering analyses, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and implemented during construction. The proposed improvements may be supported on shallow foundations consisting of isolated and continuous spread footings.

As mentioned earlier, the details of the project are not finalized at this time. In the absence of specific information, the recommendations provided in this report are general in nature. We assumed that the finished floor elevation of the proposed renovations will closely match the existing surface elevations. We recommend that we review the project structural and grading plans prior to finalizing design to ensure that the recommendations in this report are properly incorporated.

Our geotechnical engineering analyses performed for this report were based on the earth materials encountered during the subsurface exploration. If the design substantially changes, our geotechnical engineering recommendations would be subject to revision based on the additional evaluation of the changes. The following sections present our conclusions and recommendations pertaining to the engineering design for this project.

6.1. Site Preparation

After demolition and prior to grading, deleterious, organic, and oversized materials greater than 6 inches in maximum dimension should be removed from the construction area and disposed outside of the construction limits. Based upon the observed moisture condition of the on-site soil, particularly within the grass areas, compaction of these materials is not feasible without some remediation consisting of drying or otherwise stabilizing the materials. Our recommendations for subgrade preparation accommodate these conditions and are presented below for each structural component of the project.

<u>Pavilion Foundations and Floor Slab/Retaining Wall Foundations:</u> Pavilion foundations, retaining wall foundations and the pavilion floor slab should bear on at least 1½ feet of aggregate base materials underlain by geogrid. Aggregate base should consist of either Class 2 Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB). Geogrid should consist of Tensar BX-1200 or equivalent. Geogrid should be placed at the bottoms of excavations that expose relatively undisturbed site soils at the bottoms. The excavations for footings and the floor slab should extend laterally at least 1 foot beyond the edges of footings and the floor slab. Upon placement of the geogrid, aggregate base should be in placed in one lift and compacted at the surface to achieve 90 percent of the maximum dry density as determined by ASTM D 1557. A representative of QAI should observe the bottoms of excavations prior to placement of any geogrid or aggregate base materials, and a representative of QAI should be present during the placement and compaction of aggregate base materials.

<u>Exterior Flatwork/Sidewalks:</u> Exterior flatwork and sidewalks that are not subject to vehicle traffic should bear on at least 1 foot of CAB or CMB underlain by geogrid (Tensar BX-1200 or equivalent). Geogrid should be placed at the bottoms of excavations that expose relatively undisturbed site soils at the bottoms. Upon placement of the geogrid, aggregate base should be in placed in one lift and compacted at the surface to achieve 90 percent of the maximum dry density as determined by ASTM D 1557. A representative of QAI should observe the bottoms of excavations prior to placement of any geogrid or aggregate base materials, and a representative of QAI should be present during the placement and compaction of aggregate base materials.

In general, the bottoms of excavations should be kept in an undisturbed state to the maximum extent possible. If it is necessary to operate equipment within excavations, we note that the use of track-mounted excavation equipment will be required as rubber-tired vehicles will likely sink into the subgrade and cause unwanted disturbance of the excavated surface. As previously mentioned, all exposed bottoms should be observed by a representative of QAI prior to placement of any materials so that we can evaluate the suitability of the exposed soils. If any unsuitable soils are encountered, subgrade stabilization will be necessary. Procedures for subgrade stabilization are discussed in section 6.1.1. After the excavated bottoms have been approved by an engineer from QAI, they should be backfilled with engineered fill to the elevations necessary to achieve the proposed grades.

Most of the soils encountered during our subsurface exploration should be suitable for use as engineered fill; however, we note that the moisture condition of the on-site soil is well above optimum moisture and will require significant drying to achieve near optimum moisture conditions. Engineered fill should be placed in loose lifts approximately 6- to 8-inches thick, moisture conditioned to approximately 0 to 2 percent above the optimum moisture content of the soil, and compacted to at least 90 percent as determined by ASTM D 1557. The placement of engineered fill should be continuously observed during placement and tested for compaction by a representative of QAI.

In the event that fill soils are imported to meet grading plan requirements, such fill soils should be sampled, tested, and evaluated by QAI prior to transporting to the site. Imported materials should be non-plastic, contain no particles greater than 6 inches in maximum dimension, and have an expansion index no more than 20.

6.2. Subgrade Stabilization

If fill placement is required at the project, subgrade stabilization will likely be required to create a firm platform upon which fill can be placed. Subgrade stabilization can consist of removal of approximately 1 foot of subgrade soils, placement of geogrid material (Tensar BX-1200 or equivalent) at the bottom of the relatively undisturbed excavation prepared in the same manner as described in Section 6.1 and then placement of aggregate base to replace the unstable soils. The aggregate base should be placed and compacted using static compactive effort in one 1-foot-thick lift by “pioneering” from the edge(s) of the work area. The recommended depths of stabilization (i.e. approximately 1 foot) could be greater depending on the condition encountered. We recommend that a test section be prepared during construction to evaluate the effectiveness of this option.

The intent of subgrade stabilization is to achieve a non-yielding subgrade when subjected to relatively heavy, rubber tired construction equipment loading such as a loaded water truck or loader with full bucket. The stabilized subgrade should be proof-rolled with this type of equipment after remediation to confirm that it is unyielding.

6.3. Temporary Excavations and Slot-Cutting

Temporary, unsurcharged excavation sides should be sloped no steeper than an inclination of 1H:1V (horizontal to vertical). Where excavations are performed adjacent to existing buildings, slot-cutting may be necessary. For temporary excavations adjacent to existing buildings where the excavation extends deeper than the bottom of the existing footing, slot cuts may be utilized. The slots should be no wider than 8 feet and should be excavated in an A-B-C sequence so that there are at least 16 feet spacing between any two excavated slots. The excavated slots should not be left open overnight and should be backfilled on the same day it was excavated before the next set of slots are excavated. If slot cuts are necessary, QAI should provide more specific recommendations on a case-by-case basis.

6.4. Shallow Foundations

A shallow foundation system consisting of spread footings may be used for support, provided that all the footings are placed on a subgrade prepared as described in Section 6.1. Footings should be designed and reinforced in accordance with the recommendations of the structural engineer and should conform to the 2010 California Building Code. Our foundation recommendations are based on assumed maximum loads of 4 kips per foot and 25 kips for wall and column foundations, respectively. If actual loads are greater, QAI should be notified so that the foundation recommendations can be reviewed and revised if needed.

Table 2
Geotechnical Design Parameters for
Continuous and Isolated Spread Footings on Compacted Fill

Minimum Dimensions for Foundation Footings	• At least 12 inches in width. • The bottom of the footing should be at least 1 foot below the lowest adjacent grade.
Allowable Bearing Pressure for Foundation Footings	• For continuous and pad foundations near grade, an allowable bearing pressure of 1,750 pounds per square foot (psf) can be used. This value may NOT be increased with deeper embedment or width in order to control settlement. • The allowable bearing values may be increased by one-third for transient live loads from wind or earthquake.
Estimated Total Settlement (Total/Differential)	• Approximately 1 inch. Differential settlement should be considered in the design and should be taken as approximately ½ to ⅔ of the total settlement.
Coefficient of Friction Below Footings	0.35 (for footings bearing on aggregate base materials)
Unfactored Lateral Passive Resistance	250 pcf (equivalent fluid pressure)

The total allowable lateral resistance can be taken as the sum of the friction resistance and passive resistance, provided that the passive resistance does not exceed two-thirds of the total allowable resistance. The passive resistance values may be increased by one-third when considering wind or seismic loading.

6.5. Retaining Walls

The recommendations for retaining walls assume that the backfill behind the retaining walls is level and do not include surcharge loads behind the wall. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind the walls and external hydrostatic pressure will not develop behind the wall. Drainage system should consist of drainage board material on the back of wall that drains into a 4 inch diameter perforated pipe. The pipe should be encased in at least 1 cubic foot of gravel and the gravel should be completely wrapped in non-woven filter fabric (such as Mirafi 140N or equivalent).

Walls that are supporting earth, have adequate drainage, and are restrained against rotation at the top (such as by a floor deck) may be designed for the “at-rest” earth pressure equivalent to a fluid weighing 60 pcf (also referred to as EFP, equivalent fluid pressure). Where adequate drainage is not provided behind walls, an undrained or submerged EFP of 95 pcf should be used in the design. Walls that are free to rotate at the top (such as cantilevered walls) and have adequate drainage, the lateral earth pressure may be designed for the “active” EFP of 45 pcf. Where adequate drainage is not provided behind walls, the undrained or submerged EFP of 85 pcf should be used in the design.

6.6. Infiltration System

Percolation testing was performed to determine the infiltration rate of the subgrade soil at various locations of the site. Three test areas were excavated and tested in general conformance with the test procedures described in DEHS, 1992, “On-Site Waste Water Disposal System, Soil Percolation (PERC) Test Report Standards,” Division of Environmental Health Services, dated August 1992. The percolation test locations are presented on Figure 2.

Each test location was excavated with a post-hole digger to approximately 7½ inches in diameter and to approximately 36 inches below the ground surface and backfilled with approximately 2 inches of gravel to protect the bottom from scouring when the water was added. The excavated material consisted generally of dark brown sandy silt in a moist to wet condition. The holes were pre-soaked by allowing a minimum of five gallons of water to percolate and saturate the lower 12 inches of the test holes. During the testing, the water level was measured in 30-minute increments. After each measurement, the water level was readjusted to the original reference level. The final two measurements were verified to not vary by more than ten percent. The calculated percolation rate for each test location is presented in Table 1 below.

Table 1
Percolation Test Results

Percolation Number	Location	Percolation Rate (minutes per inch)
P-1	East portion of site	67
P-2	Center portion of site	60
P-3	West portion of site	48

Groundwater was found to be approximately 12 feet below the existing ground surface. The historic high is reported to be at approximately 10 feet below the ground surface. Based upon information from our borings and the very low likelihood that ground water will rise above the historic high level, it is our opinion that the ground water surface may be considered to be at an elevation no higher than 12 feet below the existing ground surface. On this basis, if an infiltration system is used, the depth of the bottom of the infiltration zone should be no deeper than 2 feet below the existing ground surface.

6.7. Drainage Control

The control of surface water is essential to the satisfactory performance of the building and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the structure, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.

- Planters should not be located adjacent to the structure wherever possible. If planters are to be located adjacent to the structure, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.

- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.

- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.

- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

6.8. Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Based on laboratory testing, and observations made during our subsurface exploration, the near-surface soil at the site has a low to moderate potential for expansion. The results of expansion index (EI) testing indicate an EI of approximately 45, which is representative of the clayey silty site soils.

6.9. Soil Corrosion

The potential for the on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on representative soil samples to evaluate pH, minimum resistivity, and soluble chloride and sulfate contents. Table B-4 in Appendix B presents the results of our corrosivity testing. General recommendations to address the corrosion potential of the on-site soils are provided below. Imported fill materials, if used, should be tested to evaluate whether their corrosion potential is more severe than those assumed.

6.9.1. Reinforced Concrete

Laboratory tests indicate that the potential of sulfate attack on concrete in contact with the on-site soils is “negligible” based on ACI 318 Table 4.3.1.

Test results also indicate the potential for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil is low. Recommendations for protection from chloride attack may be provided by a corrosion specialist.

6.9.2. Metallic

Laboratory resistivity testing indicates that the on-site soils are moderate to severely corrosive to buried ferrous metals. As a consequence of these conditions, we recommend that consideration be given to using plastic piping instead of metal, where possible. Alternatively, a corrosion specialist should be consulted regarding suitable types of piping and appropriate protection for underground metal conduits.

7. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade and fill materials will be important to the performance of the proposed improvements. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

7.1. Plans and Specifications

The design plans and specifications should be reviewed by QAI prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications. Based on the work already performed, this office is best qualified to provide such review.

7.2. Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of fill materials, fill placement, foundation installation, and other site grading operations should be observed and tested. The subgrade exposed during construction may differ from that encountered in the test borings. Continuous observation by a representative of QAI during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

8. LIMITATIONS

The recommendations and opinions expressed in this report are based on QAI's review of information obtained from field explorations, and on laboratory testing. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of removal of unsuitable soil, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which QAI has no control.

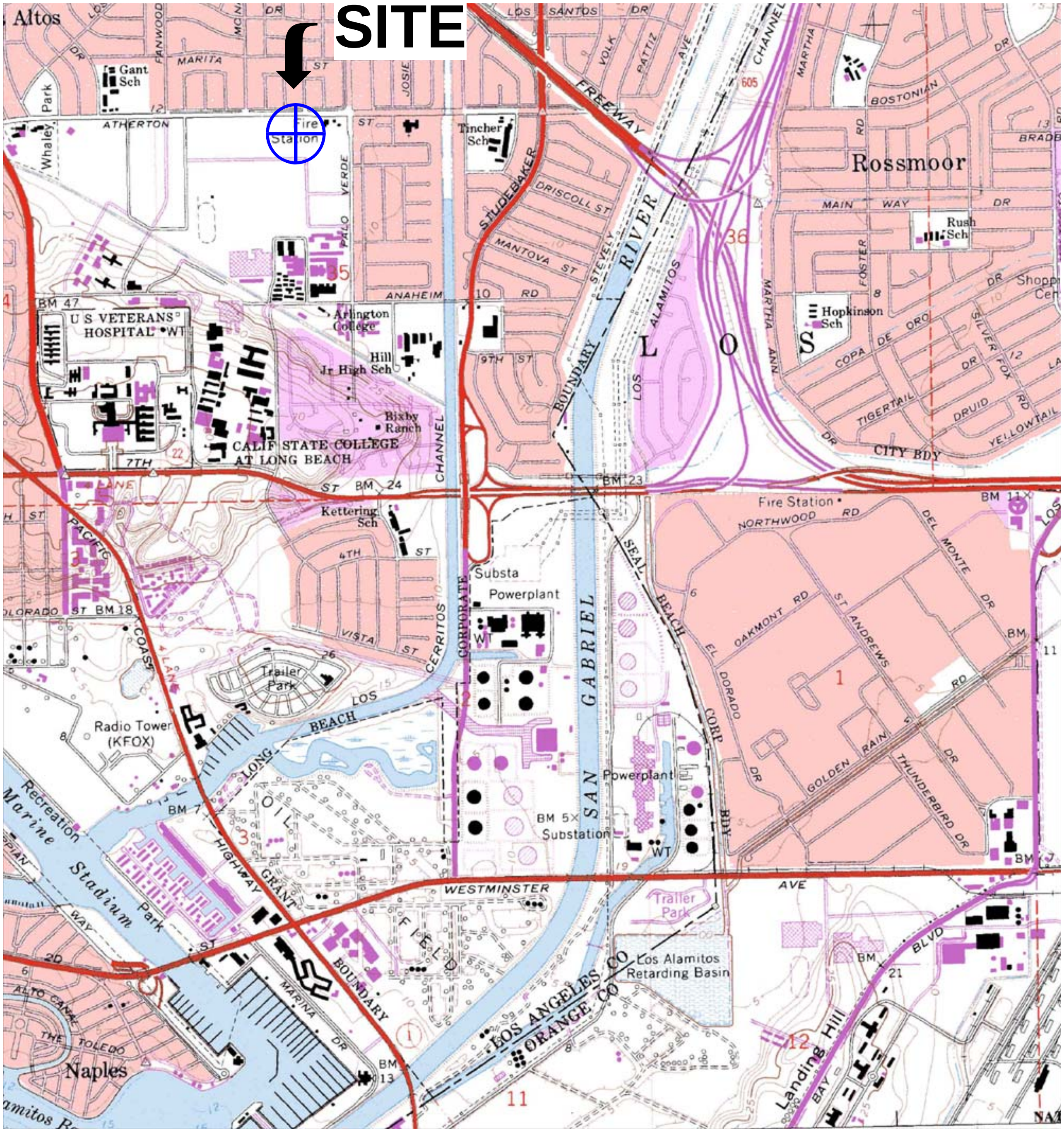
QAI's recommendations for this site are, to a high degree, dependant upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for QAI to observe grading operations and foundation excavations for the proposed construction. If parties other than QAI are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. QAI should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

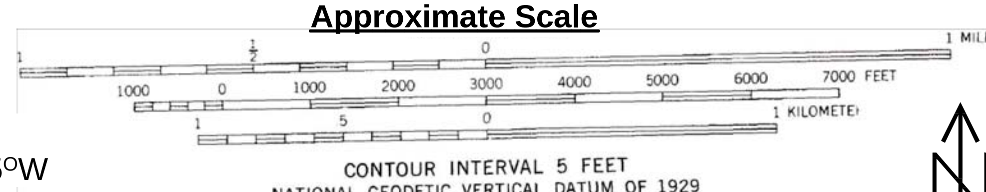
This report has been prepared for the exclusive use by California State University Long Beach and their agents for specific application to the proposed Bob Cole Conservatory of Music project on the California State University campus in Long Beach, California. Any other party, other than California State University Long Beach, who wishes to use this report for an adjacent or nearby project shall notify QAI of such intended use. Land use, site conditions or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the new project, QAI may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release QAI from any liability resulting from the use of this report by any unauthorized party.

QAI has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

FIGURES



Approximate Scale



N

Site Coordinates

Latitude: 33.78732°N
Longitude: 118.11225°W

Reference: USGS, 1964 (Photorevised 1981), Los Alamitos Quadrangle, California - Los Angeles Co., 7.5 Minute Series (Topographic)

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SITE LOCATION MAP

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Long Beach, California

PROJECT NO.	REPORT DATE	
110403.1	June 2011	FIGURE 1

LEGEND

0' 50'

B-4 approximate boring location and number

P-3 approximate percolation test location and number

N

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BORING LOCATION MAP		
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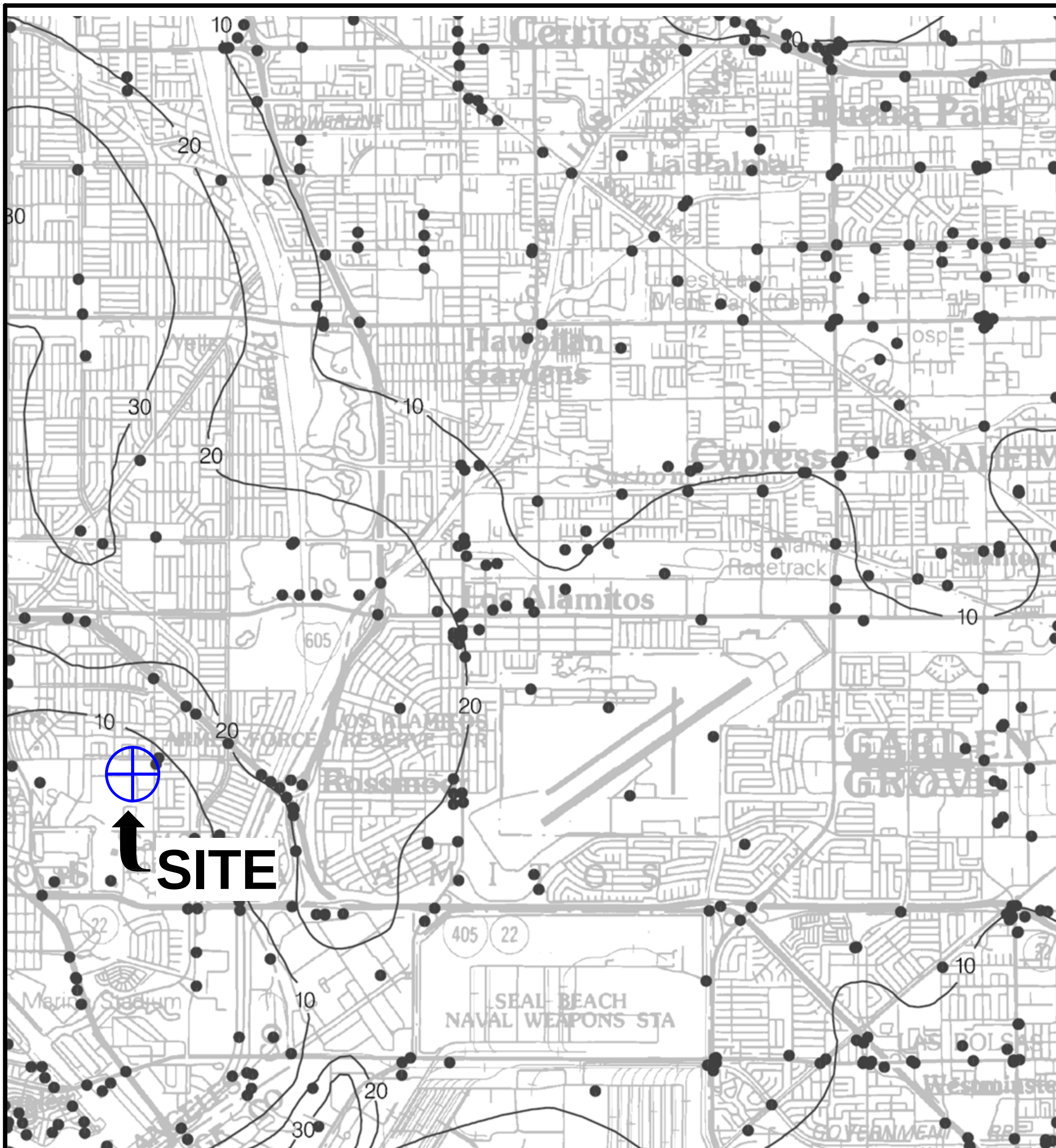
PRJECT NO. 110403.1	REPORT DATE June 2011	FIGURE 2



LEGEND

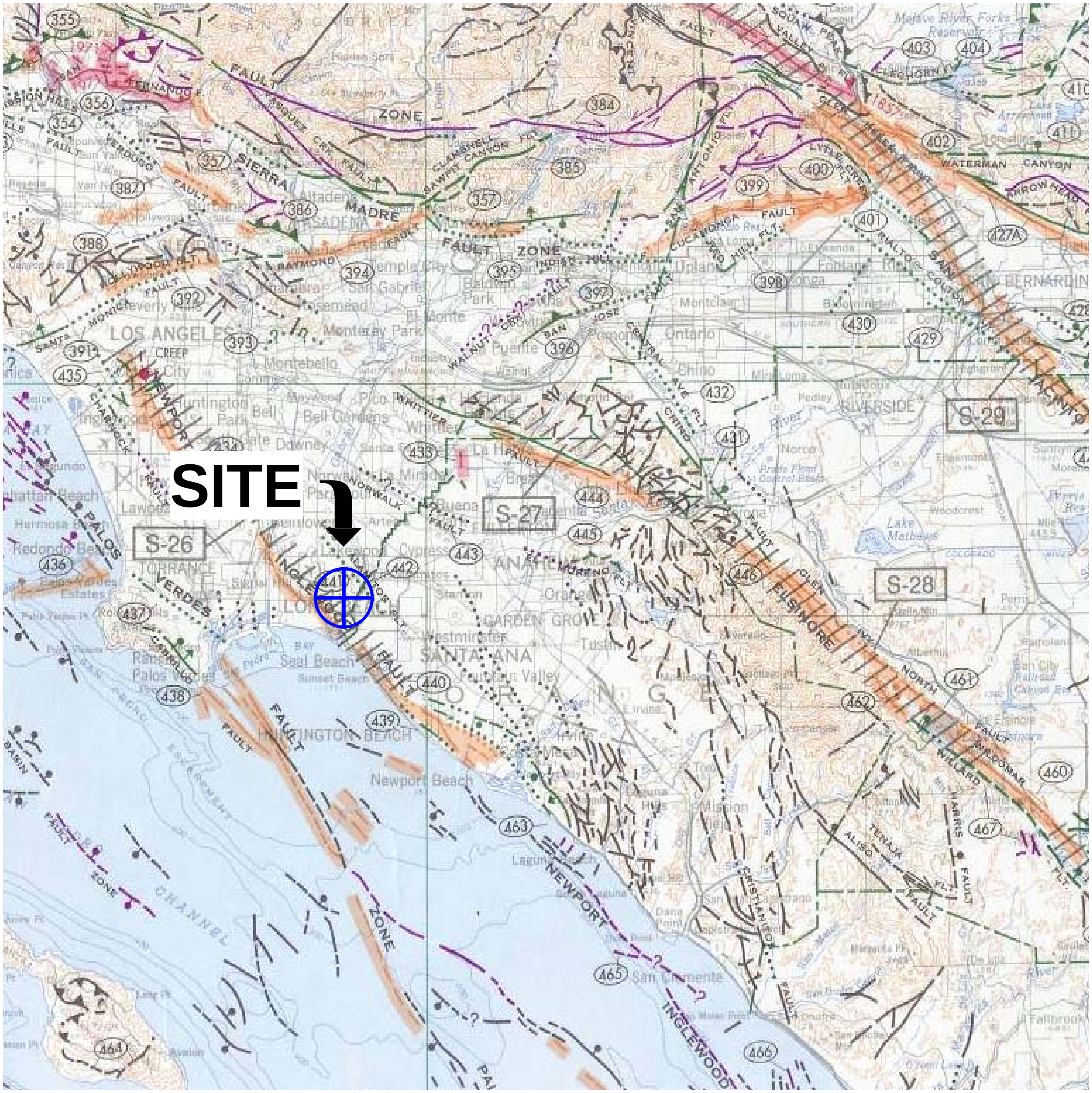
30 Depth to ground water in feet

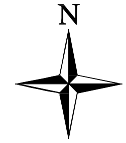
ONE MILE
SCALE

N

Reference: California Department of Conservation (2001)

QUALITY ASSURANCE QAI INSPECTIONS	HISTORICAL HIGH GROUNDWATER MAP		
	Bob Cole Conservatory of Music California State University Long Beach Long Beach, California		
	PROJECT NO. 110403.1	REPORT DATE June 2011	FIGURE 3





Reference: Jennings (1994)

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FAULT LOCATION MAP

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PROJECT NO.	REPORT DATE	FIGURE 4
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Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	10,000		? ? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000		? ? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Pleistocene age.
	Early Quaternary		1,600,000		? ?	Undivided Quaternary faults – most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
						Late Cenozoic faults within the Sierra Nevada, including parts of, but not restricted to, the Foothills fault system. These faults may have been active in Quaternary time.	
Pre-Quaternary			4.5 billion (Age of earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive. Pre-Quaternary faults not shown in Nevada and Oregon.	Fault cuts strata of Pliocene or older age.

EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage – slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.

Pink band added to emphasize location of historic fault displacement.

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FAULT LOCATION MAP

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PROJECT NO.	REPORT DATE	FIGURE 4
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ADDITIONAL FAULT SYMBOLS

U = Upthrown side (relative or apparent)
D = Downthrown side (relative or apparent)

Bar and ball on downthrown side (used where space is limited).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip.

OTHER SYMBOLS

529 Numbers refer to annotations listed in the Appendices of the accompanying report. Annotations include fault name, age of fault movement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geologist to delineate zones to encompass all potentially and recently active faults.

Cinder cone and other types of volcanoes. Most were active in Pleistocene time, some are Holocene, a few are historic.

21 8 Number in box or circle refers to Table 4 (Recent Volcanic Eruptions) in accompanying report. (Box refers to California, circle to Nevada.)

(1786 A.D.) = Date of historic volcanic eruption.

(9,500 B.P.) = Eruption occurrence in years before present (B.P.).

★ (0.5 m.y.) = Age of volcanic flow or eruption in million years (m.y.).

SPECIAL NOTATIONS

A triangle to the right or left of the date indicates termination point of observed surface displacement.

1951 1951

Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along fault break. 1906

Dot on fault indicates location where fault creep slippage has been observed and recorded.

Square on fault indicates where fault creep slippage has occurred that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points). 1968 1969 1968

Holocene fault displacement (during past 10,000 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting. Pale orange band added to emphasize location of Holocene fault displacement.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Late Cenozoic faults within the Sierra Nevada including, but not restricted to, the Foothills fault system. Faults show stratigraphic and/or geomorphic evidence for displacement of late Miocene and Pliocene deposits. By analogy, late Cenozoic faults in this system that have been investigated in detail may have been active in Quaternary time. (Data from PG&E, 1993).

Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

S-25 Fault segment associated with a significant linear trend of accurately located earthquake epicenters (magnitude 0.2 or greater). Generally aligned along strike slip faults having Quaternary displacement, but not necessarily with historic surface rupture. Lack of seismic activity along any fault is no indication that the fault may not be active in the future (e.g. San Andreas fault north of San Francisco). Epicenter data are derived from closely spaced seismic stations and include either continuing microseismicity or aftershocks associated with relatively large earthquakes.

Aligned seismicity on fault segments are referenced in Appendices C and E .

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FAULT LOCATION MAP		
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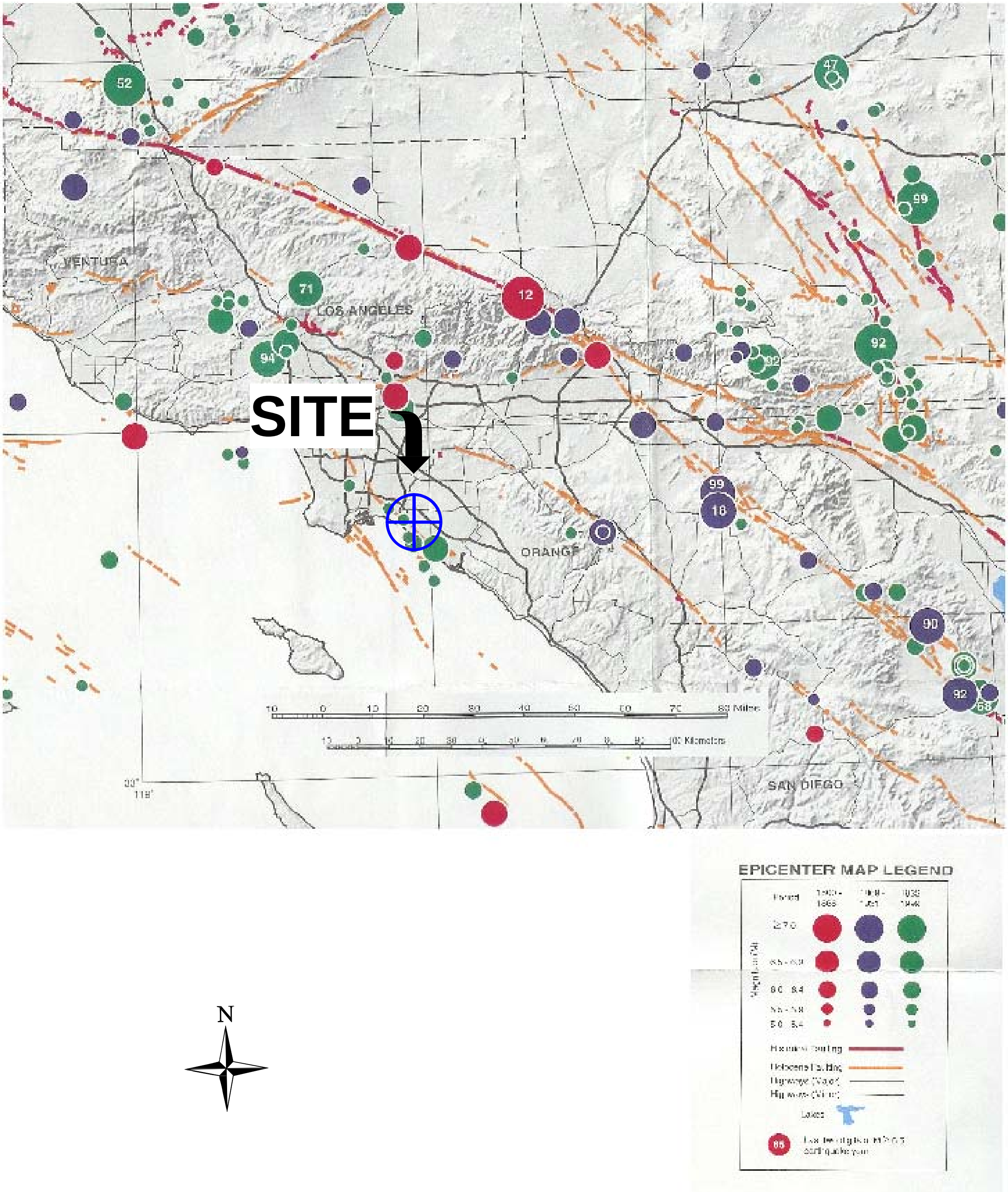
PROJECT NO. 110403.1	REPORT DATE June 2011	FIGURE 4 PAGE 3 OF 3



Reference: Toppozada and others (2000)

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HISTORICAL SEISMICITY, 1800 - 1999		
Bob Cole Conservatory of Music California State University Long Beach Long Beach, California		
PROJECT NO. 110403.1	REPORT DATE June 2011	FIGURE 5

Appendix A

Field Exploration

Appendix A
Field Exploration

General

The subsurface exploration program for the proposed project consisted of drilling and logging five exploratory borings. The exploratory borings were advanced using a track-mounted CME-75 hollow-stem-auger drill rig and hand tools. The drilling was performed by JET Drilling of Signal Hill, California. The borings reached depths of approximately 5 to 26½ feet below the existing grades. Upon completion of excavating and testing (percolation tests), the boreholes were backfilled with soil from the cuttings.

Drilling and Sampling

The Boring Logs are presented as Figures A-2 through A-6. An explanation of these logs is presented as Figure A-1. The Boring Logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The log also shows the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a QAI engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

A California modified sampler was used to obtain drive samples of the soil encountered. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that is driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in brass rings for laboratory testing. Additional soil from each drive remaining in the cutting shoe was usually discarded after visually classifying the soil. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs.

Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is advanced into the soil at the bottom of the drilled hole a total of 18 inches. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. Soil samples obtained by the SPT were retained in plastic bags. Both the California modified and the SPT sampler were driven by an automatic-trip hammer weighing 140 pounds at a drop height of approximately 30 inches.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			Graphic Symbols	Group Symbols	Typical Descriptions
COARSE-GRAINED SOILS — More than 50% retained on No. 200 sieve	GRAVELS — 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels		GW	Well-graded gravels and grave-sand mixtures, little or no fines
				GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with Fines		GM	Silty gravels, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS — More than 50% of coarse fraction passes No. 4 sieve	Clean Sands		SW	Well graded sands and gravelly sands, little or no fines
				SP	Poorly graded sands and gravelly sands, little or not fines
		Sands with Fines		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOIL — 50% or more passes No. 200 sieve	SILTS and CLAYS — Liquid Limit 50% or less			ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clay
				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 sands or silts, elastic silts
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity
Highly Organic Soils				PT	Peat, muck and other highly organic soils

COARSE-GRAINED SOILS

Relative Density	SPT (blows/ft.)	Relative Density (%)
Very Loose	< 4	0 – 15
Loose	4 – 10	15 – 35
Medium Dense	10 – 30	35 – 65
Dense	30 – 50	65 – 85
Very Dense	> 50	85 – 100

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

FINE-GRAINED SOILS		Torvane	Pocket Penetrometer
Consistency	SPT (blows/ft.)	Undrained Shear Strength (tsf)	Unconfined Compressive Strength (tsf)
Very Soft	< 2	< 0.13	< 0.25
Soft	2 – 4	0.13 – 0.25	0.25 – 0.5
Medium Stiff	4 – 8	0.25 – 0.5	0.5 – 1.0
Stiff	8 – 15	0.5 – 1.0	1.0 – 2.0
Very Stiff	15 – 30	1.0 – 2.0	2.0 – 4.0
Hard	> 30	> 2.0	> 4.0

Sample Symbol	Sample Type	Description
	SPT	1.4 in. I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube
	California Modified	No Recovery w/ California Modified Sampler
	SPT	No Recovery w/ SPT Sampler

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EXPLANATION FOR LOG OF BORINGS

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PROJECT NO.	REPORT DATE	FIGURE A-1
110403.1	June 2011	Sheet 1 of 2

ABBREVIATION OF LABORATORY TESTING			
ATT	Atterberg Limits	O	Organic Content
C	Consolidation	pH	pH
CBR	California Bearing Ratio	PP	Pocket Penetrometer
Ch	Water Soluble Chlorides	RV	Resistance Value
CORR	Corrosivity Series	SE	Sand Equivalent
DS	Direct Shear	SG	Specific Gravity
EI	Expansion Index	SO4	Water Soluble Sulfates
ER	Electrical Resistivity	TX	Triaxial Compression
GS	Grain Size Distribution	TV	Torvane Shear
K	Permeability	UC	Unconfined Compression
MAX	Moisture/Density (Modified Proctor)	WASH	No. 200 Wash Sieve

Notes:	D_{10}, D_{30}, D_{50}, and D_{100}	Particle size corresponding to specified value on the percentage finer scale
	Cu	Coefficient of uniformity
	Cc	Coefficient of curvature

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EXPLANATION FOR LOG OF BORINGS

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PROJECT NO.	REPORT DATE	FIGURE A-1
110403.1	June 2011	Sheet 2 of 2

DATE DRILLED	5/16/11	LOGGED BY	AB	**BORING NO.**	**1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER	not encountered
DRILLING METHOD	8" HSA	DRILLER	JET Drilling	SURFACE ELEVATION	17 ft +(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	SAMPLE NO.	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									ML	SANDY SILT, dark brown, dry, stiff; fine-grained sand; trace fine gravel reddish brown, moist, no gravel gas line at 3 feet
12	5									Total Depth = 5.0 feet Backfilled on 5/17/2011 Groundwater not encountered Backfilled with soil cuttings
7	10									
2	15									
-3	20									
-8	25									
-13	30									
-18	35									

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LOG OF BORING

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PROJECT NO.	REPORT DATE	FIGURE A-2
110403.1	June 2011	Sheet 1 of 1

DATE DRILLED	5/16/11	LOGGED BY	AB	**BORING NO.**	**2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER	12
DRILLING METHOD	8" HSA	DRILLER	JET Drilling	SURFACE ELEVATION	16 ft +(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	SAMPLE NO.	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0	Bulk					EI		ML	SANDY SILT, dark brown, moist, medium stiff; fine-grained sand
11	5	Driven		19	24.6	92.8				
									CL	LEAN CLAY, dark brown, moist, stiff
6	10	Driven		6			WASH, ATT		ML	SILT, medium brown, wet, medium stiff; trace very fine-grained sand
1	15	Driven		25	32.7	88.6			CL-ML	SILTY CLAY, medium brown, wet, very stiff; trace very fine-grained sand
									ML	SILT, medium brown to gray, wet, very stiff
-4	20	Driven		18						
-9	25	Driven		35	29.2	92.7				rust colored staining
										Total Depth = 26.5 feet Backfilled on 5/16/2011 Groundwater encountered at 12 feet Backfilled with soil cuttings
-14	30									
-19	35									

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LOG OF BORING

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PROJECT NO.	REPORT DATE	FIGURE A-3
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DATE DRILLED	5/16/11	LOGGED BY	AB	**BORING NO.**	**3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER	12.5
DRILLING METHOD	8" HSA	DRILLER	JET Drilling	SURFACE ELEVATION	17 ft +(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	SAMPLE NO.	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										ML	SANDY SILT, dark reddish brown, moist, medium stiff; very fine-grained sand; trace fine gravel; trace asphalt pieces up to 1 inch in diameter
12	5				9	26.9	86.0	WASH, C			
										CL	LEAN CLAY, dark brown, moist, medium stiff
7	10				4					ML	SILT, medium brown, wet, soft
2	15				23	28.3	93.8	ATT			very stiff
-3	20				12						
											increase plasticity
-8	25				28	29.2	93.9				
											Total Depth = 26.5 feet Backfilled on 5/16/2011 Groundwater encountered at 12.5 feet Backfilled with soil cuttings
-13	30										
-18	35										

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LOG OF BORING

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Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE A-4
110403.1	June 2011	Sheet 1 of 1

DATE DRILLED	5/16/11	LOGGED BY	AB	**BORING NO.**	**4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER	13
DRILLING METHOD	8" HSA	DRILLER	JET Drilling	SURFACE ELEVATION	17 ft +(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES	SAMPLE NO.	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk					CORR		ML	SANDY SILT, medium brown, moist, medium stiff; fine-grained sand
12	5	Driven		8	30.2	81.6	DS			
									CL	LEAN CLAY, dark brown, moist, medium stiff
7	10	Driven		14	33.6	85.5	C		ML	SILT, medium brown, wet, stiff; trace very fine-grained sand
2	15	Driven		15						
-3	20	Driven		18	28.1	91.2				
-8	25	Driven		11						
										Total Depth = 26.5 feet Backfilled on 5/16/2011 Groundwater encountered at 13 feet Backfilled with soil cuttings
-13	30									
-18	35									

QUALITY ASSURANCE QAI INSPECTIONS

LOG OF BORING

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Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE A-5
110403.1	June 2011	Sheet 1 of 1

DATE DRILLED	5/23/11	LOGGED BY	AB	BORING NO.	5
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER	not encountered
DRILLING METHOD	Hand Auger	DRILLER	TWINING	SURFACE ELEVATION	18 ft +(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	SAMPLE NO.	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									ML	SANDY SILT, dark brown, moist, medium stiff; fine-grained sand; trace asphalt pieces up to 1 inch in diameter
13	5									very moist
									CL	LEAN CLAY, medium brown, moist, medium stiff
8	10									Total Depth = 9.5 feet Backfilled on 5/23/2011 Groundwater not encountered Backfilled with soil cuttings
3	15									
-2	20									
-7	25									
-12	30									
-17	35									

QUALITY ASSURANCE QAI INSPECTIONS

LOG OF BORING

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PROJECT NO.	REPORT DATE	FIGURE A-6
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Appendix B

Laboratory Testing

Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A and also summarized in Table B-1.

Atterberg Limits

Plasticity index testing was performed on selected samples obtained from the borings to evaluate plasticity characteristics and to aid in the classification of the soil. The tests were performed in general accordance with ASTM D4318. The results are presented on Figure B-1.

Wash Sieve

The amount of fines passing the No. 200 sieve was evaluated by the wash sieve on selected soil samples. The test procedure was in general accordance with ASTM D 1440. The test results are presented in Table B-2.

Direct Shear Tests

Direct shear tests were performed on selected samples in general accordance with the latest version of ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The samples were inundated during shearing to represent adverse field conditions. Test results are presented on Figure B-2.

Consolidation

Consolidation tests were performed on selected samples in general accordance with the latest version of ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are presented on Figures B-3 and B-4.

Expansion Index Tests

The expansion index of a selected soil sample was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of Expansion Index test are presented on Table B-3.

Corrosivity

Soil pH and resistivity test were performed by Anaheim Test Laboratories on a representative soil sample in general accordance with the latest version of California Test Method 643. The chloride content of a selected sample was evaluated in general accordance with the latest version of California Test Method 422. The sulfate content of a selected sample was evaluated in general accordance with the latest version of California Test Method 417. The test results are presented on Table B-4.

Table B-1
Laboratory Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Unit Weight (pcf)
B-2	5	24.6	92.8
B-2	15	32.7	88.6
B-2	25	29.2	92.7
B-3	5	26.9	86.0
B-3	15	28.3	93.8
B-3	25	29.2	93.9
B-4	5	30.2	81.6
B-4	10	33.6	85.5
B-4	20	28.1	91.2

Table B-2
No. 200 Wash Sieve Results

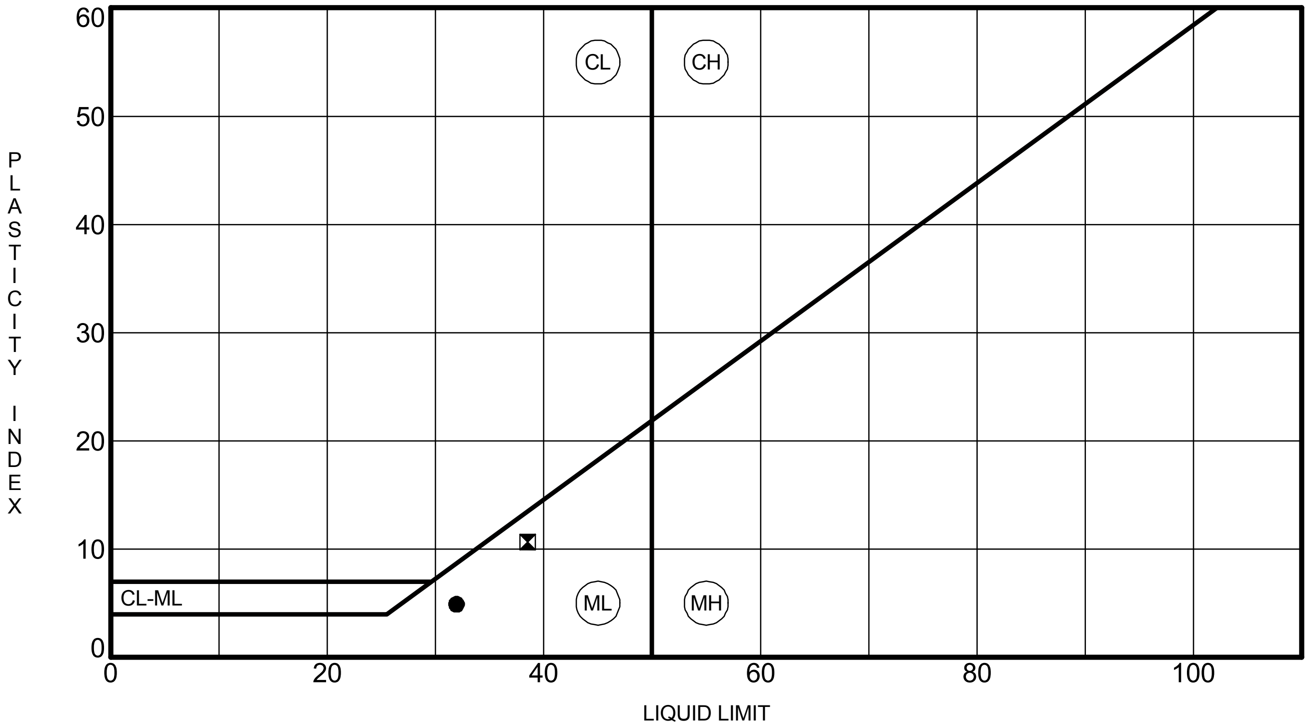
Boring No.	Depth (feet)	Percent Passing #200
B-2	10	91.0
B-3	5	95.0

Table B-3
Expansion Index Test Result

Boring No.	Depth (feet)	Expansion Index
B-2	0-5	45

Table B-4
Corrosivity Test Results

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (%)	Water Soluble Chloride (%)	Minimum Resistivity (ohm-cm)
B-4	0-5	7.5	0.0379	0.0445	1,548



	Sample Location	LL	PL	PI	U.S.C.S. Classification
●	**2 at 10 ft**	**32**	**27**	**5**	**SILT (ML)**
⧗	**3 at 15 ft**	**39**	**28**	**11**	**SILT (ML)**

QUALITY ASSURANCE QAI INSPECTIONS

ATTERBERG LIMITS

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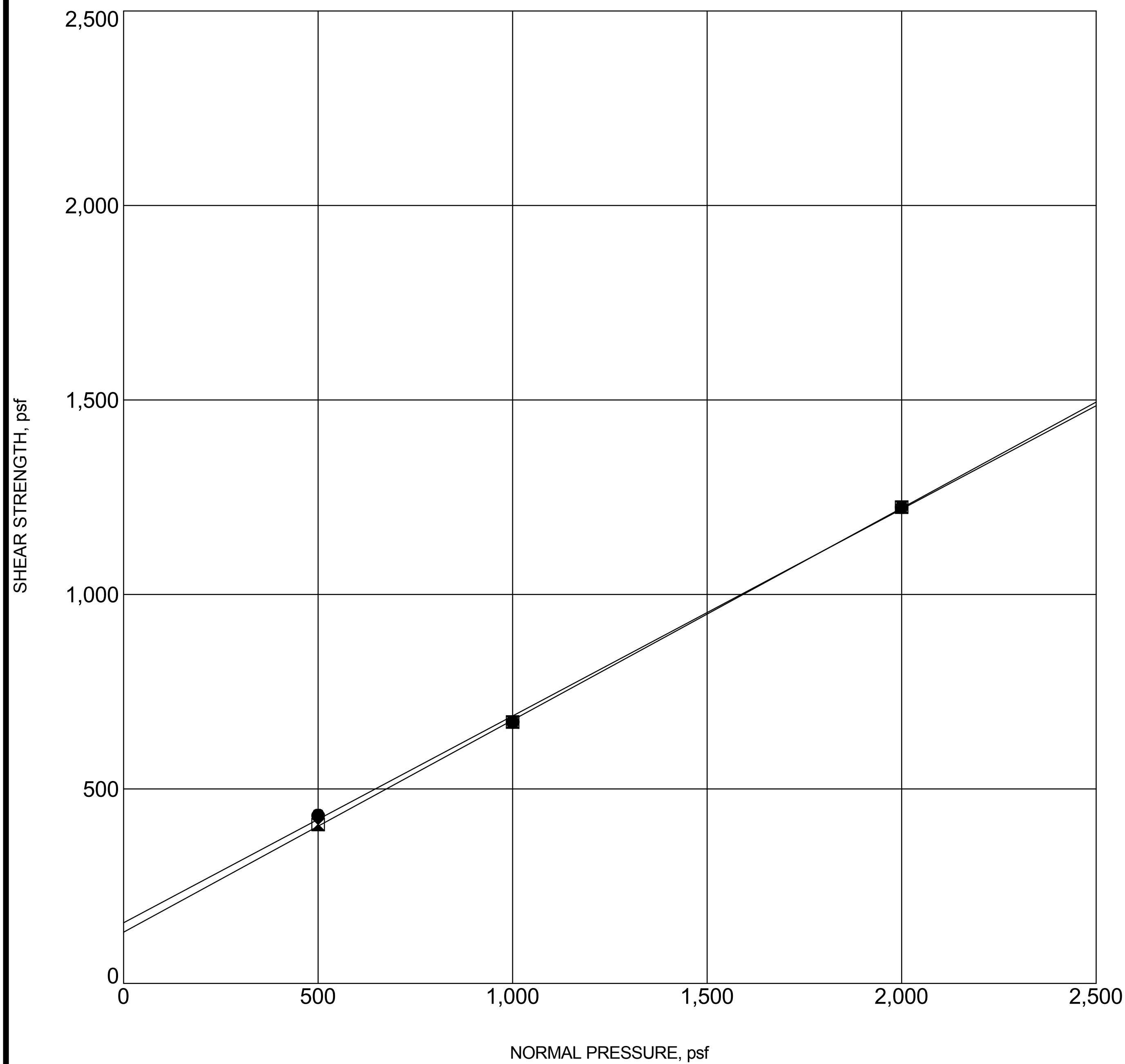
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-1
110403.1	June 2011	Sheet 1 of 1



Shear Strength Parameters

Boring No.:	4		**Peak**	**Ultimate**
Sample Depth (ft):	5	**Cohesion, C (psf):**	155	130
Sample Description:	SILT (ML)	**Friction Angle, Ø (deg):**	28	29
Strain Rate (in./min):	0.005			
Dry Density (pcf):	81.6	**Initial Moisture (%):**	30.2	
		Final Moisture (%):	31.6	

QAI DIRECT SHEAR 110403.1 LAB DATA.GPJ TWINING LABS.GDT 3/6/11

QUALITY ASSURANCE QAI INSPECTIONS

DIRECT SHEAR TEST

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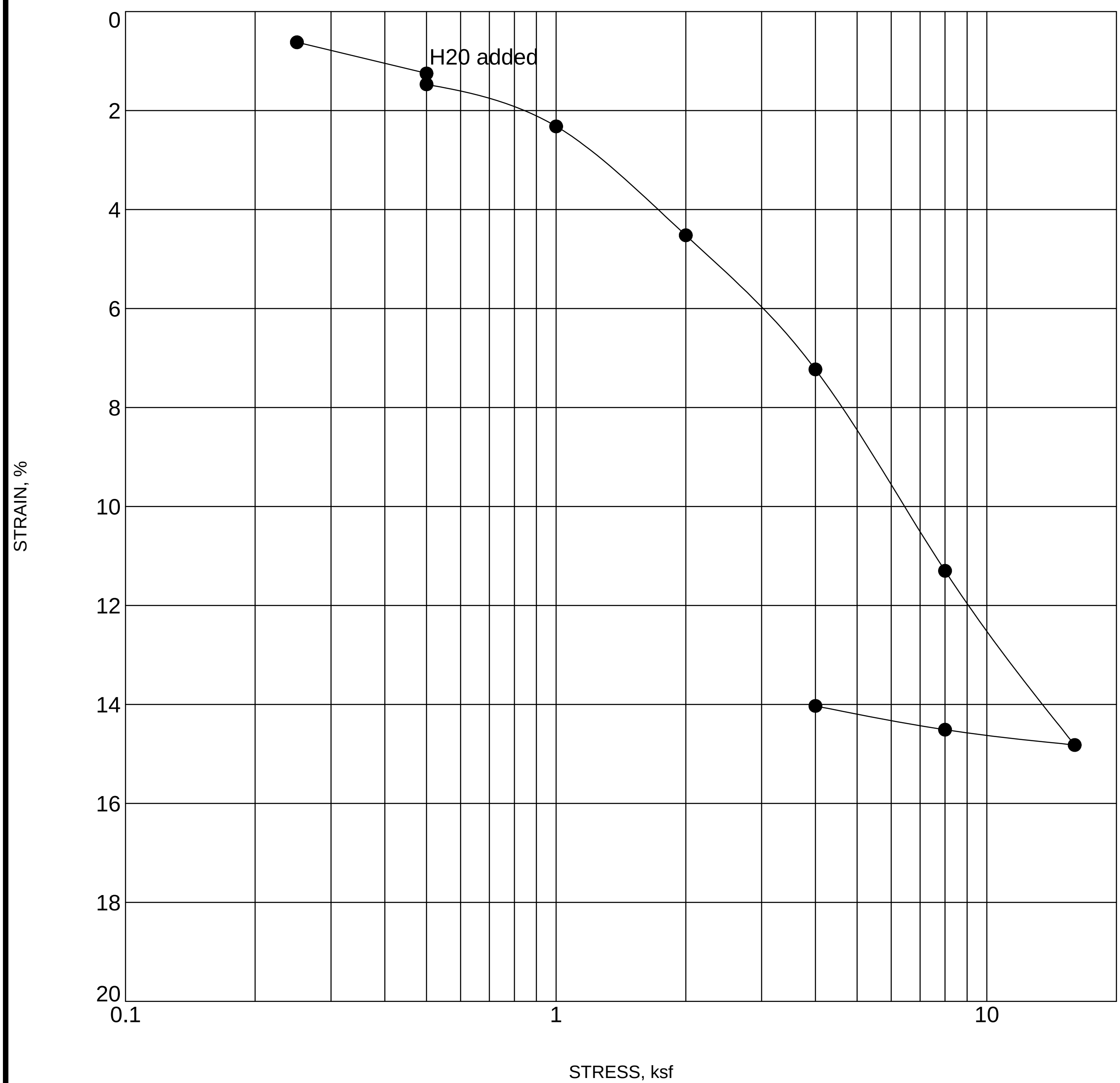
California State University Long Beach
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-2
110403.1	June 2011	



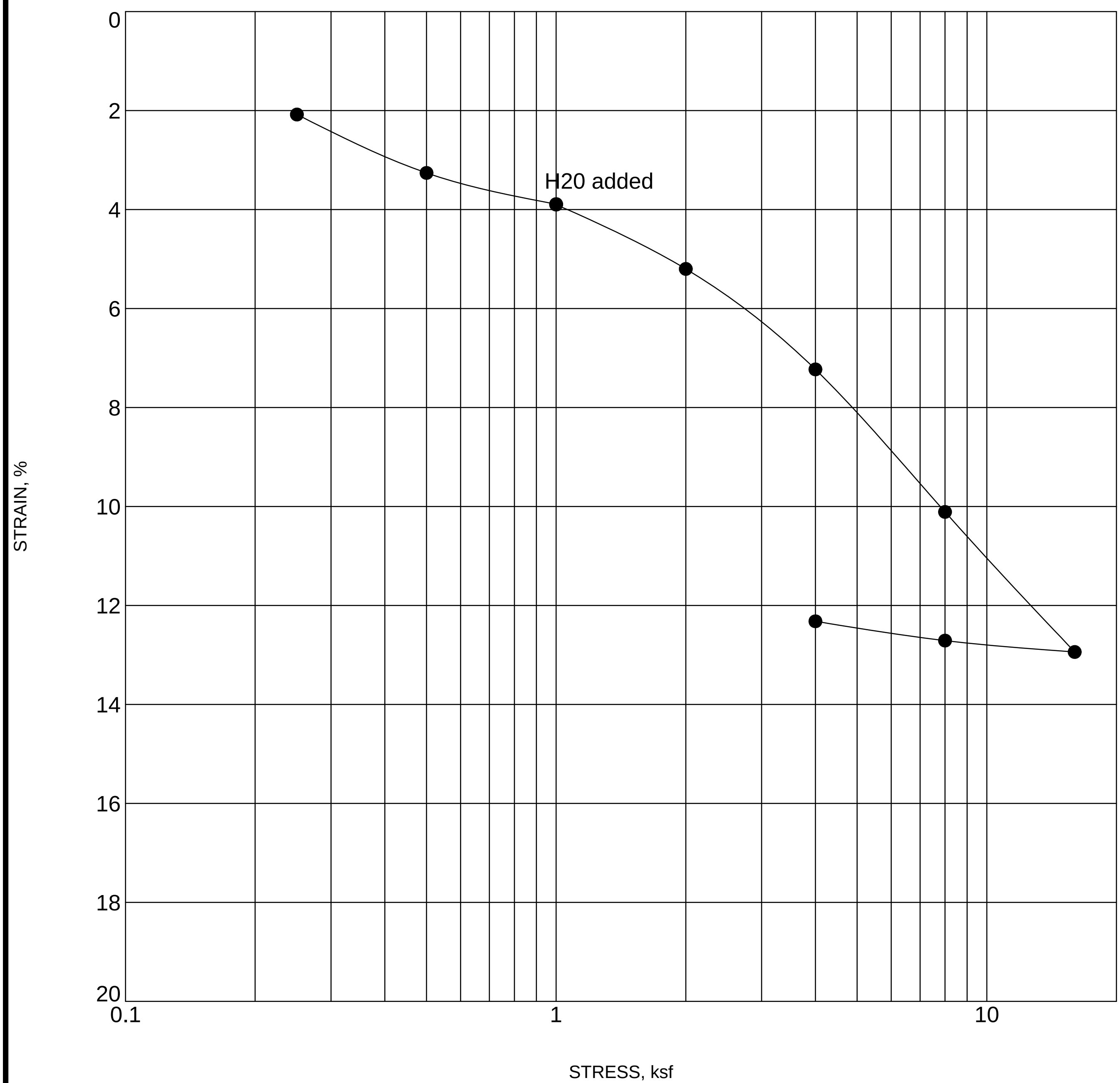
	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**3 at 5 ft**	**SILT (ML)**	**86.1**	**26.9**

QUALITY ASSURANCE QAI INSPECTIONS

CONSOLIDATION TEST

Bob Cole Conservatory of Music
California State University Long Beach
Long Beach, California

PROJECT NO.	REPORT DATE	
110403.1	June 2011	FIGURE B-3



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**4 at 10 ft**	**SANDY SILT (ML)**	**87.2**	**33.6**

QAI CONSOL STRAIN 110403.1 LAB DATA.GPJ TWINING LABS.GDT 3/6/11

CONSOLIDATION TEST

Bob Cole Conservatory of Music
California State University Long Beach
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-4
110403.1	June 2011	