344-01

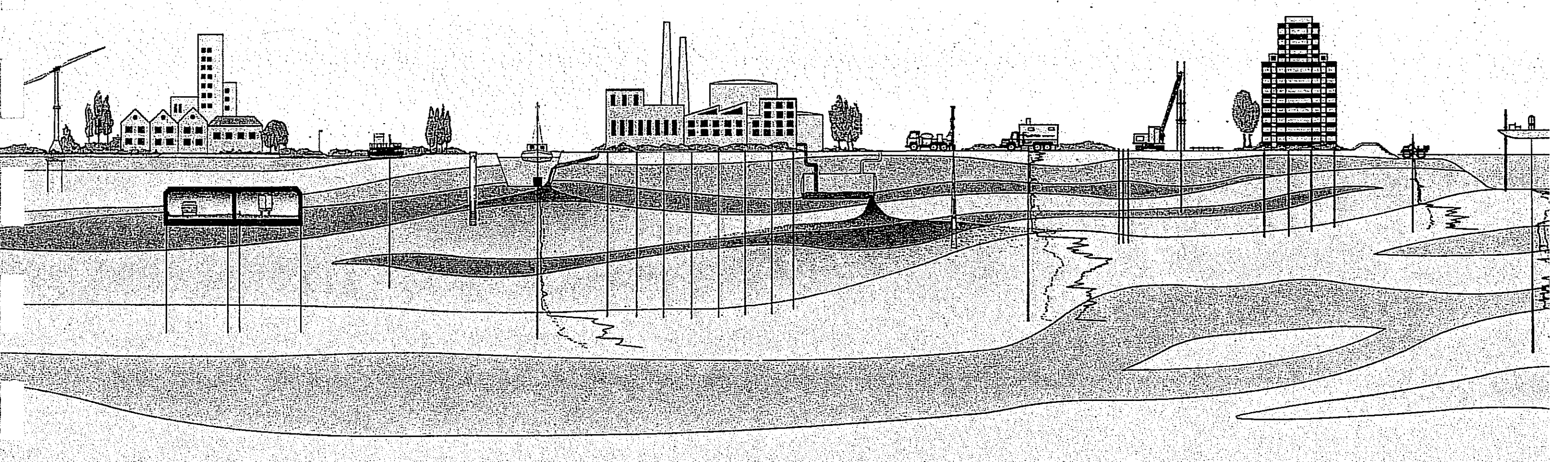
FUGRO WEST, INC.

GEOTECHNICAL REPORT
DEVEREUX AND PHELPS CREEK BRIDGES
UNIVERSITY OF CALIFORNIA SANTA BARBARA
SANTA BARBARA COUNTY, CALIFORNIA

Prepared for:

UNIVERSITY OF CALIFORNIA

March 2007



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March 23, 2007
Project No. 3064.045

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Attention: *Mr. Erich Brown*

Subject: Geotechnical Report, Devereux and Phelps Creek Bridges, University of California Santa Barbara, Santa Barbara County, California.

Dear Mr. Brown:

Fugro West, Inc., (Fugro) is pleased to present this geotechnical report for the proposed bridges at Devereux Creek and Phelps Creek. The structures will be located west of the main campus of the University of California Santa Barbara (UCSB) and are part of the proposed North Campus faculty and student housing projects. Fugro previously prepared geotechnical engineering reports for those two housing projects. The proposed bridges, however, were not a part of the projects at that time. Our findings and recommendations for the housing projects are provided in Fugro (2004a) and Fugro (2004b).

On the basis of information provided in your email dated December 8, 2006, and discussions with Penfield & Smith Engineers, we understand that Con/Span-type bridges are being considered for the project and that CONTECH will likely be responsible for the design and construction of the bridges.

The geotechnical report presents field and laboratory data collected during our geotechnical investigation, and provides geotechnical recommendations for the design of the bridges. The opinions and recommendations presented herein were developed from a review of existing geotechnical and geologic data acquired by Fugro. Additional geotechnical engineering and geologic services may be required as the project continues through the design process.

This report was prepared in general accordance with our proposal dated January 9, 2007. Our services were performed under our Professional Services Agreement between the Regents of the University of California and Fugro West, dated February 8, 2007. Authorization for our services was provided by UCSB Fund Number FM070351/988031/648051, Authorization No. 001 dated January 17, 2007.

We appreciate the opportunity to provide our services on this project. Please contact the undersigned if you have questions regarding this report or require additional information.

Sincerely,
FUGRO WEST, INC.

Gregory S. Denlinger, GE 2249
Associate Engineer

REGISTERED PROFESSIONAL ENGINEER
GREGORY S. DENLINGER
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GEOTECHNICAL
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Copies: 7 – Addressee

CONTENTS

Page

1.0 SITE AND PROJECT DESCRIPTION 1
1.1 Devereux Creek Bridge 1
1.2 Phelps Creek Bridge 1

2.0 WORK PERFORMED 2
2.1 Purpose 2
2.2 Scope 2
2.3 Field Exploration 2
2.4 Laboratory Testing 3
2.5 General Conditions 3

3.0 SITE CONDITIONS 3
3.1 Geologic Setting 3
3.2 Subsurface Conditions 4
3.2.1 Devereux Creek Bridge 4
3.2.2 Phelps Creek Bridge 5

4.0 GEOLOGIC HAZARDS 6
4.1 Probabilistic Seismic Hazard Analysis 7
4.2 Ground Rupture Potential 7
4.3 Liquefaction 7

5.0 CONCLUSIONS AND RECOMMENDATIONS 9
5.1 Summary of Findings 9
5.1.1 Devereux Creek Bridge 9
5.1.2 Phelps Creek Bridge 9
5.1.3 Suggested Materials Specifications 10
5.1.4 Grading for Structures 11
5.1.5 Grading for Pavement Areas 12
5.1.6 Fill Placement 12
5.1.7 Backfill and Compaction 13
5.2 Con/Span Bridge Structure Design 13
5.2.1 Shallow Foundation Design 13
5.2.2 Resistance to Lateral Loads 14
5.2.3 Deep Foundation Design 14
5.2.4 Lateral Earth Pressures 18
5.2.5 Retaining Wall Backfill and Drainage 18
5.3 Corrosion Considerations 19
5.4 Construction considerations 19

5.4.1 Temporary Slopes 19
5.4.2 Dewatering 20

6.0 CONTINUATION OF SERVICES 20

7.0 REFERENCES 22

PLATES

Plate

Vicinity Map 1
Exploration Location Plan – Devereux Bridge 2a
Exploration Location Plan – Phelps Bridge 2b

APPENDICES

APPENDIX A FIELD EXPLORATION

Log of Drill Hole Nos. DH-101 through DH-104 ...... Plates A-1 through A-4
Key To Terms & Symbols Used On Logs .... Plate A-5
Log of CPT Nos. CPT-101 through CPT-104.......... Plates A-6 through A-9
Key to CPT Logs .... Plate A-10

APPENDIX B LABORATORY TEST RESULTS

Summary of Laboratory Test Results .... Plate B-1
Plasticity Chart .... Plate B-2
Consolidation Test Results .... Plates B-3a through 3c

1.0 SITE AND PROJECT DESCRIPTION

1.1 DEVEREUX CREEK BRIDGE

The Devereux Creek Bridge site is located at the north end of the Devereux Slough immediately adjacent to the Ocean Meadows Golf Course. The proposed bridge will span the small drainage channel formed by Devereux Creek. Access across this drainage channel is currently provided by an embankment and concrete apron constructed over a 36-inch-diameter storm drain. The concrete apron is at an elevation lower than the surrounding topography and currently serves as an overflow area during periods of high flow in the channel. The location of the proposed Devereux Creek Bridge is shown on Plate 1 – Site Vicinity Map and the general layout of the Devereux Creek Bridge is shown on Plate 2a – Exploration Location Plan – Devereux Bridge.

We understand the proposed bridge will be a pre-fabricated, Con/Span bridge that will be designed and constructed by CONTECH Inc. The Con/Span bridge will be a pre-cast concrete arch-type bridge with a span of about 42 feet. The distance from the bridge soffit to the creek bottom will be about 7 feet. The creek bottom will be at approximately elevation 0.33 feet and will be covered with armortec concrete erosion control material. Abutment walls will extend both upstream and downstream of the bridge. The grade in the bridge approach areas will be slightly lower than the existing grade and the bridge and adjacent area are designed to accommodate overflow conditions. Also, a boulder berm and some minor grading is planned upstream of the bridge and new pavement sections are planned east and west of the bridge.

1.2 PHELPS CREEK BRIDGE

The Phelps Creek (ditch) Bridge site is located about 3,000 feet north of the proposed Devereux Creek Bridge at the west end of Marymount Drive in Goleta, California. Marymount Drive currently terminates immediately east of Phelps Creek. The proposed bridge will span the Phelps Creek flood control channel. Phelps Creek consists of an unimproved channel about 7 to 10 feet deep and about 15 to 20 feet wide at the top of the bank. The creek banks are steeply inclined and covered with a dense growth of small trees and brush. The location of the proposed Phelps Creek Bridge is also shown on Plate 1.

We anticipate the proposed Phelps Creek Bridge will be similar to the Devereux Creek Bridge and will be designed and constructed by CONTECH Inc. However, the bridge will have a 48-foot span and the bridge soffit will be about 13 feet above the creek bottom. The sides and bottom of the channel for the Phelps Creek Bridge will be left in an undisturbed condition. The proposed bridge layout is shown on Plate 2b – Exploration Location Plan – Phelps Bridge.

2.0 WORK PERFORMED

2.1 PURPOSE

The purpose of this report is to characterize the subsurface conditions and provide geotechnical recommendations for grading and design of the proposed foundations for the new Con/Span bridges.

2.2 SCOPE

We performed the following scope of work to evaluate the geotechnical considerations for the project:

- Site visits to observe the general site conditions and coordinate the field exploration program;
- Field exploration consisting of excavating two hollow-stem-auger drill holes and advancing two cone penetration test (CPT) soundings at each of the two bridge sites. Subsurface explorations were advanced to depths ranging from about 35 feet and 70 feet;
- Laboratory testing of selected samples obtained from the drill holes;
- Preparing this report summarizing the geotechnical data obtained for the project, and our opinions and recommendations regarding;
    - Soil and groundwater conditions encountered;
    - Local geology, faulting, and seismicity;
    - Potential for geologic hazards such as ground shaking, ground rupture and liquefaction;
    - Foundation support, grading, embedment, settlement, and lateral earth pressures for the design of the Con/Span bridge system;
    - Construction considerations: need for dewatering, site preparation, temporary excavations, and shoring;
    - Suggested specifications for on-site or imported materials used as fill; and
    - Compaction and material requirements for on-site and imported fill and backfill materials.

2.3 FIELD EXPLORATION

The field exploration consisted of advancing two cone penetration test (CPT) soundings and drilling two soil borings at approximate locations of the proposed Con/Span bridge foundations. The locations of the explorations are shown on Plates 2a and 2b.

Details of the exploration program and logs of the CPT soundings and drill holes are provided in Appendix A – Field Exploration.

2.4 LABORATORY TESTING

Laboratory tests for unit weight, moisture content, compaction, and shear strength were performed as part of the laboratory testing program. Laboratory testing was performed in general accordance with the applicable standards of ASTM. Laboratory test results are presented in Appendix B – Laboratory Testing.

2.5 GENERAL CONDITIONS

Fugro prepared the conclusions, recommendations, and professional opinions of this report in accordance with the generally accepted geotechnical principles and practices at this time and location. This warranty is in lieu of all other warranties, either expressed or implied. This report was prepared for the exclusive use of the University of California Santa Barbara and their authorized agents only. It is not intended to address issues or conditions pertinent to other parties, projects or for other uses. The report and the drawings contained herein are not intended to act as construction drawings or specifications. Explorations and services have not been requested nor performed to assess the presence or absence of hazardous, toxic, or biological materials.

Our characterization of the subsurface conditions is based on explorations performed at specific locations, and the interpolation and extrapolation of data between points of exploration and testing. The boundaries and extent of the subsurface conditions described are approximate, and transitions can be gradual. The subsurface soil and groundwater conditions will vary between points of exploration and observation, may change with time, and should be reviewed based on the conditions revealed by construction.

3.0 SITE CONDITIONS

3.1 GEOLOGIC SETTING

The proposed Devereux Creek and Phelps Creek bridges are located within the western portion of the Transverse Ranges Province between the Pacific Ocean on the south and the Santa Ynez Mountains on the north. The Transverse Ranges Province is locally dominated by the east-west trending Santa Ynez Mountain Range, the most prominent mountain range in the coastal zone of Santa Barbara County. The mountain range extends almost continuously from Point Arguello eastward for 75 miles into Ventura County. The Santa Ynez Mountains and adjacent lowlands are composed almost entirely of Holocene to Eocene age sedimentary rocks.

The predominant geologic units at the Devereux Creek Bridge site consist of artificial fill (Af), estuary deposits (Qe), older alluvium (Qoal), and claystone bedrock of the

Pico Formation (Tp). The significant geologic units at the Phelps Creek bridge site are older alluvium (Qoal), marine terrace deposits (Qmt), and Pico Formation Bedrock (Tp). More detailed descriptions of those materials are provided below in Section 2.3.

Regional compressive forces on the coastal areas of Santa Barbara County have resulted in generally east-west trending folds and faults, typical of the Transverse Ranges Province. Many of the faults are regionally extensive and are considered active (Mualchin 1996, California Division of Mines and Geology 2002, Santa Barbara County 1991). The closest mapped faults to the project sites are the north and south branches of the More Ranch fault. Gurrola (2004) maps the South Branch of the More Ranch fault trending northeast-southwest roughly through the site of the Devereux Creek Bridge. Gurrola (2004) maps the North Branch of the More Ranch fault trending roughly east-west through the northern portion of the Ocean Meadows Golf Course about 600 feet south of the Phelps Creek bridge site. Additional information regarding local and regional faulting is provided in Fugro (2004a, 2004b).

3.2 SUBSURFACE CONDITIONS

3.2.1 Devereux Creek Bridge

Geotechnical Conditions. As discussed above, the general geotechnical conditions at the Devereux Creek Bridge site consist of a relatively thin layer of artificial fill underlain by estuarine deposits, older alluvium, and claystone bedrock of the Pico Formation. The artificial fill materials consist of about 8 feet of medium stiff lean clay and medium dense clayey sand. The artificial fill is underlain by estuarine deposits consisting of interbedded layers of soft to medium stiff lean clay and loose to dense clayey sand and silty sand. Those materials were encountered to depths of about 40 feet in the west abutment area (CPT-103 and DH-103) and to a depth of about 50 feet in the east abutment area (CPT-104 and DH-104). On the basis of the CPT, pocket penetrometer and triaxial test data, the undrained shear strength of the clayey estuarine soils is estimated to range from 300 psf to about 700 psf.

The estuarine deposits in the west abutment area (CPT-103 and DH-103) are underlain by highly weathered, poorly indurated, soft claystone bedrock of the Pico Formation. The undrained shear strength of the claystone bedrock is estimated to be about 3,000 to 4,000 psf. Pico Formation was encountered in this area to the maximum depth explored of about 55 feet below the existing ground surface.

In the east abutment area, the estuarine deposits are underlain by older alluvial soils consisting of stiff to very stiff fat to lean clay. On the basis of the CPT, pocket penetrometer and triaxial test data, the undrained shear strength of the clayey older alluvium is estimated to range from about 2,000 to 3,000 psf.

Groundwater Conditions. Groundwater was encountered in the drill holes excavated for the Devereux Creek Bridge at depths of about 17 feet below the existing ground surface. That depth corresponds to an elevation of about -7 feet. Very moist soils were encountered above this level and the measured 17-foot depth to groundwater may not represent a static groundwater level. Therefore, for construction planning purposes, we recommend the depth to groundwater be assumed equal to the elevation of the surface water in the creek channel (or the bottom of the channel if there is not surface water flow).

Soil moisture and groundwater conditions will vary seasonally and with fluctuations in rainfall, storm runoff, irrigation and other factors and the depth of groundwater at the time of construction could vary from the reported depth.

3.2.2 Phelps Creek Bridge

Geotechnical Conditions. Geotechnical conditions at the Phelps Creek Bridge site consist of older alluvium, marine terrace deposits, and claystone and siltstone bedrock of the Pico Formation. The older alluvial soils consist of stiff to very stiff sandy lean clay to medium dense clayey sand. The undrained shear strength of the clayey older alluvial soils is estimated to range from about 2,000 to 4,000 psf. The older alluvial soils were encountered to a depth of about 15 feet to 19 feet below the existing ground surface and are underlain by a 10-foot-thick layer of marine terrace deposits. The marine terrace deposits generally consist of very dense poorly graded fine to medium sand.

The marine terrace deposits are underlain by highly weathered, poorly indurated, soft claystone and siltstone bedrock of the Pico Formation. Pico Formation was encountered at the Phelps Creek Bridge site to the maximum depth explored of about 40 feet below the existing ground surface.

The conditions encountered in our explorations for the proposed bridge are similar to the conditions encountered in our study for UCSB's North Campus Housing project and additional information regarding the subsurface conditions in the Phelps Creek Bridge area is provided in Fugro (2004a).

Groundwater Conditions. Groundwater was encountered in the drill holes excavated for the Phelps Creek Bridge at depths of about 13.5 and 19 feet below the existing ground surface. This depth may not represent a static groundwater level and for construction planning purposes we recommend the depth to groundwater be assumed equal to the elevation of the surface water in the creek channel (or the bottom of the channel if there is not surface water flow).

Soil moisture and groundwater conditions will vary seasonally and with fluctuations in rainfall, storm runoff, irrigation and other factors and the depth of groundwater at the time of construction could vary from the reported depth.

4.0 GEOLOGIC HAZARDS

The project site is located within Seismic Zone 4 based on the California Building Code (2001). The More Ranch fault is the nearest mapped and controlling seismic source for the site. Gurrola (2004) maps the South Branch of the More Ranch fault trending roughly east-west through the proposed Devereux Creek Bridge site and the North Branch of the More Ranch fault trending roughly east west about 600 feet south of the Phelps Creek Bridge site. Data regarding previous fault studies for the North Branch and South Branch of the More Ranch fault are provided in Fugro (2002, 2004a, 2004b) and CFS (2000), respectively.

The fault is classified as a type "B" seismic source based on the building code criteria. On the basis of our characterization of the site seismicity, we recommend that the following values be used if the structures will be designed using code-based methods.

Table 1a. Code-Based Seismic Design Values – Devereux Bridge

California Building Code Chapter 16, Table Number	Seismic Parameter	Seismic Values for Design
16-I	Seismic Zone Factor (Z)	0.4
16-J	Soil Profile Type	S_E
16-Q	Seismic Coefficient (C_a)	$0.36N_a$
16-R	Seismic Coefficient (C_v)	$0.96N_v$
16-S	Near Source Factor (N_a)	1.3
16-T	Near Source Factor (N_v)	1.6
16-U	Seismic Source Type	B

Table 1b. Code-Based Seismic Design Values – Phelps Bridge

California Building Code Chapter 16, Table Number	Seismic Parameter	Seismic Values for Design
16-I	Seismic Zone Factor (Z)	0.4
16-J	Soil Profile Type	S_D
16-Q	Seismic Coefficient (C_a)	$0.44N_a$
16-R	Seismic Coefficient (C_v)	$0.64N_v$
16-S	Near Source Factor (N_a)	1.3
16-T	Near Source Factor (N_v)	1.6
16-U	Seismic Source Type	B

4.1 PROBABILISTIC SEISMIC HAZARD ANALYSIS

A project-specific probabilistic evaluation of strong ground shaking was considered beyond the scope of this study. However, as mentioned in Fugro (2004a, 2004b), Fugro performed a seismic hazard analyses for the proposed seismic upgrades to the UCSB Francisco Torres student housing complex located northeast of the intersection of Storke Road and El Colegio Road (about 1/3 and 1/2 mile from the Devereux and Phelps Creek Bridge sites, respectively). The probabilistic seismic analyses for that study (Fugro 2002), indicate that an earthquake with a 10 percent probability of exceedance in a 50-year exposure period could generate a peak horizontal ground acceleration of about 0.52g at the site.

4.2 GROUND RUPTURE POTENTIAL

As described above, the More Ranch fault is mapped proximal to the proposed bridge sites. Gurrola (2004) maps the North Branch about 600 feet south of the Phelps Creek Bridge site. Considering the distance from the site to the mapped trace fault and previous fault study data (Fugro 2004a), in our opinion, the potential for ground surface rupture at the Phelps Creek site is considered to be relatively low.

However, Gurrola (2004) maps the South Branch of the More Ranch fault roughly at the proposed bridge site and CFS (2000) map the fault in the immediate vicinity of the Devereux Creek Bridge site. Although the possibility of an earthquake occurring on the South Branch of the More Ranch fault during the design life of the bridge is probably remote, the potential for an event to occur resulting in displacement of the ground surface and significant damage to the bridge cannot be ruled out. Potential alternatives for addressing the potential for ground rupture on the Devereux Creek Bridge could consist of accepting the risk of significant damage to the bridge or relocating the proposed structure away from the mapped trace. In our opinion, the University of Santa Barbara California should assess the potential risks and benefits and make a determination on whether to relocate the structure or not. We note that a more detailed fault study (likely including additional CPT soundings) could potentially be performed to provide additional characterization of the potential for ground rupture at the site.

4.3 LIQUEFACTION

General. Liquefaction is described as the sudden loss of soil strength because of a rapid increase in soil pore water pressures due to cyclic loading during a seismic event. In order for liquefaction to occur, three general geotechnical characteristics are generally required: 1) groundwater must be present within the potentially liquefiable zone; 2) the potentially liquefiable soil must be granular and the grain size distribution should fall within a relatively specific range; and 3) the potentially liquefiable soil must be of low relative density. If those criteria are met and strong ground motion occurs, then those soils may liquefy, depending upon the intensity and cyclic nature of the strong ground motion.

Liquefaction that produces surface effects generally occurs in the upper 40 to 50 feet of the soil column, although the phenomenon can occur deeper than 100 feet.

Devereux Creek Bridge. At the Devereux Creek site, loose to dense granular soils were encountered in the estuarine deposits below the estimated depth to groundwater. Groundwater was encountered in our explorations generally at depths of about 17 feet below the ground surface in our explorations in this area (elevation about -7 feet).

Layers of medium dense to dense granular soil about 2 to 3 feet thick were encountered in the east abutment area (CPT-104 and DH-104) at depths of about 16 and 23 feet below the ground surface. On the basis of the measured cone penetration test tip resistance those layers are dense and, by inspection, the potential for liquefaction is considered to be relatively low.

The soil materials encountered in the west abutment area of the Devereux Creek Bridge generally consist of soft to medium stiff lean clay interbedded with layers of loose to medium dense clayey sand and silty sand (CPT-103 and DH-103) and a greater frequency of granular soil layers were encountered in this area. Layers of granular soil were encountered in the west abutment area between a depth of 15 and 30 feet and 36 and 42 feet below the ground surface. On the basis of the CPT data, those soils are generally medium dense and are considered to be susceptible to liquefaction.

We used version 6.3.1 of the program CPT Analyst (Larson 2004) to further evaluate the potential for liquefaction at the west abutment of the Devereux Bridge using the CPT data assuming a design earthquake equal to a 7.5 magnitude weighted earthquake event with a 10 percent probability of exceedance in 50 years (475-year return period). The CPT analyst program evaluates the potential factor of safety for triggering of liquefaction using the empirical procedures described in Youd et al. (2001).

On the basis of our evaluation, there are soil layers below the groundwater in the west abutment area that could liquefy under the design earthquake conditions. The thickness of potentially liquefiable layers (individual layer thicknesses) range from about 2 to 5 feet. The cumulative liquefiable layer thickness (summation of individual liquefiable layer thicknesses) in CPT-103 ranges from about 9 to 14 feet. The results of the analysis indicated that overall the granular sols layers between about 15 and 30 feet are susceptible to liquefaction. The granular soils between about 36 and 42 feet deep are also susceptible to liquefaction, but to a lesser extent. The consequences of liquefaction of those soils could consist of ground surface settlements of about 2 to 3 inches and downdrag loads on deep foundation elements.

From our evaluation of the CPT and drill hole data, the clayey older alluvial soils and Pico Formation bedrock at the site are not considered to be liquefiable.

Phelps Creek Bridge. Very dense granular terrace deposits were encountered at the Phelps Creek site between depths of about 15 and 30 feet below the existing ground surface. The cone tip resistance measured in the marine terrace deposits generally exceeded 400 tons per square foot (tsf), and based on those data, are not considered to be potentially liquefiable.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations presented in this report are based on our understanding of the project as presently planned, review of the referenced information, plans and published information, and geotechnical analyses.

5.1 SUMMARY OF FINDINGS

5.1.1 Devereux Creek Bridge

- Because the site is underlain by up to 40 to 50 feet of relatively soft, compressible estuarine deposits, in our opinion, the Con/Span bridge structure and pre-cast wing walls at Devereux Creek will likely require deep foundation support such as driven piles. Recommendations for driven piles are provided in Section 5.2.3.

- Liquefiable soils are present at the Devereux Creek site. Liquefaction of those soils during a significant earthquake could result in ground settlements of about 2 to 3 inches and downdrag loads on deep foundations.

- Soft soils and foundation subgrade conditions will be encountered in excavations and a working surface of gravel, sand-cement slurry or lean concrete will likely be required beneath structures and proposed fill areas.

- Control of surface water in the creek and groundwater in the foundation excavations should be anticipated to allow proper grading and construction of the pile caps and other improvements. Temporary shoring may be required adjacent to the creek channel depending on the depth of the proposed foundations.

- Select backfill will be required behind all abutment and wing walls.

5.1.2 Phelps Creek Bridge

- The Con/Span bridge structure and pre-cast wing walls at Phelps Creek can likely be supported on shallow spread footings bearing on compacted fill and the older alluvium or marine terrace deposits. Recommendations for shallow foundations for the Phelps Creek Bridge are provided in Section 5.2.1.

- A limited depth of older alluvium or marine terrace deposits will need to be over-excavated below the spread footings to provide a firm working surface for construction.
- Control of surface water in the creek and groundwater in the foundation excavations should be anticipated to allow proper grading and construction of the spread footing and temporary shoring may be required adjacent to the creek channel and in areas adjacent to existing public or private improvements.
- Select backfill will be required behind all abutment and wing walls.

5.1.3 Suggested Materials Specifications

The following materials are referenced in various sections of the report.

General fill material can consist of on-site soils free of organics, debris, trash or other unsuitable materials. Soils to be used as general fill should be at a water content suitable for placement and compaction.

General imported fill material brought to the site shall be free of organics, oversized rock (that is over 3 inches in diameter), trash and other debris, and other deleterious materials. General imported materials shall have an Expansion Index of no more than 40 and material placed within 3 feet of finished grade in pavement areas should have an R-Value of at least 15. General imported fill should be reviewed by the geotechnical engineer prior to being brought to the site.

Select imported backfill material placed as backfill behind the Con/Span bridge structures shall consist of imported material conforming to CONTECH Inc.'s criteria for abutment and wing wall backfill. For other uses, we recommend that structure backfill conform to criteria in Caltrans Standard Specifications for Structural Backfill, Section 19-3.06, and have a sand equivalent of at least 30.

Aggregate base shall consist of imported material conforming to Caltrans Standard Specifications for Class 2 aggregate base, Section 26-1.02A.

Drainage Material should consist of Class 2 permeable material, conforming to Section 68-1.025 of the Caltrans Standard Specifications. Class 1 materials could also be used provided they are used in conjunction with filter fabric or a separation geotextile.

Geocomposite Drain should consist of a manufactured plastic core not less than 0.25 inches thick with both sides integrally bonded to a layer of filter fabric that will provide a drainage void. The drain shall produce a flow rate through the drainage void of at least 10 gallons per minute per foot of width at a hydraulic gradient of 1.0 at maximum externally

Geotextile for separation (filter fabric) shall conform to the Caltrans Standard Specifications Section 88, Engineering Fabric. Filter fabric shall consist of a geotextile suitable for separation and conform to the requirements of the Caltrans Standard Specifications, Filter Fabric for Underdrains, Section 88-1.03.

Geogrid reinforcement used for subgrade stabilization shall consist of Tensar BX1100.

3-inch minus crushed rock used for subgrade stabilization shall consist of hard, durable crushed rock with 95 percent passing the 3" sieve and less than 50 percent passing 1" sieve, and less than 15 percent passing the No. 4 sieve

5.1.4 Grading for Structures

Phelps Creek Bridge. Prior to commencing grading operations, existing fills, soils containing debris, organics, or other unsuitable materials should be cleared from the construction area. The Con/Span structure and retaining walls are currently proposed to be supported by the underlying soils at a depth of about 10 feet below the existing grade. We recommend that the existing soils within a depth of about 2 feet of the foundation level be overexcavated and replaced with compacted aggregate base. The 2-foot overexcavation should extend at least 3 feet beyond the proposed footing limits.

The excavation should be dewatered and free from groundwater seepage and the excavation should be made in a manner that minimizes disturbance of the soils. The excavated subgrade should be cut as neat as possible and observed by Fugro prior to placing fill material. The project specifications should provide for deepening the excavation as needed to provide a stable surface for construction and fill placement.

Devereux Creek Bridge. Prior to commencing grading operations, existing fills, soils containing debris, organics, or other unsuitable materials should be cleared from the construction area. The Con/Span structure and retaining walls are currently proposed to be supported by the underlying soils at a depth of about 15 feet below the existing grade. However, we anticipate the foundation depth will be reduced if the abutment and wing walls are supported on deep foundations.

To provide a firm working surface for construction, we recommend that the existing soils within a depth of about 2 feet of the foundation level be overexcavated and replaced with compacted aggregate base, sand cement slurry, or lean concrete. The 2-foot overexcavation should extend at least 3 feet beyond the proposed footing limits. In fill and backfill areas, stabilization of the subgrade will likely require stabilization using a combination of geogrid reinforcement and 3-inch minus crushed rock.

The excavation should be dewatered and free from groundwater seepage and the excavation should be made in a manner that minimizes disturbance of the soils. The excavated subgrade should be cut as neat as possible and observed by Fugro prior to placing fill material. The project specifications should provide for deepening the excavation as needed to provide a stable surface for construction and fill placement.

5.1.5 Grading for Pavement Areas

Site Preparation. Fill placement and grading operations should be performed according to the recommendations of this report. To provide relatively uniform support for new pavement, we recommend that the existing soils be overexcavated to a depth of 1-foot below the bottom of the proposed structural section. Exposed subgrade should be compacted to a minimum of 90-percent relative compaction as determined by ASTM D1557.

Subgrade Stabilization. We recommend that the project specifications provide for stabilization of the subgrade in areas where exposed subgrade is wet and yielding (exhibiting pumping conditions). In our opinion, subgrade stabilization can likely be provided by placing geogrid reinforcement and 3" minus crushed rock over relatively undisturbed subgrade. The geogrid should be placed on the subgrade prior to placing the float rock. A layer of geotextile fabric should be placed over the top of the float rock to provide separation between the rock and the overlying compacted fill. To reduce the potential for disturbance of the subgrade during excavation, we suggest that the subgrade be excavated using tracked equipment.

Crushed rock used for subgrade stabilization should be placed in one lift and in a manner that construction traffic does not operate on the undisturbed subgrade. The rock should conform to the recommendations of this report. The depth of subgrade to be removed and replaced with crushed rock should be assessed by the contractor with input from the owner and Fugro personnel during site grading. However, for preliminary estimating purposes, the thickness of rock required for subgrade stabilization should be assumed to be about 12 to 18 inches.

5.1.6 Fill Placement

The fill and select backfill material should be placed and compacted to at least the minimum relative compaction recommended in this report. The moisture content of the fill should be within 2 percent of optimum and suitable to achieve the recommended compaction. Each layer of fill material should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and be replaced with properly compacted fill material, prior to placing the next layer. Rock, and other oversized material, greater than 4 inches in diameter, should be removed from the fill material being placed. Rocks should not be nested and voids should be filled with compacted material.

When the moisture content of the fill material is below that sufficient to achieve the recommended compaction, water should be added to the fill. While water is being added, the soil should be bladed and mixed to provide relatively uniform moisture content throughout the material. When the moisture content of the fill material is excessive, the fill material should be aerated by blading or other methods. Fill should be spread in lifts no thicker than approximately 8 inches prior to being compacted. Fill and backfill materials may need to be placed in thinner lifts to achieve the recommended compaction with the equipment being used.

Based on our drill holes and laboratory tests, the soils encountered within the likely depths of excavation consist of lean clay, clay with sand, and silty fine sand. As a result of the relatively high fines content of the clayey and silty soils encountered, there is a potential that the on-site soils could be sensitive to changes in moisture content. Control of moisture content and compaction layer thickness will likely be necessary to achieve the recommended compaction.

5.1.7 Backfill and Compaction

Fill placement and grading operations should be performed in accordance with the grading recommendations in this report. CONTECH Inc., specifications may have more stringent compaction and placement requirements for the satisfactory performance of their structure. We recommend that fill materials be compacted to at least 90 percent relative compaction, as determined by the latest approved edition of ASTM D1557, unless a higher degree of compaction is otherwise recommended. We recommend the minimum relative compaction be provided for the locations indicated on the following table:

Table 2. Fill Compaction Criteria

Location	Recommended Minimum Relative Compaction
General	90 percent U.O.N.
Select backfill material and materials placed within 12 inches of finished subgrade in pavement areas	95 percent
Select backfill material placed below spread footings and behind retaining structures	95 percent
Aggregate base placed below spread footings	95 percent

U.O.N. = unless otherwise noted

5.2 CON/SPAN BRIDGE STRUCTURE DESIGN

5.2.1 Shallow Foundation Design

General. In our opinion, the Con/Span bridge proposed at Phelps Creek can be supported on shallow foundations. Shallow foundations for the bridge abutment and wing walls should be supported on 2 feet of compacted aggregate base and underlain by firm, undisturbed older alluvial soils or marine terrace deposits. The subgrade should be prepared as described in Section 5.1.4. The spread footings should be designed using a maximum allowable bearing pressure of 3,000 pounds per square foot (psf). Based on the preliminary design and foundation loads provided to us by CONTECH Inc., footings for the proposed bridge will be about 10 feet wide assuming a 48-foot-wide span designed for an allowable bearing capacity of 3,000 psf and 3 feet of soil cover.

We recommend that retaining wall foundations bearing on material prepared according to the recommendations of this report be designed using a maximum allowable bearing pressure of 3,000 pounds psf. The footing should be at least 3 feet wide and be embedded at least 2.5 feet below adjacent ground surface or finished grade. For retaining wall footing design, the toe pressure can exceed the recommended maximum allowable bearing pressure provided the resultant force acts within the middle third of the footing. The maximum allowable average and toe bearing pressures can be increased by one-third when considering short-term wind or seismic loads.

We estimate that settlements resulting from static foundation loads should generally be less than approximately 1-inch total and approximately 1/2-inch differential in 30 feet along the bridge abutment or wing wall foundation.

5.2.2 Resistance to Lateral Loads

Sliding Resistance. Sliding friction acting on the base of the spread footing can provide resistance to lateral loading. Ultimate sliding resistance can be estimated using a coefficient of friction of 0.40 along the base of the footings.

Passive Pressure. Passive resistance developed from lateral bearing of below-grade walls or footings bearing against compacted fill (assumed to be select fill) can be estimated using a passive pressure corresponding to an equivalent fluid weight of 400 pcf above the groundwater and 200 pcf above the groundwater level.

Factor of Safety. The ultimate sliding resistance and passive pressure may be used together without reduction when evaluating overturning or sliding of foundations or below grade walls provided the design incorporates an appropriate factor of safety. Minimum factors of safety of 1.5 and 2.0 are recommended for foundation overturning and sliding, respectively when friction and passive pressure are used together.

5.2.3 Deep Foundation Design

Axial Capacity. In our opinion, the Con/Span bridge proposed at Devereux Creek should be supported on a deep foundation system consisting of driven piles. We have developed recommendations assuming the proposed bridge abutment and wing walls are supported on 14-inch square precast concrete piles. However, other pile sections could be used and recommendations for other pile sections could be developed, if requested, as additional services.

Design tip elevations for axial loading were estimated for 14-inch-square concrete piles. The capacity was evaluated from the frictional resistance and end bearing of the soil materials at the site using the computer program APILE Plus (Ensoft, 2004). The SHAFT program uses the methods and procedures described in FHWA (1993). For static conditions, resistance from shaft friction in the upper 15 feet was not considered in estimating the axial capacity. Groundwater was assumed to be about 10 feet below the top of the pile. For seismic or liquefaction conditions, the resistance from shaft friction was neglected in the upper 40 feet.

On the basis of information provided to us by CONTECH, Inc., we estimate that the foundation load for the proposed 42-foot-wide Con/Span bridge will be about 20 kips per foot. Our analyses indicate that a pile embedment length of about 65 to 75 feet is needed for a 14-in-square precast concrete pile to achieve an estimated static allowable capacity of 45 tons. The design pile tip elevations for a 45 ton vertical load are provided on Table 3 – Pile Data Table For Axial Loads.

Table 3. Pile Data Table for Axial Loads

Pile Location	Assumed Bottom of Pile Cap Elevation	Design Load (service load) (tons)	Nominal Resistance (tons)	Design Tip Elevation:	Specified Tip Elevation
East Abutment	el. +0	45	90 tons Compression 0 tons Tension (assumed)	el. -75[1]	el. -75
West Abutment	el. +0	45	90 tons Compression 0 tons Tension (assumed)	el. -65[1] el. -75[4]	el. -75

Design tip elevation is controlled by the following demands:
1 Compression; 2 Tension; 3 Lateral; 4 Liquefaction

Pile Spacing and Group Capacity. We anticipate that several piles will be required at each abutment location and that multiple rows of piles may also be required. Adjacent piles should be spaced no closer than 3 pile diameters measured center to center. Piles spaced closer than 3 pile diameters, a group efficiency factor of less than unity would likely be required and Fugro should provide additional pile input if closer pile spacing is used.

Settlement. We estimate that settlement for an isolated pile should not exceed one inch. Differential settlements between the abutments can be estimated as about half of the estimated total settlement.

Lateral Capacity. The lateral load capacity of driven 14-inch-square precast concrete pile was estimated using a soil resistance-pile deflection model (p-y analysis). Our analysis was performed using the computer program LPILE^{PLUS} (Ensoft, 2006).

Lateral loads, deflections, maximum bending moment, and shear values were estimated for isolated pile conditions (an isolated pile is one that is not affected by lateral loading from other nearby piles). The moment of inertia of the pile was assumed equal to one half the gross moment of inertia of pile section to accommodate the potential for cracking of the pile. The modulus of elasticity was assumed equal to that of plain concrete. Loading conditions involved only lateral and axial loads. No factor of safety was applied to the estimated loads or deflections.

Data from CONTECH Inc., indicates the lateral design loading for the abutment wall for the 42-foot Con/Span bridge is about 8 kips per foot. Lateral load analyses were performed to estimate lateral deflections and bending moments for a 14-inch-square precast concrete pile subjected to lateral loading. An axial load of 45 ton was assumed in the analyses. The results are summarized in Table 4 – Lateral Pile Capacity.

Table 4. Lateral Pile Capacity

Pile Type	Pile Head Fixity	Lateral Load (kips)	Head Displacement (in)	Maximum Bending Moment (ft-kips)	Estimated Depth of Fixity* (ft)
14-inch-precast concrete pile	Free	6	0.2	15	14
14-inch-precast concrete pile	Free	8	0.33	23	16
14-inch-precast concrete pile	Free	10	0.5	31	22
14-inch-precast concrete pile	Free	15	1.2	54	26
14-inch-precast concrete pile	Fixed	6	<0.1	17	15
14-inch-precast concrete pile	Fixed	8	0.1	25	15
14-inch-precast concrete pile	Fixed	10	0.15	33	17
14-inch-precast concrete pile	Fixed	15	0.3	57	20
14-inch-precast concrete pile	Fixed	20	0.6	83	25

* Assumed to equal the depth where there is no moment remaining in the pile.

Group effects may impact lateral load capacities when the pile-spacing-to-diameter ratio is low. The group efficiency is the ratio of lateral load capacity for the group divided by the lateral load for an isolated pile times the number of piles in the group. Group effects for closely spaced piles result in a reduction of the average lateral load per pile in a group compared to the lateral capacity of an isolated pile. Fugro should provide input for group effects for lateral loading after the pile layout has been determined.

Pile Driving Considerations. Piles should be driven and installed to the required penetration(s) in accordance with Section 49 of the Caltrans Standard Specifications (Caltrans 1999). Because the site conditions are relatively soft and long piles will likely be required for the project, we recommend that a test pile driving program be performed prior to ordering or delivery of the production piles to evaluate the effects of the site conditions, to aid in developing a driving criteria, to assess the selected pile hammer/driving system, and to better evaluate the variability in the pile length(s) required for the project. The contractor should be responsible for selecting the equipment to be used for pile driving and for achieving the required penetration with the pile remaining in good condition after driving.

Fugro should be contacted to provide observation and monitoring of the indicator test pile and production pile driving activities.

5.2.4 Lateral Earth Pressures

Con/Span bridge structure retaining walls (including pedestal walls) should be designed according to the recommendation of this report. We recommend the following equivalent fluid weights for use in estimating the lateral earth pressures that will act on retaining walls with level backfill conditions and active earth pressure conditions. Retaining wall backfill should consist of select backfill materials conforming to the recommendations of this report.

Table 5. Lateral Earth Pressures

Lateral Earth Pressure Distribution	Backfill Material	Equivalent Fluid Weight (pcf)
Active – Unbraced Drained	Select Backfill	35
Active – Unbraced Undrained	Select Backfill	80
At Rest – Braced Drained	Select Backfill	55
At Rest – Braced Undrained	Select Backfill	90

Drained values do not provide for hydrostatic forces (for example, standing water in the backfill material). If drainage cannot be provided behind the walls, undrained conditions should be assumed and used for design. Surcharge stresses from vehicle traffic can be estimated as a uniform surface load of 250 psf resulting in a uniform pressure on the wall of about 100 psf. If conditions (other than surcharge resulting from traffic loads) are anticipated, Fugro should be advised so we can provide additional recommendations as needed.

5.2.5 Retaining Wall Backfill and Drainage

As discussed above, for drained backfill conditions, drainage should be provided behind retaining walls to reduce the potential for the buildup of hydrostatic pressures. Retaining walls designed for drained loading conditions should be designed with weep holes or collector pipes to assist in the removal of water from the backfill, and to prevent the build up of hydrostatic pressures behind the wall.

Select backfill material should be placed within the critical backfill zone as indicated by Con/Span on either side of the structure or between the wall and a 1h:1v backslope projected up from the heel of the retaining wall footing, whichever is greater. If the design of the wall assumes no hydrostatic pressures (drained condition) acting on the wall, a continuous layer of drainage material consisting of either 1-foot of drainage material, or Geocomposite Drain panels should be provided along the backside of the wall. The drainage material should be terminated 2 feet below the finished grade of the wall backfill, and be topped with on-site soil or topsoil. Select backfill material and drainage materials should conform to the materials recommendations of this report.

5.3 CORROSION CONSIDERATIONS

Selected samples of the soils from the Devereux Creek and Phelps Creek sites were obtained in our study and tested for pH, resistivity, and chloride and sulfate content. The results of the test are provided in Appendix B and outlined below in Table 6 - Summary of Corrosion Test Results.

Table 6. Summary of Corrosion Test Results

Drill Hole No.	Sample Depth (ft)	Material Type	Resistivity (ohms/cm)	PH	Chloride (ppm)	Sulfate (wt %)
2	14	Older Alluvium (CL)	1180	8.1	Not Tested	Not Tested
4	16.5	Estuarine Deposits (CL)	130	8.4	<5	<0.0005
DH-10 (Fugro 2004a)	3 to 6	Older Alluvium (CL)	230	7.5	568	0.027

Corrosion. The above corrosion data suggest that the estuarine deposits at the Devereux Creek site have a very high potential for corrosion of buried ferrous metals. Data for the older alluvium at the Phelps Creek site indicate that those soils are non-corrosive to metals. However, data from Fugro (2004a) indicate the older alluvial soils are very corrosive to buried metal. Therefore, for design purposes, we recommend that the soils at the Phelps and Devereux Creek sites be considered corrosive to buried ferrous metals.

Cement Type. The data indicate the soils at the Devereux and Phelps Bridge sites are non-aggressive to cement. Therefore, concrete that is in contact with native materials can use type II cement.

5.4 CONSTRUCTION CONSIDERATIONS

5.4.1 Temporary Slopes

The design of temporary slopes and shoring systems needed for construction is the responsibility of the contractor. Temporary slopes should be braced or sloped according to the requirements of (Cal) OSHA. We expect the soil within the Phelps Creek area will generally consist of older alluvium, which classifies as a Type B soil based on (Cal) OSHA. Temporary slopes for the Phelps Bridge should be adequately dewatered.

Soil conditions at the Devereux Creek Bridge site will consist of artificial fill and estuarine deposits. These materials are moist to very moist and soft and generally classify as Type C soil based on (Cal) OSHA. We recommend temporary slopes excavated in the fill and estuarine soils be limited to a height of about 10 feet and that the soils be adequately dewatered prior to constructing the slope.

The need for temporary shoring should be evaluated by the contractor. In addition, the contractor should be responsible for design, nad installation of the shoring used on the project.

5.4.2 Dewatering

As discussed previously in this report, wet soil and groundwater seepage was encountered at a depth of approximately 13 feet at the Phelps Creek Bridge site and about 17 feet at the Devereux Creek site. The groundwater depth measured in our drill holes may not represent a static groundwater condition and the actual depth to groundwater may be less. Therefore, for construction planning purposes, we recommend the depth to groundwater be assumed at the level of surface water in the creek or at the bottom of the creek channel, whichever is higher. Data from our drill holes and CPT soundings (such as soil stratigraphy and material types) should be considered in the design of temporary dewatering systems.

We recommend that provisions be incorporated into the contract documents that address groundwater, seepage, and the potential for soft subgrade conditions to exist at the foundation level. The contractor should determine the means and methods for controlling groundwater at the site. Because proposed excavations are likely to extend below the groundwater, we recommend that the groundwater level be lowered as necessary to allow the required excavations to be free of groundwater seepage and provide for stable excavation slopes. For relatively minor seepage, local sumping with the placement of gravel in the bottom of the excavation and pumping of water that accumulates in the excavations may be sufficient. However, for more significant seepage, dewatering wells or dewatering trenches may be required to maintain relatively dry excavation conditions.

6.0 CONTINUATION OF SERVICES

The geotechnical evaluation consists of an ongoing process involving the planning, design, and construction phases of the project. To provide this continued service, we recommend that the geotechnical engineer be provided the opportunity to review the project plans and specifications, and observe portions of the site grading and fill placement during construction.

Subsurface conditions, excavations, foundations, and fill placement should be reviewed by the geotechnical professional during construction to evaluate if the subsurface conditions encountered, and construction methods used, are consistent with those assumed for design. The geotechnical professional should also review the project plans and specifications prior to construction. The purpose of the review is to evaluate if the plans and specifications were prepared in general accordance with the recommendations of this report.

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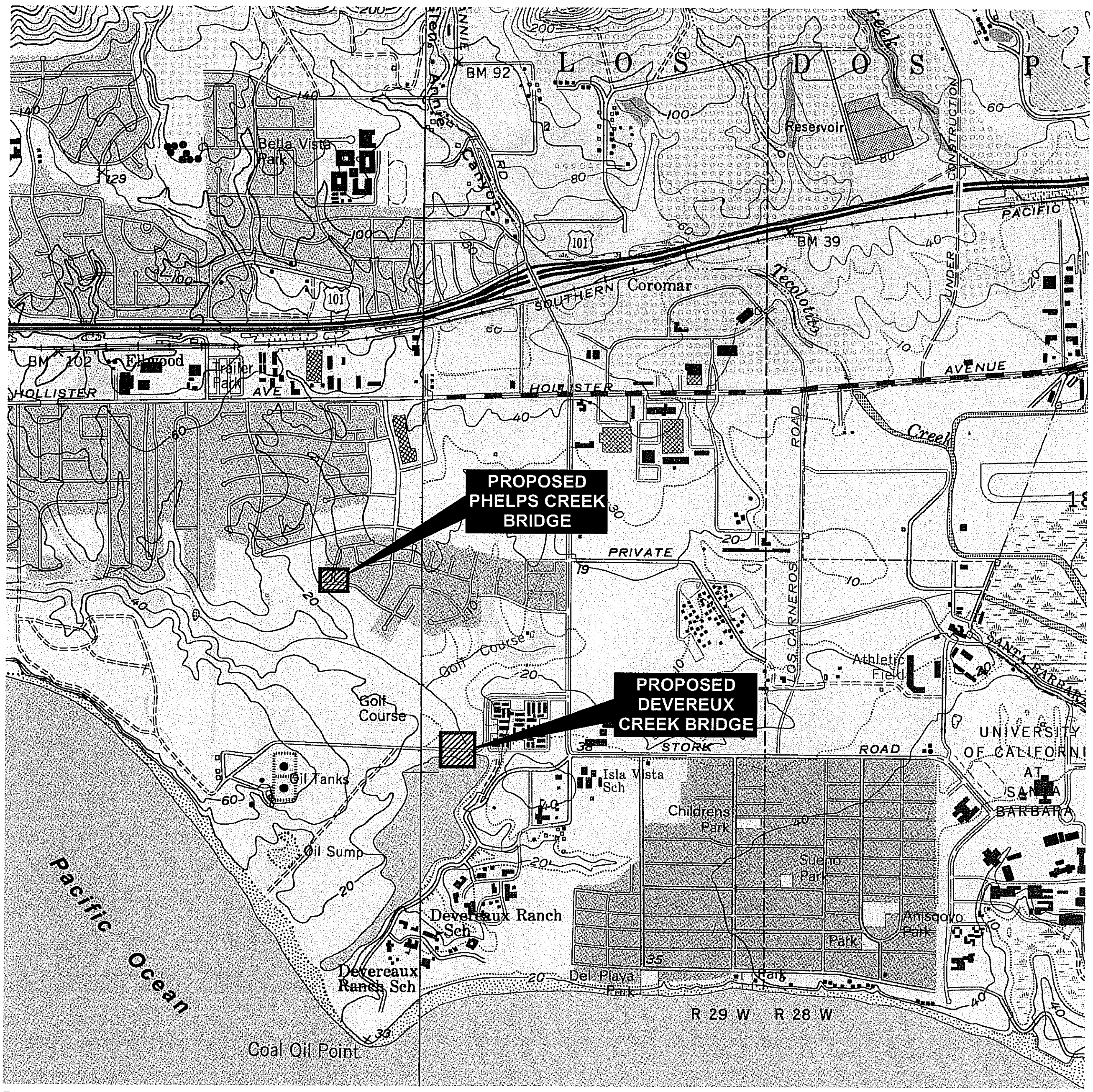

Base map sources: USGS 7.5' Dos Pueblos Canyon and Goleta, California quadrangle maps (photorevised 1967, 1982, & 1988).

NORTH
0 1000 2000
FEET

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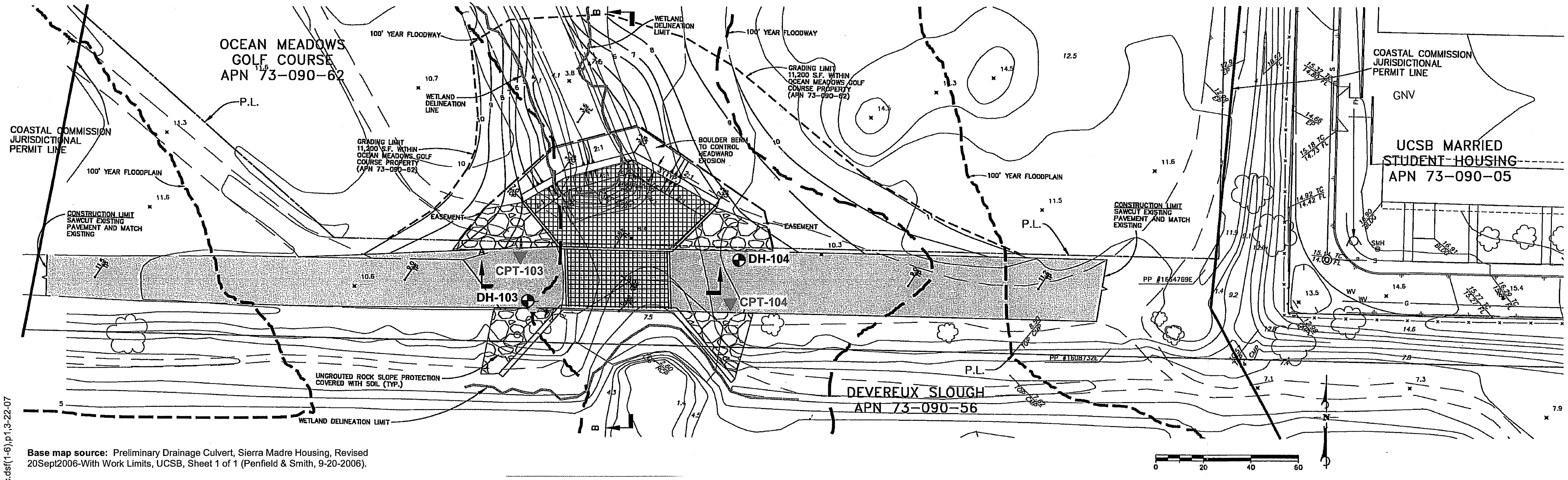
VICINITY MAP
Devereux and Phelps Creek Bridges
University of California Santa Barbara

PLATE 1



Base map source: Preliminary Drainage Culvert, Sierra Madre Housing, Revised 20Sept2006-With Work Limits, UCSB, Sheet 1 of 1 (Penfield & Smith, 9-20-2006).

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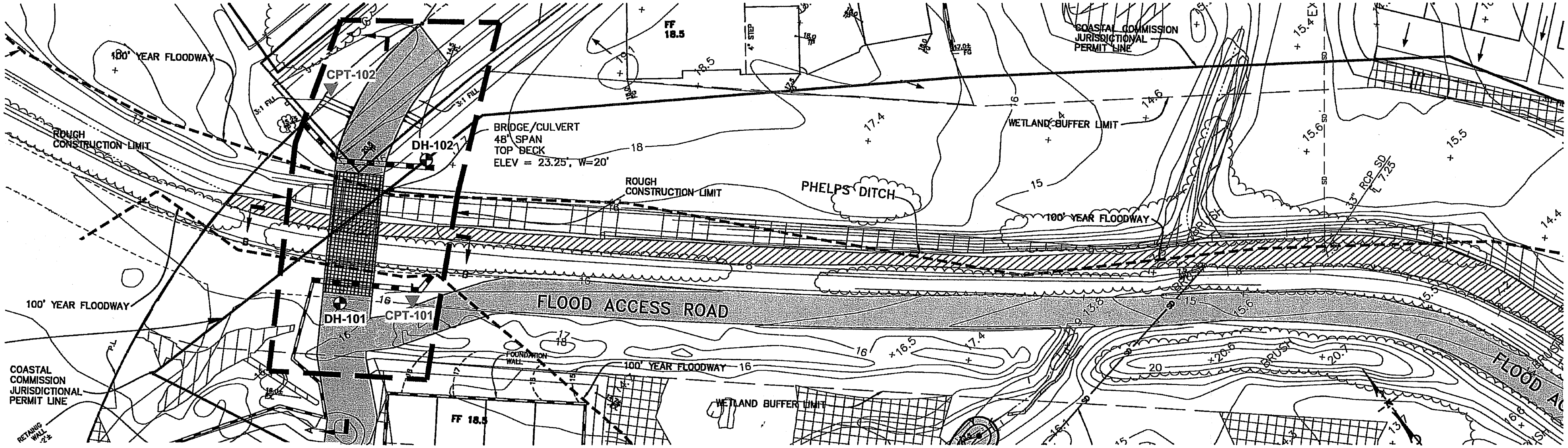
LEGEND

▼ CPT-102	Approximate location of cone penetration test
DH-102	Approximate location of boring

NORTH

0 40 80
FEET

EXPLORATION LOCATION PLAN - DEVEREUX BRIDGE
Devereux and Phelps Creek Bridges
University of California Santa Barbara



Base map source: Preliminary Drainage Culvert, North Campus Faculty Housing, UCSB, Sheet 1 of 1 (Penfield & Smith, 7-2006).

0 20 40 60

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LEGEND

CPT-102 Approximate location of cone penetration test

DH-102 Approximate location of boring

NORTH

0 40 80
FEET

EXPLORATION LOCATION PLAN - PHELPS CREEK BRIDGE
Devereux and Phelps Creek Bridges
University of California Santa Barbara

APPENDIX A
FIELD EXPLORATION

APPENDIX A
FIELD EXPLORATION

General

The field exploration for this project consisted of advancing four CPT soundings on February 6, 2007, and drilling four hollow-stem-auger drill holes on February 14 and 15, 2007. The field explorations were performed in general accordance with our proposal dated January 9, 2007.

Drill Holes

The drilling subcontractor used for this project was S/G Testing Laboratories of Lompoc, California. A truck-mounted CME 75 hollow-stem-auger drilling rig was used to excavate the drill holes. The drilling was performed under the observation of a Fugro staff engineer who prepared logs of the soil conditions encountered and obtained soil samples for laboratory observation and testing. The soils were classified in the field according to the Unified Soil Classification System. The drill holes were drilled to depths ranging from approximately 35 to 70 feet below the existing ground surface. The drill holes were backfilled with excavated soil cuttings.

Drive samples were obtained from the drill holes using either a modified California sampler or a standard penetration test split spoon sampler. The modified California sampler has a 3-inch outside diameter and a 2-3/8-inch inside diameter. The sampler was generally driven 12 inches into the material at the bottom of the drill hole using a 140-pound automatic trip hammer dropping 30 inches. The sampler contained 1-inch high brass rings. The number of blows needed to drive the sampler 12 inches into the soils was recorded and is shown on the Log of Drill Holes. Recovered samples were placed in transport containers and returned to the laboratory for further classification and testing.

Standard penetration tests (SPT) were performed at selected depths within the drill holes in general accordance with ASTM Standard Test Method D 1586. Soil samples obtained from the SPT were retained for further laboratory observation and testing. The SPT split spoon was generally driven approximately 18 inches into the material at the bottom of the hole using a 140-pound automatic trip hammer with a 30-inch drop. The number of blows required to drive the split spoon to three, 6-inch increments was recorded. The number of blows per foot (SPT N-Value) is equal to the sum of the last two 6-inch increments, and is reported on the Log of Drill Holes.

Bulk samples were collected during the course of drilling by taking cuttings obtained from the auger flights. The bulk samples were selected for classification and testing purposes and may represent a mixture of soils within the noted depths. Recovered samples were bagged and returned to the laboratory for further classification and testing.

The Logs of Drill Holes show the depths and descriptions of the conditions encountered, geologic structure where applicable, vertical locations of drive samples, results of density, moisture content and sieve tests, and plasticity index. Logs of the drill holes are provided on Plates A-1 through A-4. The logs represent the interpretation of field logs and tests, the interpolation of soil conditions between samples, and the results of laboratory tests performed. The noted stratification lines represent approximate boundaries between soil types; the transitions can be gradual. A legend to the drill hole logs is provided on Plate A-5 – Key to Terms and Symbols Used on Logs.

Cone Penetration Test (CPT) Soundings

The CPT soundings for this project were performed by Fugro Geosciences of Santa Fe Springs, California. Fugro used an approximately 20-ton truck equipped with a hydraulic ram to advance CPT soundings to depths of approximately 35 to 60 feet below the existing ground surface. The CPT profiles were performed using an electric cone penetrometer with a diameter of approximately 15 square centimeters.

Cone penetration resistance (q_c), and sleeve resistance (f_s) values were recorded nearly continuously during penetration. CPT data and soil classifications were used in conjunction with drill hole data to estimate soil boundaries encountered at the site. Logs of the CPT profiles are shown on Plates A-6 through A-9. A legend to the CPT logs is provided on Plate A-10 - Legend to CPT Sounding.

FUGRO

LOCATION: Phelps Creek, West Abutment

SURFACE EL: 16 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
	0	A		2" grass and thin layer of gravel for access road							
14	2			**OLDER ALLUVIUM (Qoal)** Sandy CLAY (CL): stiff, reddish brown, moist, low plasticity, fine to coarse sand with trace v. fine sand			16				
12	4	1	14				16				
10	6										
8	8			Sandy CLAY (CL)/Clayey Sand (SC): stiff/medium dense, light reddish brown, moist, very fine sand, mottled with CL/SC light grey, lean gravelly clay							
6	10	2a 2b	(25)		130	108	21				p 3.5 u 2.1
4	12										
2	14	3a 3b	(32)	Lean CLAY with gravel (CL): very stiff, brown and gray mottled, moist, moderate plasticity, fine to coarse gravel	122 131	98 108	24 22		47	28	u 4.0 p 4.0
0	16										
-2	18			**MARINE TERRACE DEPOSITS (Qmt)** Poorly-graded SAND (SP): very dense, light gray, wet, fine to medium sand (5-10% fines)							
-4	20	4	33				24	5			
-6	22										
-8	24			flowing sand, unable to sample							
-10	26										
-12	28			**PICO FORMATION (Tp)** CLAYSTONE (Rx): soft, moderately weathered, poorly indurated, dark gray, massive, moist, trace fine sand							
-14	30										
-16	32			flowing sand, unable to sample							
-18	34	5a 5b	(62)		128	104	23				

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 35.0 ft
DEPTH TO WATER: 13.5 ft , first encountered
BACKFILLED WITH: Cuttings
DRILLING DATE: February 15, 2007

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Testing
LOGGED BY: J Hutchins
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-101

Phelps and Devereux Bridges
University of California Santa Barbara, California

PLATE A-1

FUGRO

LOCATION: Phelps Creek, East Abutment

SURFACE EL: 17 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
16		B		**OLDER ALLUVIUM (Qoal)** Sandy Lean CLAY with gravel (CL): very stiff, reddish brown and light gray mottled, moist, very fine sand, fine to coarse gravels							
14	2										
12	4	1a 1b	(42)		130	110	19		47	25	p 3.0
10	6										
8	8	2	15	gravel becomes less frequent			24				p 2.5
6	10										
4	12										
2	14	3a	(37)	Clayey SAND (SC): medium dense, reddish brown, moist to wet, lean clay, fine sand	129	116	11				
		3b		Sandy Lean CLAY with gravel (CL): very stiff, reddish brown and light gray mottled, moist, very fine sand, fine to coarse gravels	129	105	23	53			
0	16										
-2	18	4a 4b	42	**MARINE TERRACE DEPOSITS (Qmt)** Poorly-graded SAND with clay (SP-SC): dense, light gray, wet, fine to medium sand							
-4	20										
-6	22										
-8	24										
-10	26										
-12	28			**PICO FORMATION (Tp)** SILTSTONE (Rx): soft, moderately weathered, poorly indurated, gray, massive, moist, very fine sand							
-14	30										
-16	32										
-18	34	5	65				26				

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 35.0 ft
DEPTH TO WATER: 19.0 ft , first encountered
BACKFILLED WITH: Cuttings
DRILLING DATE: February 15, 2007

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Testing
LOGGED BY: J Hutchins
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-102

Phelps and Devereux Bridges
University of California Santa Barbara, California

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: Devereux Creek, West Abutment SURFACE EL: 10 ft +/- (rel. MSL datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
			C			**ARTIFICIAL FILL (af)** 4.5" AC, no base Lean CLAY (CL): medium stiff, brown, moist, trace fine sand and gravel							
8	2												
6	4		1a 1b		(13)	Clayey SAND (SC): loose, brown, moist, fine to medium sand, becomes more clayey on shoe	126	108	17	17			p 1.5
4	6												
2	8		2		(9)	**ESTUARINE DEPOSITS (Qe)** Fat CLAY (CH): medium stiff, dark gray, moist, pp 1.0	116	84	38				p 0.5
0	10					Sandy CLAY (CL): medium stiff, brown, moist, fine to medium sand							
-2	12												
-4	14												
-6	16		3a 3b		13	Poorly-graded SAND with clay (SP-SC): medium dense, gray, wet, fine to medium sand			25	5			
-8	18					Fat CLAY (CH): dark gray, moist, moderate plasticity, trace shells							
-10	20					increasing sand							
-12	22					Lean CLAY with sand (CL): soft, gray, wet, fine sand, moderate plasticity							
-14	24		4a 4b		(6)		124	94	32				p 0.5
-16	26												
-18	28		5a 5b		9	becomes medium stiff							
-20	30					Clayey SAND (SC): loose, gray, wet, fine to medium sand							
-22	32												
-24	34		6a 6b		(18)	Fat CLAY (CH): stiff, bluish gray and brown mottled, moist, moderate to high plasticity, trace organics	124	97	27		53	32	p 2.0

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 55.0 ft
DEPTH TO WATER: 18.0 ft , after drilling, 17 ft first encountered
BACKFILLED WITH: Cuttings
DRILLING DATE: February 14, 2007

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Testing
LOGGED BY: J Hutchins
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-103
Phelps and Devereux Bridges
University of California Santa Barbara, California

PLATE A-3a

FUGRO

LOCATION: Devereux Creek, West Abutment

SURFACE EL: 10 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
-26	36										
-28	38										
		7	13				27				
-30	40										
				OLDER ALLUVIUM (Qoal) Sandy Lean CLAY with gravel (CL): stiff, light brown, moist, very fine sand, fine to coarse gravel							
-32	42										
-34	44										
		8a 8b	(40)	**PICO FORMATION (Tp)** CLAYSTONE (Rx): soft, moderately weathered, poorly indurated, gray, massive, low to moderate plasticity, trace fine sand	129	104	24				p 4.0
-36	46										
-38	48										
-40	50										
-42	52										
-44	54										
		9a 9b	(70)		130	107	22				
-46	56										
-48	58										
-50	60										
-52	62										
-54	64										
-56	66										
-58	68										
-60	70										

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 55.0 ft
DEPTH TO WATER: 18.0 ft , after drilling, 17 ft first encountered
BACKFILLED WITH: Cuttings
DRILLING DATE: February 14, 2007

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Testing
LOGGED BY: J Hutchins
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-103

Phelps and Devereux Bridges
University of California Santa Barbara, California

PLATE A-3b

FUGRO

LOCATION: Devereux Creek, East Abutment

SURFACE EL: 11 ft +/- (rel. MSL datum)

ELEVATION, ft: 10, 8, 6, 4, 2, 0, -2, -4, -6, -8, -10, -12, -14, -16, -18, -20, -22, -24

DEPTH, ft: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34

SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
D 1		**ARTIFICIAL FILL (af)** 3" AC, no base, hand auger upper 1', layer of stabilization fabric			20				
		Lean CLAY (CL): medium stiff, brown mottled, moist, trace fine sand lenses							
2a 2b 2c	(9)	Clayey SAND (SC)/Sandy Clay (SC): loose, brown, moist, fine to medium sand	126	106	20 19	 38			p 1.0
3a	(8)	**ESTUARINE DEPOSITS (Qe)** Fat CLAY (CH): medium stiff, gray, moist, moderate to high plasticity	106	76	39				p 1.0
3b		Silty CLAY with sand (CL-ML): medium stiff, brown, moist, very fine sand	120	90	34				u 1.0
4	5	Fat CLAY (CH): medium stiff, gray, moist, moderate to high plasticity, becomes more sandy and wet with depth			44		74	52	p 0.5
5	25	Silty SAND (SM): medium dense, gray, wet, fine to medium sand, minor clay			24	15			
		Fat CLAY (CH): soft, gray, wet, moderate to high plasticity							
6	10	Clayey SAND (SC): loose, gray, wet, fine to medium sand							p 0.3
7	Push	Lean CLAY (CL)/Fat CLAY (CH): very soft, dark gray, moist, moderate plasticity, very fine sand			38				

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 70.0 ft
DEPTH TO WATER: 19.0 ft , after drilling, 17 ft first encountered
BACKFILLED WITH: Cuttings
DRILLING DATE: February 14, 2007
Hand auger upper 1'

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Testing
LOGGED BY: J Hutchins
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-104

Phelps and Devereux Bridges
University of California Santa Barbara, California

PLATE A-4a

FUGRO

LOCATION: Devereux Creek, East Abutment

SURFACE EL: 11 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
-26	36			Lean CLAY (CL)/Fat CLAY (CH): very soft, dark gray, moist, moderate plasticity, very fine sand							
-28	38										
	40	8a 8b	(8)	Fat CLAY (CH): soft, dark gray, moist, moderate to high plasticity	112 119	78 87	43 36				u 0.7
-30	42										
-32	44										
-34	46										
-36	48										
-38	50	9a 9b	(16)	becomes medium stiff with trace very fine sand, moderate plasticity, possible older alluvium (Qoal)	132	108	22		31	15	
-40	52										
-42	54			**OLDER ALLUVIUM (Qoal)** Lean CLAY (CL): stiff, mottled, gray with brown, moist, with trace fine sand and iron oxide staining							
-44	56										
-46	58										
-48	60	10a 10b	(22)		125	95	31				p 2.0 u 2.7
-50	62										
-52	64	11a 11b	(30)	Fat CLAY (CH): very stiff, dark gray, moist, moderate to high plasticity, trace very fine sand							
-54	66										
-56	68			Lean CLAY (CL): stiff, mottled, gray with brown, moist, trace fine sands							
-58	70	12a 12b	(25)		126	100	26				p 4.0

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 70.0 ft
DEPTH TO WATER: 19.0 ft , after drilling, 17 ft first encountered
BACKFILLED WITH: Cuttings
DRILLING DATE: February 14, 2007
Hand auger upper 1'

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Testing
LOGGED BY: J Hutchins
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-104

Phelps and Devereux Bridges
University of California Santa Barbara, California

PLATE A-4b

FUGRO

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	BLOW COUNT REC"/DRIVE"	MATERIAL DESCRIPTION	
-12	2	1	25	Well graded GRAVEL (GW)	COARSE GRAINED
-14	4	2	(25)	Poorly graded GRAVEL (GP)	COARSE GRAINED
-16	6			Well graded SAND (SW)	COARSE GRAINED
-18	8	3	(25)	Poorly graded SAND (SP)	COARSE GRAINED
-20	10	4	(25)	Silty SAND (SM)	COARSE GRAINED
-22	12	5	18"/ 30"	Clayey SAND (SC)	COARSE GRAINED
-24	14	6		Silty, Clayey SAND (SC-SM)	COARSE GRAINED
-26	16			Elastic SILT (MH)	FINE GRAINED
-28	18	7		SILT (ML)	FINE GRAINED
-30	20	8	20"/ 24"	Silty CLAY (CL-ML)	FINE GRAINED
-32	22	9	(25)	Fat CLAY (CH)	FINE GRAINED
-34	24			Lean CLAY (CL)	FINE GRAINED
-36	26	10	30"/ 30"	CONGLOMERATE	ROCK
-38	28	11	20"/ 24"	SANDSTONE	ROCK
-40	30	12		SILTSTONE	ROCK
-42	32	13		MUDSTONE	ROCK
-44	34			CLAYSTONE	ROCK
-46	36			BASALT	ROCK
-48	38			ANDESITE BRECCIA	ROCK
				Paving and/or Base Materials	ROCK

LOCATION: The drill hole location referencing local landmarks or coordinates

SURFACE EL: Using local, MSL, MLLW or other datum

General Notes

Soil Texture Symbol

Sloped line in symbol column indicates transitional boundary

Samplers and sampler dimensions (unless otherwise noted in report text) are as follows:

Symbol for:

1. SPT Sampler, driven 1-3/8" ID, 2" OD
2. CA Liner Sampler, driven 2-3/8" ID, 3" OD
3. CA Liner Sampler, disturbed 2-3/8" ID, 3" OD
4. Thin-walled Tube, pushed 2-7/8" ID, 3" OD
5. Bulk Bag Sample (from cuttings)
6. CA Liner Sampler, Bagged
7. Hand Auger Sample
8. CME Core Sample
9. Pitcher Sample
10. Lexan Sample
11. Vibracore Sample
12. No Sample Recovered
13. Sonic Soil Core Sample

Sampler Driving Resistance

Number of blows with 140 lb. hammer, falling 30" to drive sampler 1 ft. after seating sampler 6"; for example,

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
86/11"	After driving sampler the initial 6" of seating, 36 blows drove sampler through the second 6" interval, and 50 blows drove the sampler 5" into the third interval
50/6"	50 blows drove sampler 6" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

Classification of Soils per ASTM D2487 or D2488

Geologic Formation noted in bold font at the top of interpreted interval

Strength Legend

Q = Unconfined Compression
u = Unconsolidated Undrained Triaxial
t = Torvane
p = Pocket Penetrometer
m = Miniature Vane

Water Level Symbols

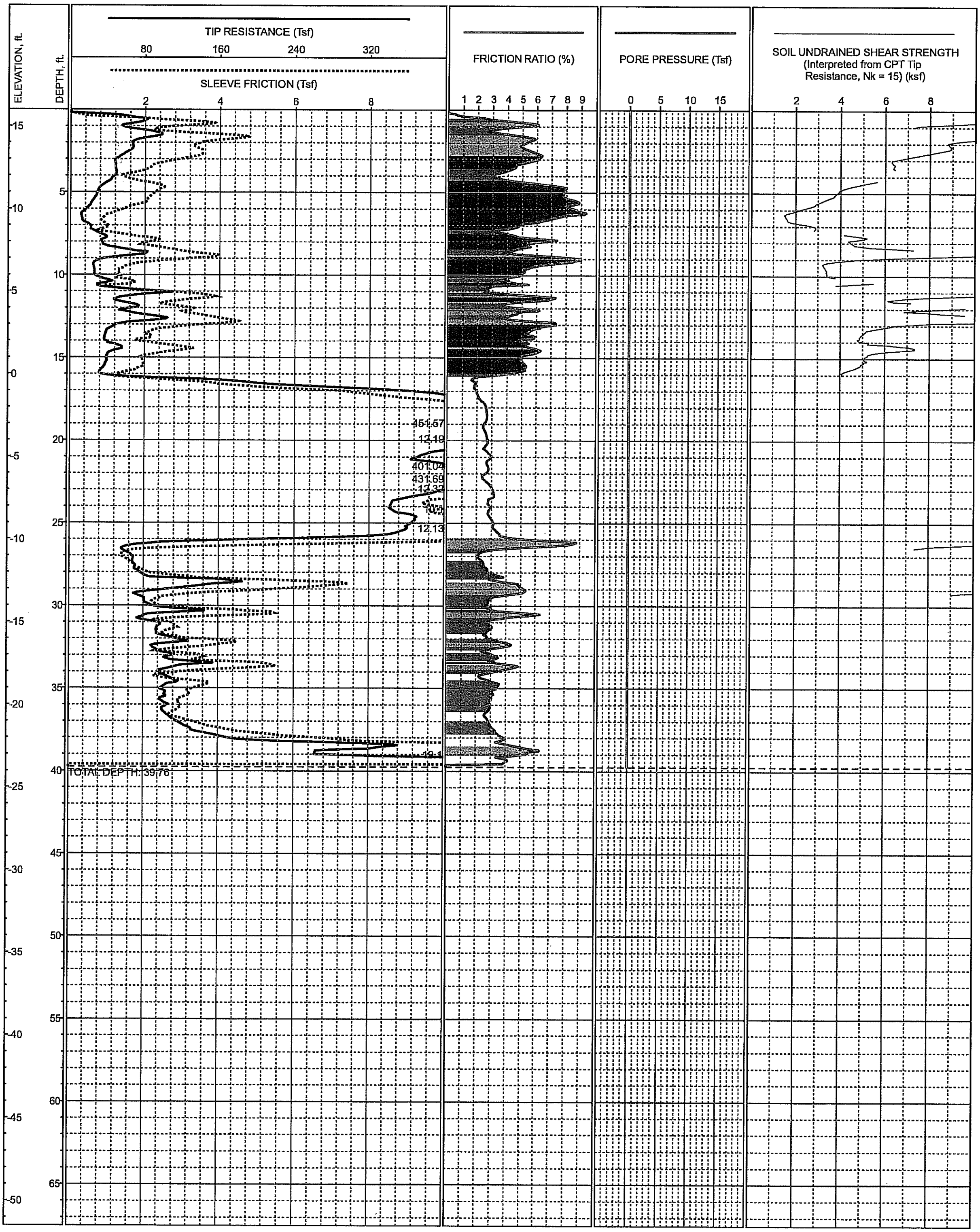
▽ Initial or perched water level
▼ Final ground water level
Seepages encountered

Rock Quality Designation (RQD) is the sum of recovered core pieces greater than 4 inches divided by the length of the cored interval.

KEY TO TERMS & SYMBOLS USED ON LOGS

PLATE A-5

FUGRO



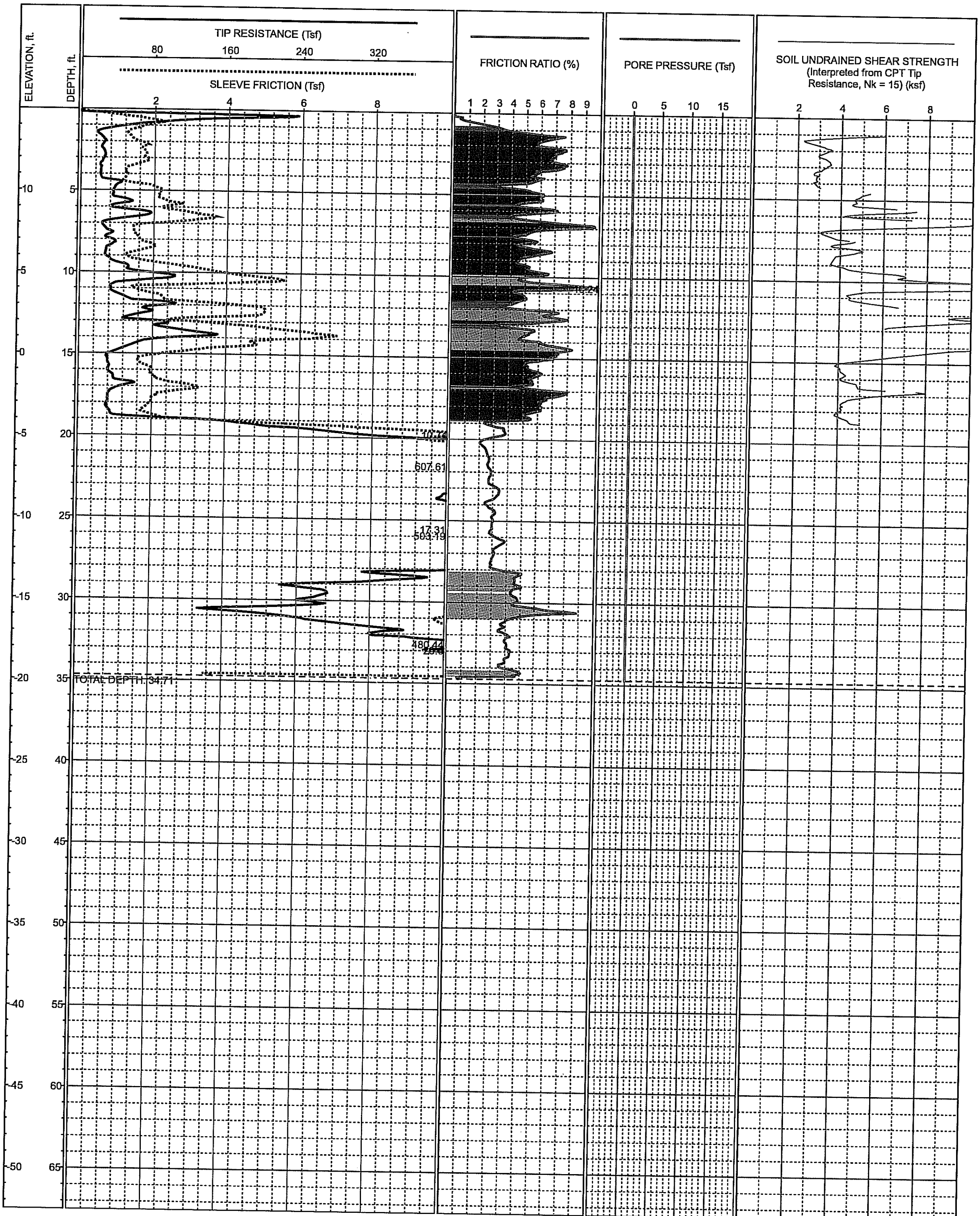
LOCATION: Phelps Creek, West Abutment
SURFACE EL: 16ft +/- (MSL)
COMPLETION DEPTH: 39.76ft
TESTDATE: 2/6/2007

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: G S Denlinger

LOG OF CPT

CPT-101

Devereux and Phelps Creek Bridges, University of California, Santa Barbara



LOCATION: Phelps Creek, East Abutment
SURFACE EL: 15ft +/- (MSL)
COMPLETION DEPTH: 34.71ft
TESTDATE: 2/6/2007

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: G S Denlinger

LOG OF CPT

CPT-102

Devereux and Phelps Creek Bridges, University of California, Santa Barbara

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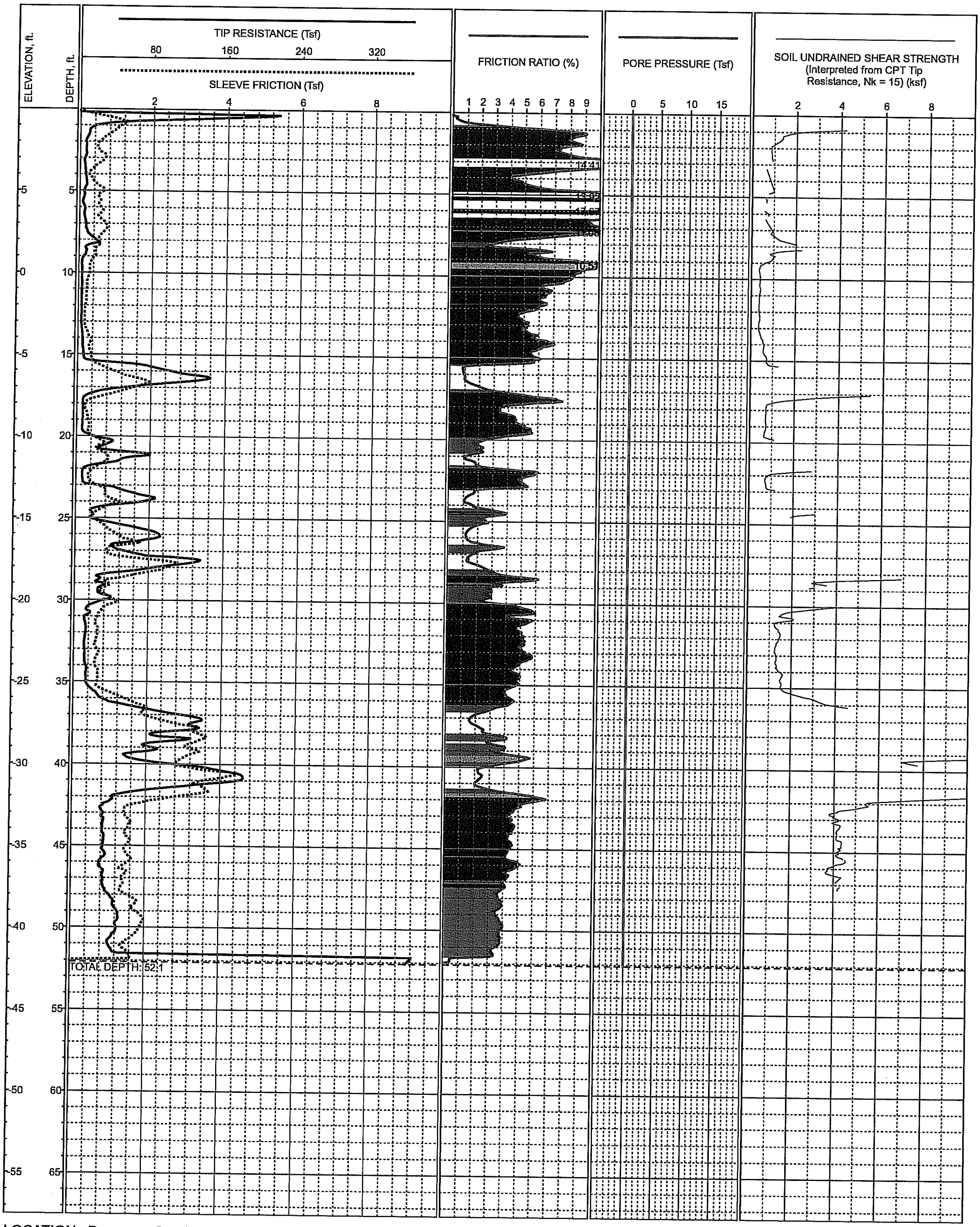

LOCATION: Devereux Creek, West Abutment
SURFACE EL: 10ft +/- (MSL)
COMPLETION DEPTH: 52.1ft
TESTDATE: 2/6/2007

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: G S Denlinger

LOG OF CPT

CPT-103

Devereux and Phelps Creek Bridges, University of California, Santa Barbara

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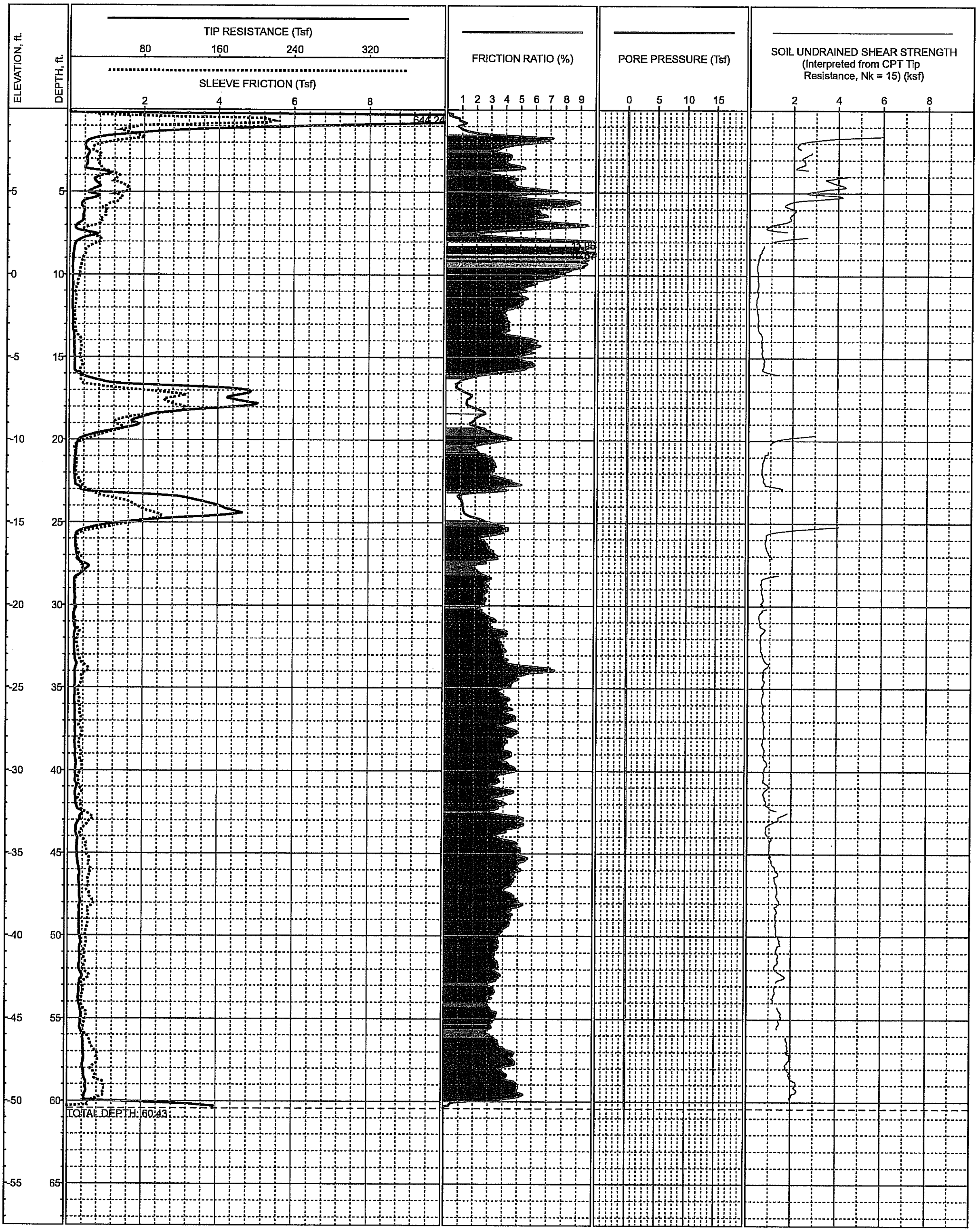


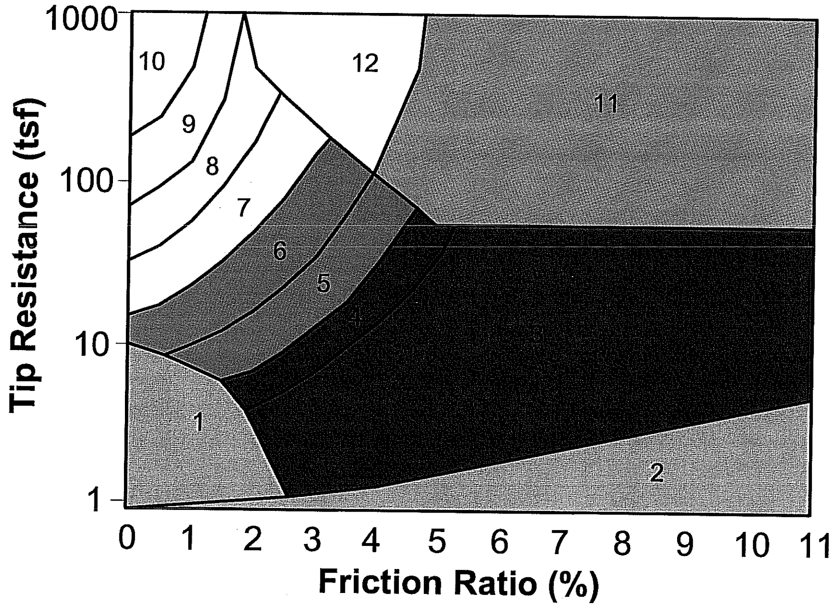
LOCATION: Devereux Creek, East Abutment
SURFACE EL: 10ft +/- (MSL)
COMPLETION DEPTH: 60.43ft
TESTDATE: 2/6/2007

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: G S Denlinger

LOG OF CPT

CPT-104

Devereux and Phelps Creek Bridges, University of California, Santa Barbara



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

CPT CORRELATION CHART
(Robertson and Campanella, 1984)

KEY TO CROSS SECTIONS

Devereux and Phelps Creek Bridges
University of California, Santa Barbara

N:\Projects\3064_UCSB\3064-045_CreekBridges\Outputs\Working\mxd\Explorations\CPTkey-1-8-07.mxd, 03/06/07, jdecker

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

This appendix discusses the results of the laboratory testing program performed for this geotechnical study. Laboratory tests were performed on selected samples obtained from the field to help classify the soils encountered and to estimate some of their engineering properties. The program was carried out employing, wherever practical, test procedures of the American Society for Testing and Materials (ASTM).

Driven-ring, SPT, and bulk samples used in the laboratory testing program were obtained from various locations during the course of the field exploration, as discussed in Appendix A. Each sample is identified by sample number and depth. The sample depth refers to the depth to the bottom of the hole prior to sampling. The various laboratory tests performed are described below. A summary of the laboratory tests performed on selected samples is presented on Plate B-1 - Summary of Laboratory Test Results.

Laboratory Moisture and Density. Moisture content and dry density tests were performed on selected driven samples obtained during the field exploration to evaluate the natural moisture content and dry density of the various soil encountered. The results are presented on Plate B-1 and the drill hole logs.

Percent Finer than 75μm. Tests for fines content or percent finer than 75μm were made for selected soil samples in general accordance with ASTM C117. The test results are tabulated on Plate B-1 and the drill hole logs.

Atterberg Limit Tests. Tests for liquid limit, plastic limit, and plasticity index were conducted on selected samples in general accordance with ASTM Test Method D4318. The Atterberg limit test results are presented on Plate B-2 - Plasticity Chart and Plate B-1.

Consolidation. Two one-dimensional consolidation tests were performed on selected driven-ring samples of clayey estuarine deposits. The samples were incrementally loaded to 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, 16.0, and 32.0 kips per square foot (ksf). The samples were allowed to consolidate under each load increment and water was added to the samples at a loading pressure of 0.25 ksf. Results of the consolidation tests are presented on Plate B-3 - Consolidation Test Results.

Unconsolidated Undrained Triaxial Compression Tests. Triaxial compression tests were performed on selected ring-driven samples of the clayey estuarine deposits from the Devereux Creek site and the older alluvium from the Phelps Creek site. The tests were performed in general accordance with ASTM D2850. Undrained shear strength values estimated from the triaxial compression tests is provided on Plate B-1 and the drill hole logs.

Chemical (Corrosion) Tests. Corrosivity tests for resistivity, pH, chloride and sulfate content were performed on a selected samples obtained from the drill holes.

Corrosivity and resistivity were estimated according to California Tests 532 and 643. The results of the corrosion testing are provided on Plate B-1.

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS LL	PI	COMPACTION TEST MAX DD pcf	OPT MC %	DIRECT SHEAR C ksf	PHI deg	COMPRESSIVE STRENGTH TESTS Qu, ksf	S_u (Cell Prs.) ksf	CORROSIVITY TESTS R	pH	Cl	So_4 (%)	R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	SPECIFIC GRAVITY
DH-101	1.0	A	Lean CLAY (CL)			16																	
DH-101	4.0	1	Sandy Lean CLAY (CL)			16																	
DH-101	9.5	2b	Lean CLAY (CL)	130	108	21									2.1(1.1)								
DH-101	14.0	3a	Lean CLAY (CL)	122	98	24									4(1.7)								
DH-101	14.5	3b	Lean CLAY (CL)	131	108	22		47	28														
DH-101	19.0	4	Poorly-graded SAND (SP)			24	5																
DH-101	34.5	5b	CLAYSTONE (Rx)	128	104	23																	
DH-102	4.5	1b	Sandy Lean CLAY (CL)	130	110	19		47	25														
DH-102	9.0	2	Sandy Lean CLAY (CL)			24																	
DH-102	14.0	3a	Clayey SAND (SC)	129	116	11										1180	8.10						
DH-102	14.5	3b	Sandy Lean CLAY (CL)	129	105	23	53																
DH-102	34.0	5	SILTSTONE (Rx)			26																	
DH-103	4.5	1b	Silty SAND (SM)	126	108	17	17																
DH-103	9.0	2	Fat CLAY (CH)	116	84	38																	
DH-103	16.5	3a	SAND with CLAY (SC)			25	5																
DH-103	24.5	4b	Lean CLAY (CL)	124	94	32																	
DH-103	34.5	6b	Fat CLAY (CH)	124	97	27		53	32														
DH-103	39.0	7	Fat CLAY (CH)			27																	
DH-103	44.5	8b	CLAYSTONE (Rx)	129	104	24																	
DH-103	54.5	9b	CLAYSTONE (Rx)	130	107	22																	
DH-104	1.0	1	Lean CLAY (CL)			20																	
DH-104	4.0	2a	Lean CLAY (CL)	126	106	20																	
DH-104	5.0	2c	Clayey SAND (SC)/Sandy CLAY (CL)			19	38																
DH-104	9.5	3a	Fat CLAY (CH)	106	76	39																	
DH-104	10.0	3b	Lean CLAY (CL)	120	90	34									1(1.2)								
DH-104	16.5	4	Fat CLAY (CH)			44		74	52							130	8.40	<5	<0.0005				
DH-104	18.0	5	Silty SAND (SM)			24	15																
DH-104	23.5	6	Clayey SAND (SC)																				
DH-104	29.5	7	Lean CLAY (CL)			38																	
DH-104	39.5	8a	Fat CLAY (CH)	112	78	43									0.7(4.8)								

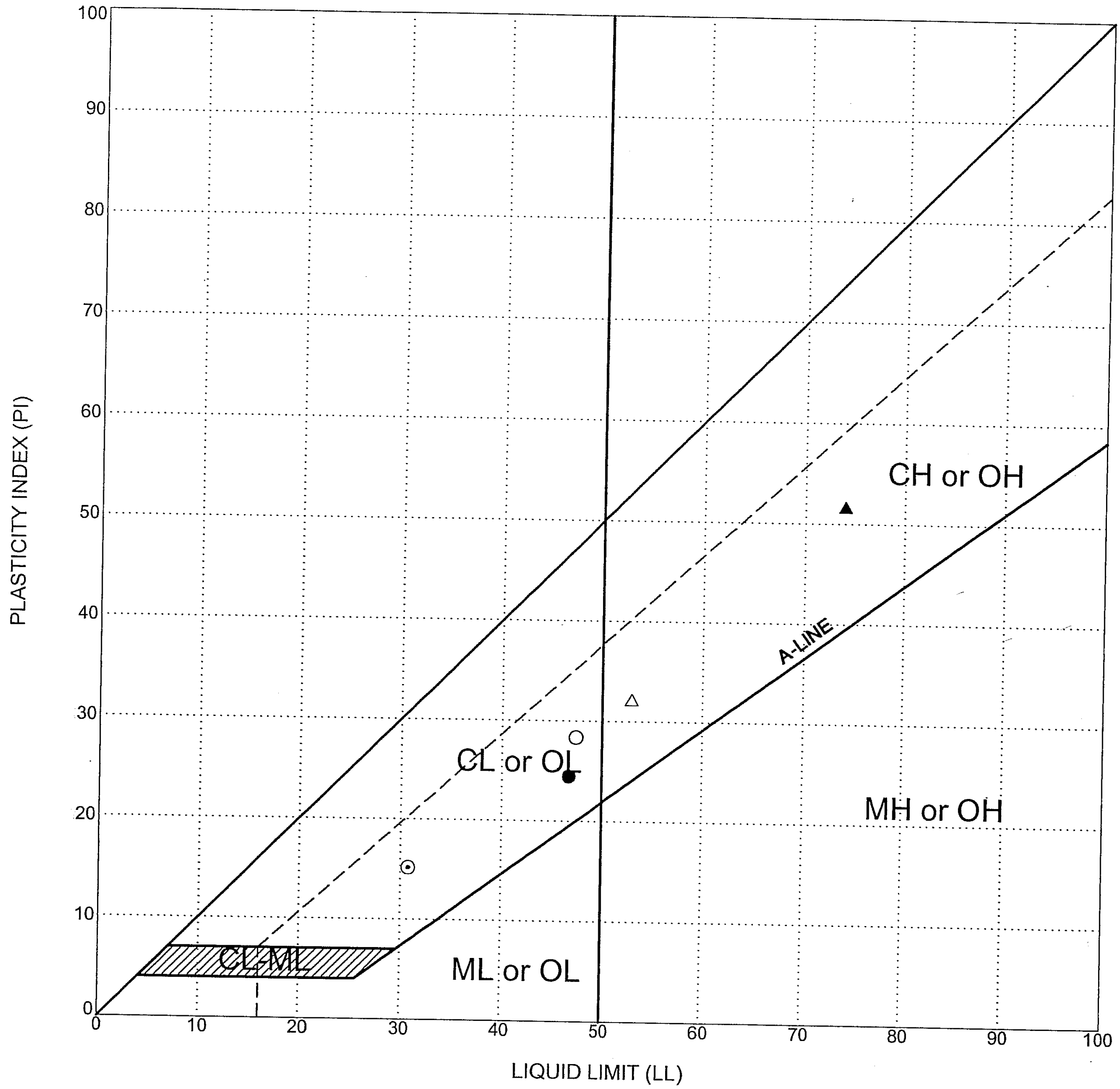
SUMMARY OF LABORATORY TEST RESULTS
Phelps and Devereaux Bridges
University of California Santa Barbara, California

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	SPECIFIC GRAVITY
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4 (%)				
DH-104	40.0	8b	Fat CLAY (CH)	119	87	36																	
DH-104	50.0	9b	Fat CLAY (CH)	132	108	22		31	15														
DH-104	59.5	10a	Lean CLAY (CL)												2.7(7.1)								
DH-104	60.0	10b	Lean CLAY (CL)	125	95	31																	
DH-104	69.5	12b	Lean CLAY (CL)	126	100	26																	

SUMMARY OF LABORATORY TEST RESULTS
Phelps and Devereaux Bridges
University of California Santa Barbara, California

PLATE B-1b

FUGRO

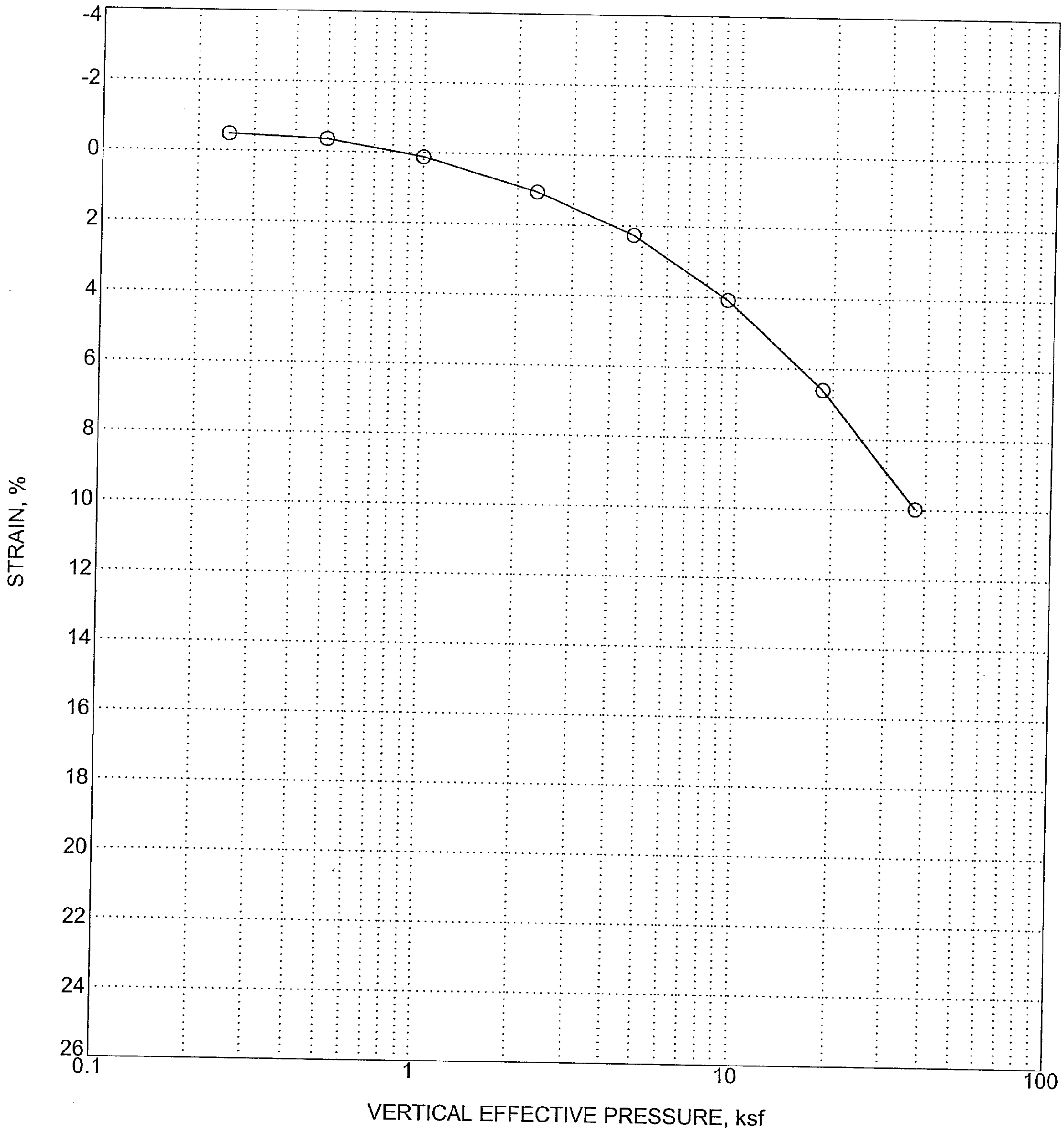


LEGEND location	depth, ft	CLASSIFICATION	LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
○ DH-101	14.5	Lean CLAY (CL)	47	19	28
● DH-102	4.5	Sandy Lean CLAY (CL)	47	22	25
△ DH-103	34.5	Fat CLAY (CH)	53	21	32
▲ DH-104	16.5	Fat CLAY (CH)	74	22	52
⊙ DH-104	50.0	Fat CLAY (CH)	31	16	15

ATTERBERG LIMITS TEST RESULTS

PLASTICITY CHART
Phelps and Devereaux Bridges
University of California Santa Barbara, California

PLATE B-2

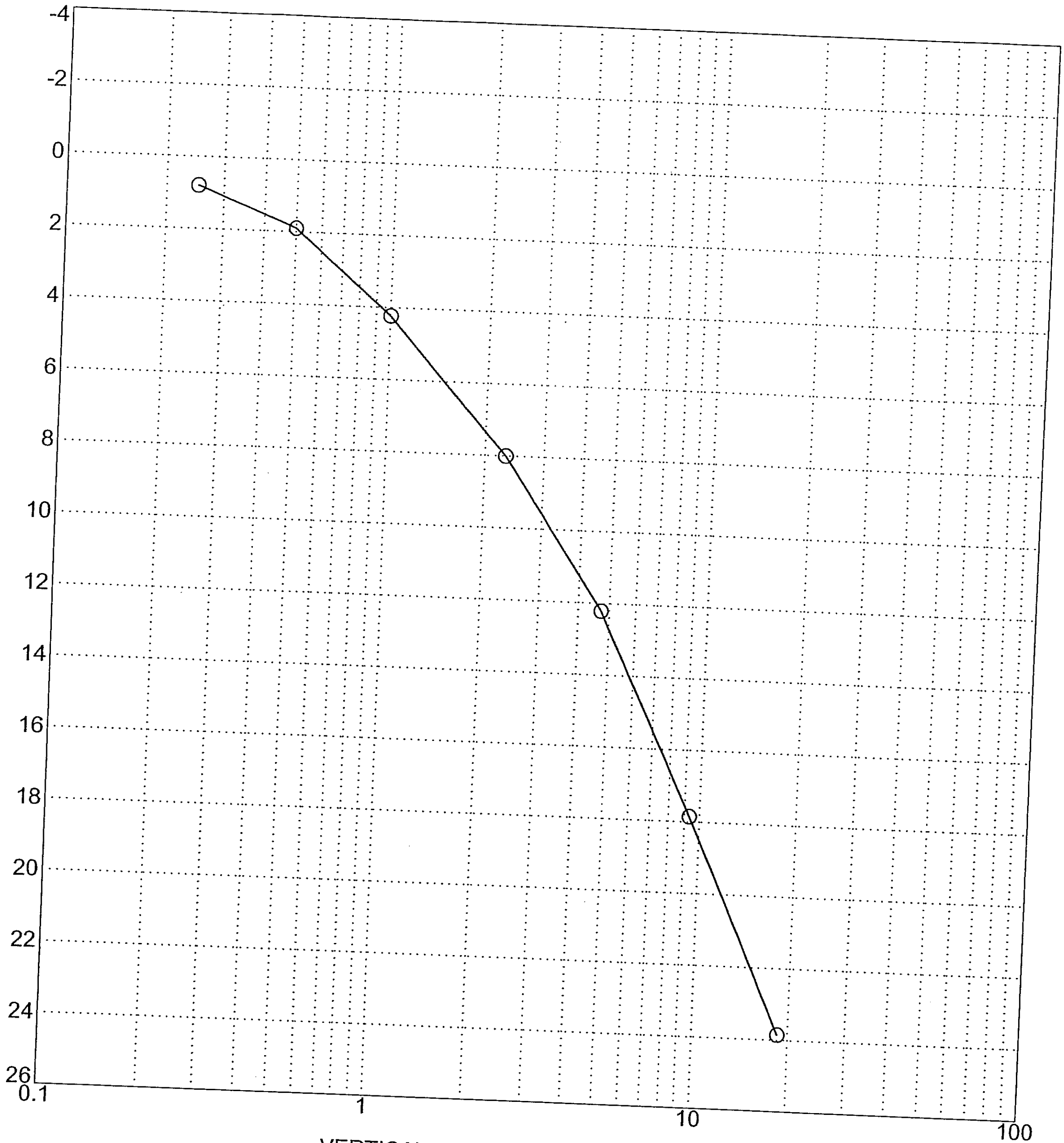


LOCATION	DH-101
DEPTH, ft	14.5
INITIAL MOISTURE CONTENT, %	22
UNIT DRY WEIGHT, pcf	108
MATERIAL DESCRIPTION	Lean CLAY (CL)
SAMPLE CONDITION	

CONSOLIDATION TEST RESULTS

Phelps and Devereaux Bridges
University of California Santa Barbara, California

PLATE B-3a

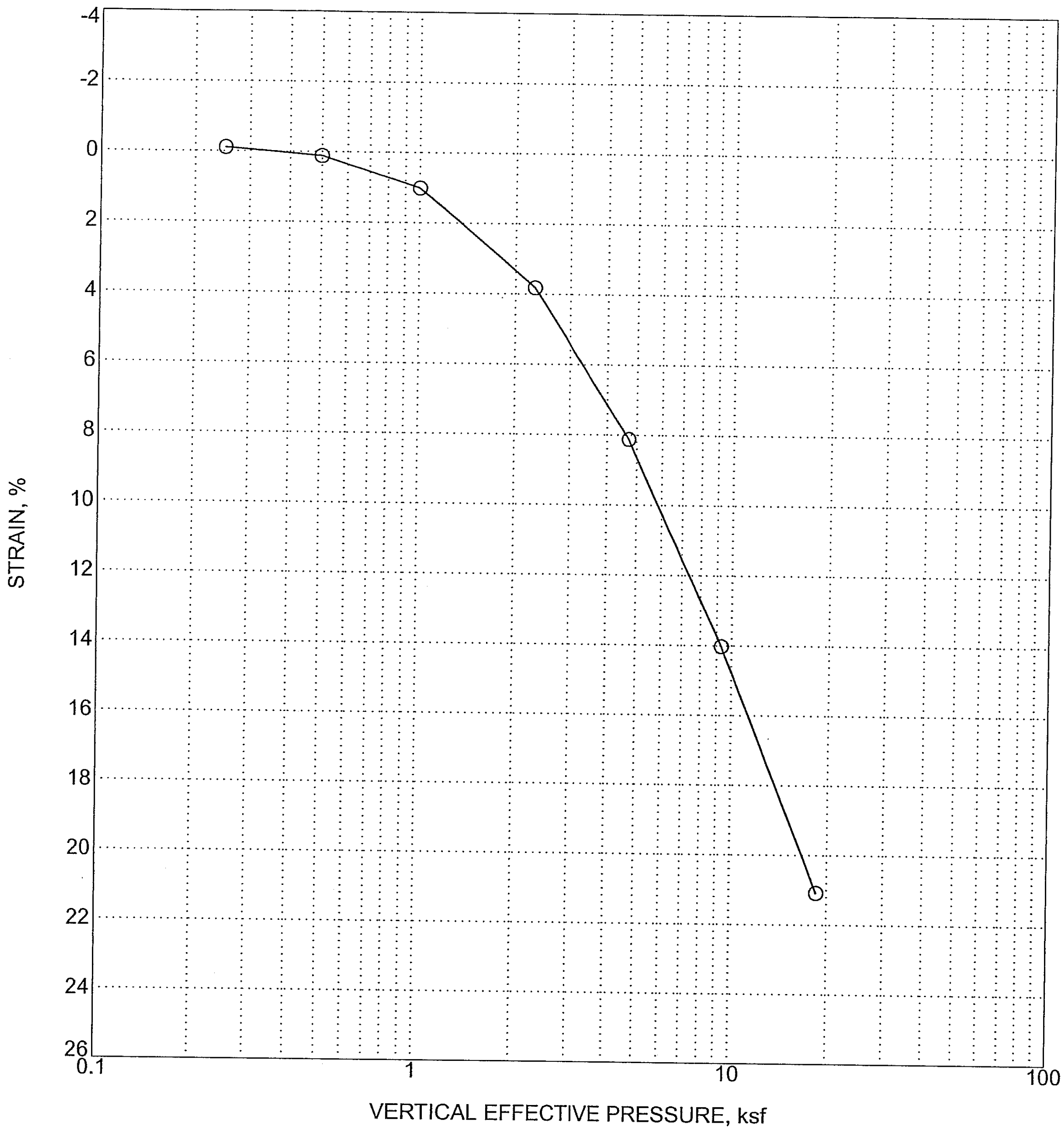


LOCATION
DEPTH, ft
INITIAL MOISTURE CONTENT, %
UNIT DRY WEIGHT, pcf
MATERIAL DESCRIPTION
SAMPLE CONDITION

DH-104
9.5
39
76
Fat CLAY (CH)

CONSOLIDATION TEST RESULTS
Phelps and Devereaux Bridges
University of California Santa Barbara, California

PLATE B-3b



LOCATION	DH-104
DEPTH, ft	40
INITIAL MOISTURE CONTENT, %	36
UNIT DRY WEIGHT, pcf	87
MATERIAL DESCRIPTION	Fat CLAY (CH)
SAMPLE CONDITION	

CONSOLIDATION TEST RESULTS
Phelps and Devereaux Bridges
University of California Santa Barbara, California

PLATE B-3c