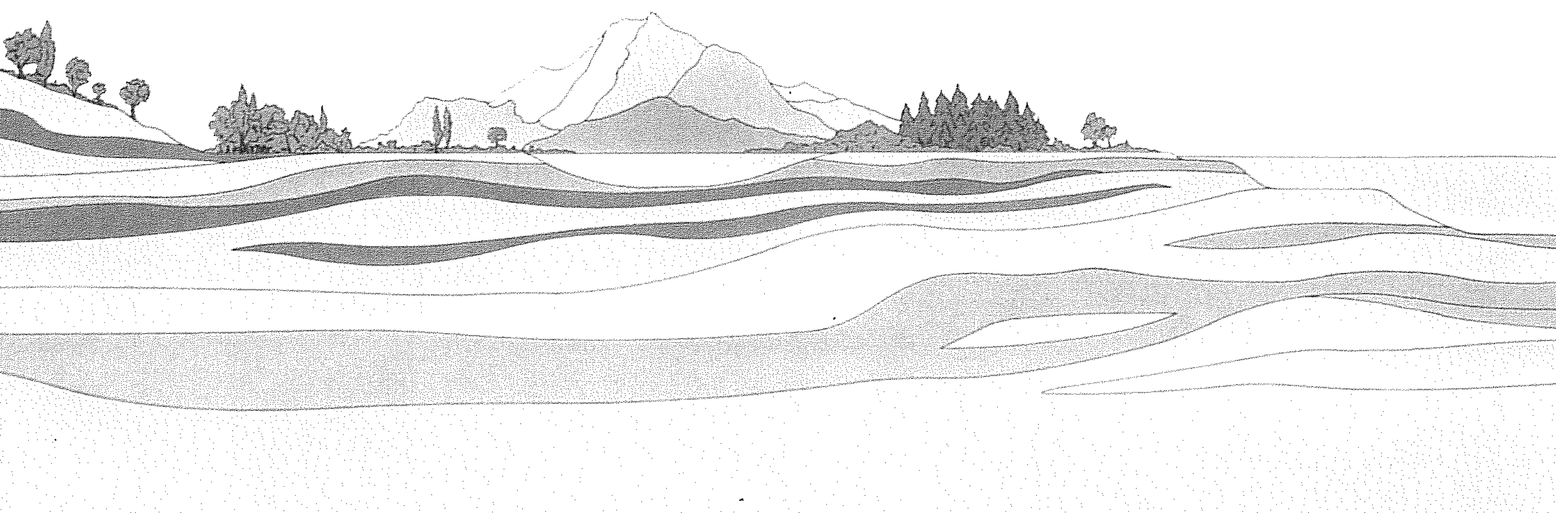
FUGRO WEST, INC.

GEOTECHNICAL REPORT
OCEAN SCIENCE EDUCATION BUILDING
UNIVERSITY OF CALIFORNIA, SANTA BARBARA
SANTA BARBARA, CALIFORNIA

Prepared for:
UNIVERSITY OF CALIFORNIA, SANTA BARBARA
REPORT No. 336

January 2006



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January 31, 2006
Project No. 3064.042

University of California at Santa Barbara
Office of Planning and Construction, Bldg. 439
Santa Barbara, California 93106

Attention: Mr. Gary Banks

Subject: Geotechnical Engineering Report – UCSB Report Number 336
Proposed Ocean Science Education Building,
University of California, Santa Barbara, California

Dear Mr. Banks:

Fugro is pleased to present this geotechnical engineering report for the proposed Ocean Science Education Building at the University of California, Santa Barbara. This study was performed in general accordance with our proposal dated July 22, 2004. Authorization for our services was provided by the University of California Authorization No. 021/05-06 dated September 29, 2005.

This report presents the findings of our subsurface exploration and laboratory testing programs and provides geotechnical engineering recommendations for site development and grading, foundation systems, and pavements. The submittal of this report completes our scope of services for the project. We appreciate the opportunity to work on this interesting project and to continue our professional relationship with the University of California, Santa Barbara. Please call our office if you have any questions regarding the findings, conclusions, or recommendations provided in this report.

Sincerely,

FUGRO WEST, INC.

Gregory S. Denlinger, GE 2249
Associate Engineer

REGISTERED PROFESSIONAL ENGINEER GREGORY S. DENLINGER NO. 2249 EXP. 3/31/06 GEOTECHNICAL STATE OF CALIFORNIA

Copies: 5 - Addressee

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1.0 INTRODUCTION

1.1 Proposed Project

We understand that the proposed Ocean Science Education Building (OSEB) project will be constructed between the existing Bio-Sciences II building and Lagoon Road in an area formerly occupied by Salt Water Holding Tanks, bicycle parking and adjacent landscaping. The general location of the project site is shown on Plate 1 – Vicinity Map. Our understanding of the project is based on discussions with Mr. Gary Banks of the University of California at Santa Barbara (UCSB), conceptual project information provided by UCSB, our previous experience with the Marine Science Research Building and similar projects on the main campus.

On the basis of information provided by UCSB, we understand the OSEB project will consist of a one- to two-story, reinforced concrete and structural steel structure with a building area footprint of about 6,000 square feet. The project will also consist of two to three exterior habitat structures ranging from 800 to about 1,400 square feet. From discussions with Mr. Banks, we understand that there is a possibility the main building will incorporate a basement level. Building loads are anticipated to range from about 75 to 150 kips for interior column loads and about 2 kips per foot for perimeter exterior building walls.

Fugro prepared the geotechnical engineering report for the Marine Sciences Research building, just north of the proposed project site. Data considered relevant from that project was incorporated into this report.

Grading requirements for the proposed project are also not known at this time. If the project does not incorporate a basement level, we anticipate the site preparation and grading work for the project will consist of general site clearing, preparation of subgrade soils for pavement and hardscape areas, excavation work to abandon and re-route existing utilities, and excavation work for foundation preparation and construction. However, more significant grading could be required in the building areas depending on the foundation system selected for the project and whether a basement level is included in the design.

1.2 Purpose and Scope

The purpose of our geotechnical engineering study was to obtain subsurface data at the site and prepare a design-level geotechnical engineering report for the project. The report provides geotechnical recommendations for site development and grading, foundation systems, and pavements. Existing geotechnical data from Fugro (2001) [Marine Science Research Building) and Law/Crandall (1994) [Bren Hall] were also used in the preparation of this report.

1.3 Work Performed

Our scope of services was presented in our proposal dated July 22, 2004. An outline of our work completed for this study is presented below:

Project Initiation and Data Review. We visited the site to locate proposed exploration locations, check the access for exploration equipment, and coordinate with facilities management staff and Underground Services Alert regarding locations of underground utilities. In addition, we reviewed previous data and information provided in Fugro (2001) and Law/Crandall (1994).

Subsurface Exploration. Three hollow-stem-auger drill holes were advanced near the location shown on Plate 2 – Subsurface Exploration Plan. Drill holes were excavated to depths ranging from about 20 below the existing ground surface. A more detailed description of the drilling and sampling work for the project is provided in Appendix A – Subsurface Exploration.

Laboratory Testing. Geotechnical laboratory tests were performed on selected samples obtained from the drill holes. Laboratory tests consisted of total and dry unit weight measurements, consolidation tests, direct shear tests, expansion index tests, and fines content determinations. A more detailed description of the geotechnical laboratory tests performed for this study together with the results of the various tests is provided in Appendix B – Laboratory Testing.

Geotechnical Evaluation and Reporting. Geotechnical evaluation consisted of characterizing the field and laboratory test data, assessing subsurface conditions, performing geotechnical evaluations for foundation design, and preparing this report that summarizes the findings and recommendations of the study.

The report provides information regarding the following:

- General summary of the soil and groundwater conditions encountered at the site;
- Liquefaction potential of on-site soils;
- Shallow and deep foundation support consisting of soil bearing pressures, foundation embedment depths, and anticipated settlement;
- Geotechnical parameters for seismic design in accordance with the UBC equivalent lateral force method;
- General design information regarding groundwater control and excavation conditions;
- Lateral earth pressures for the design of retaining walls, and recommendations for backfill, compaction, and drainage;
- Swell potential of the on-site materials;
- Design of concrete slabs-on-grade and asphalt pavements;
- General site grading recommendations and compaction requirements for compacted fill; and
- Material specifications for selected pavement, backfill, and drainage materials.

2.0 SITE CONDITIONS

2.1 Site Description

As shown on Plate 1, the OSEB project will be constructed between Lagoon Road and the existing Bio-Sciences II Building (Bio-II) and south of the existing Marine Science Research Building. The building area is currently occupied by an asphalt-paved parking lot, a bike path, and landscaping. The site is generally at an elevation of about 42 to 43 feet above mean sea level (MSL). As shown on an undated preliminary topographic map of the site provided to us by Penfield & Smith, underground utilities are common in this area and consist of water, sewer, gas, storm drains, and electrical conduits. Information regarding invert depths (or installed depths) of the various underground utilities has not been was provided to us by UCSB staff.

3.0 GEOLOGIC SETTING

3.1 Regional Geology

The project site is situated within the western Transverse Ranges Geomorphic/Geologic Province of southern California. The western Transverse Ranges province is characterized by an east-west structural grain, which is transverse to the general north-south structural grain of the remainder of California. In the project region, the province is dominated by the Santa Ynez Mountain Range, which extends continuously for about 75 miles from Point Arguello eastward into Ventura County.

The Santa Ynez Mountains and adjacent lowlands are generally composed of sedimentary rocks and soil materials ranging in age from Cretaceous to Recent. Structural geology in the Santa Barbara and Goleta area consists of a south-dipping homocline and adjacent coastal plain cut by a series of subparallel east-west trending faults and folds that are the result of north-south compressional tectonics. The faults and folds parallel the Santa Ynez Mountains and extend into the Santa Barbara Channel.

3.2 Local Geology

The main campus of UCSB is located on the northern portion of an elevated mesa that is generally bounded by the Pacific Ocean to the south and east, the Goleta Slough to the north, and the Devereux Slough to the west. The mesa has a gently undulatory surface, but is generally a flat lying marine terrace elevated 30 to 50 feet above MSL. Tectonic uplift during the Pleistocene-age is believed to have caused the elevation of the terrace (Dibblee, 1966). Erosion has dissected the marine terrace to produce the present isolated mesa.

The general geology of the campus area consists of a relatively thin cap of Pleistocene-age older alluvial deposits unconformably overlying Tertiary-age sedimentary rocks of the Sisquoc Formation. The Sisquoc Formation generally consists of massive to thickly bedded, poorly to slightly indurated, fractured/jointed siltstone. From a geotechnical engineering standpoint, the Sisquoc Formation can be classified as a very stiff to hard, fissured, elastic silt.

4.0 SUBSURFACE CONDITIONS

4.1 Soil Conditions

The description of soil conditions is based on visual classification of samples obtained from our field exploration. Information from our study and from previous investigations performed for adjacent sites were used to assist in characterizing the types and characteristics of the soil materials encountered. The consistency of the soils encountered was estimated from sampler blow counts recorded in the drill holes and from the cone penetration test (CPT) sounding data in Fugro (2001). The soil stratigraphy encountered in the drill holes excavated for this study is generally consistent with the conditions reported in Fugro (2001) and Law/Crandall (1994). However, we have interpreted the relative density of the terrace deposits below the groundwater level to be less than that suggested by the Law/Crandall (1994) data. Data from this study suggest that locally the terrace deposits below the groundwater are potentially susceptible to liquefaction.

The subsurface profile encountered consists of a nominal thickness of existing artificial fill material overlying granular terrace deposits. The fill and terrace deposit materials are underlain by Sisquoc Formation bedrock at depth. A description of the earth materials encountered in our drill holes is provided below.

Artificial Fill. Artificial fill material encountered in our drill holes is probably associated with the construction of the existing structures and improvements, underground utilities, and landscaping. The artificial fill generally consists of pavement materials (asphalt concrete and aggregate base) and silty sand. Differentiation between the fill and underlying terrace deposits is difficult because the fill material and the terrace deposits have similar characteristics (color, grain size, etc). Thus, the thickness of artificial fill at the site is somewhat uncertain but the fill is estimated to be about 5 feet thick at the project site. Deeper fill depths could locally be present (utility corridors, planter areas, etc.).

Terrace Deposits. Terrace deposits were encountered below the existing fill materials to a depth of between about 12 to 13 feet below the ground surface at the locations explored. The terrace deposits generally consist of loose to medium dense fine silty sand with zones of sandy silt with occasional layers of sandy lean clay. Field (uncorrected) standard penetration test (SPT) N-values (and SPT N-values estimated from California liner samples) measured in the drill holes ranged from about 8 to 19 blows per foot (bpf) with a mean value of about 13 bpf. Sampler blow count and cone penetration test data (Fugro 2001) show a trend of decreasing SPT N value with depth. Fines content (percent finer than .075 mm) measured on selected samples of terrace deposits from this study ranged from about 12 to 36 percent with an average value of about 25 percent.

Sisquoc Formation. Sisquoc Formation was encountered below the terrace deposits at depths of about 12 to 13 feet. The approximate elevation where Sisquoc Formation bedrock was encountered in our drill holes generally ranges from about +29 feet to +31 feet MSL and is generally consistent with the elevation of the terrace deposits/Sisquoc Formation contact reported in Fugro (2001) and Law/Crandall (1994). The Sisquoc Formation extended to the maximum depth explored of approximately 20 feet below the existing ground surface.

The Sisquoc Formation encountered in our drill holes generally consists of massive, highly to moderately weathered, poorly indurated, fractured/jointed siltstone. The extremely to highly weathered siltstone typically has a light brown to gray color and is characterized as claylike with minor rock texture. This material is generally more compressible with a lower SPT blow count and unconfined compressive strength than the underlying moderately weathered siltstone. The zone of extremely to highly weathered Sisquoc bedrock in the upper part of the formation on the main campus is generally about 5 feet thick. Shear strength of the Sisquoc Formation measured in unconsolidated undrained triaxial compression tests performed as part of Fugro (2001) ranged from about 4.5 to 13.5 ksf.

Although not observed in this study, we note that localized pockets of very hard siliceous material have been encountered in the Sisquoc Formation at other locations throughout the main campus. Law/Crandall (1994) reportedly encountered very hard siliceous bedrock materials between 21 and 26 feet in a bucket auger drill hole excavated in southeast corner area of the existing Bio II building. In our opinion, the potential for similar very hard siliceous bedrock layers to be present in the project area should be anticipated.

4.2 Groundwater Conditions

On the basis of our explorations and past experience at the University of California at Santa Barbara, groundwater generally exists in the terrace deposits as a result of groundwater perched on the underlying Sisquoc Formation or as seepage along joints and fractures in the Sisquoc Formation. Groundwater (or groundwater in the form of seepage) was not encountered at the completion of the drilling for the three drill holes excavated for this study. However, very moist soils and soils with high insitu moisture contents were noted at depths of about 8 to 10 feet below the existing ground surface (elevation of about 35 to 33 feet MSL).

Groundwater was noted in previous explorations reviewed for this project and those data are summarized in the following table.

Table -1 Summary of Previous Groundwater Data

Reference	Approximate Depth to Groundwater	Approximate Groundwater Elevation
Fugro (2001) MSRB	5 to 7-1/2 ft	36.5 ft
Law/Crandall (1994) Bren Hall	9 to 10-1/2 ft	32 to 35.5 ft
Bing Yen (2001) Life Sciences	8-1/2 ft	34.5 ft

On the basis of groundwater data acquired for this study and data reported in geotechnical reports prepared for adjacent projects, the depth to groundwater at the project site is estimated to be about 7 to 9 feet below the existing ground surface or at an elevation of about 36 to 34 feet (MSL). Excavations extending near or below that elevation have a high potential of encountering groundwater or seepage. In addition, pumping or unstable subgrade conditions could be encountered in excavations extending to within about 3 feet above the groundwater level when subjected to construction equipment loading. Therefore, we recommend the structural engineer or architect consider the potential for groundwater seepage or pumping subgrade conditions when evaluating proposed footing elevations. We also note that variation in the depth to perched groundwater and groundwater seepage can occur as a result of changes in land use, landscape irrigation, rainfall, or runoff.

5.0 FAULTING AND SEISMICITY

5.1 Fault Setting

Regional compressive tectonic forces acting in the Santa Barbara and Goleta area have resulted in generally east-west-trending near vertical faults with associated northeast and northwest splays. Displacement on those faults is generally believed to be mainly vertical, with the majority of faults having upthrown south blocks. Faults mapped proximal to the UCSB campus generally consist of the More Ranch fault system; Campus fault; and the offshore Coal Oil Point and Goleta Point faults.

The More Ranch fault system is part of the larger More Ranch-Mission Ridge-Arroyo Parida fault system. That fault system extends through the south coast of Santa Barbara County from Carpinteria on the east to Ellwood on the west where the fault trends offshore. The More Ranch fault consists of a steeply dipping thrust fault and is possibly one of the more significant regional faults in the Santa Barbara area in terms of strong ground shaking and ground rupture potential.

Olson (1982) maps the More Ranch fault system as consisting of a single fault trace east of Mescalitan Island and a north and south branch west of Mescalitan Island. Olson (1982) shows the north and south branches of the fault as trending roughly east-west about

2,000 feet north of the structure site. The More Ranch fault system is considered to be active to potentially active.

Upson (1951) maps an unnamed fault (Campus fault) trending northeast-southwest within the northern portion of the main campus and extending through Isla Vista. Olson (1982) maps the Campus fault as trending in a southwest direction through the main campus and Isla Vista. On the basis of data provided in Olson (1982), the Campus fault is located 1,500 feet northwest of the project site.

In the early 1970's R.C. Briggs, a geology student at UCSB, described a subsurface lineation across the northern portion of the main campus. The subsurface lineation was evaluated on the basis of soil boring data acquired for foundation design studies and corresponds to a relatively abrupt 3 to 4 foot elevation change (north side up) at the contact between the Sisquoc Formation and the overlying terrace deposits.

Dames & Moore (1973a, 1973b) indicates that evidence of faulting was observed in a trench excavation at the northwest corner of Building 489 and observed in the existing bluff slope northeast of Building 489. Dames & Moore (1973b) indicates that a 5 to 10 foot wide zone of faulting was observed in that trench striking about 45 degrees to the northeast and dipping about 80 degrees to the northwest with an approximate 5 foot offset (north side up) in the bedrock surface. Dames & Moore (1973b) indicates that the overlying Quaternary-age terrace deposits were not offset and suggests that the Briggs Lineation is structurally related to the Campus fault.

Gurolla and Alex (1997) map the location of the Briggs Lineation as extending southwest from near the southeast corner of Phelps Hall to about 1,000 feet east of the Events Center. The mapped location of the lineation is shown about 1,500 feet northwest of the site. The location of the Briggs Lineation as mapped by Gurolla and Alex is based on soil boring data and contour mapping of the Sisquoc Formation/terrace deposit contact. Gurolla and Alex position the Briggs Lineation/Campus fault at the location of a noticeable 3- to 4- foot step (north side up) in the Sisquoc Formation/terrace deposit contact.

Olson (1982) maps the Coal Oil Point fault as trending east-west offshore of the Devereaux-Univeristy mesa. The Goleta Point fault has been mapped by others as trending northeast-southwest in the vicinity of the Goleta Point, southeast of the site. The Coal Oil and Goleta Point faults are considered to be inactive to potentially active.

Other regional faults in the Santa Barbara or southern California area have the potential to produce strong ground motions at the site. A listing of those faults and their proximity to the site is provided in Table 2 – Summary of Fault Parameters.

Table - 2 Summary of Fault Parameters

Fault Name	Distance Between Site and Surface Projection of Earthquake Rupture Area (km)	MCE Magnitude	Source Type (A,B, C)	Slip Rate (mm/yr)
More Ranch-Mission Ridge-Arroyo Parida	<1	6.7	B	0.4
Santa Ynez (West)	13	6.9	B	2.0
Red Mountain	19	6.8	B	2.0
Santa Ynez (East)	23	7.0	B	2.0
Los Alamos - W. Baseline	28	6.8	B	0.7
Ventura – Pitas Point	30	6.8	B	1.0
Santa Cruz Island	39	6.8	B	1.0
Santa Rosa Island	44	6.9	B	1.0
Lions Head	46	6.6	B	0.02
Big Pine	51	6.7	B	0.8
Anacapa – Dume	51	7.3	B	3.0
San Luis Range (S. Margin)	59	7.0	B	0.2
Oak Ridge (Onshore)	64	6.9	B	4.0
San Cayetano	64	6.8	B	6.0
San Andreas – 1857 Rupture	72	7.8	A	34.0

In February 1998, the International Conference of Building Officials (ICBO 1998) released "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, to be used with the 1997 Uniform Building Code." That document contains a list of faults that is slightly different from the faults that comprise the original CDMG fault database for the State of California. The primary differences were the exclusion of blind faults in ICBO (1998) and the upgrading of several Type C faults to Type B faults and one Type B fault to a Type A fault. In the area of the subject site, those changes resulted in the exclusion of the Channel Islands (eastern half), the Montalvo-Oak Ridge Trend, North Channel Slope, and the Oak Ridge (offshore blind thrust) faults in ICBO (1998). As a result, the UBC Type B fault that is nearest to the site is the More Ranch-Mission Ridge-Arroyo Parida fault system.

5.2 Historical Seismicity

As with many other locations in southern California, the site has experienced strong ground shaking from historical earthquakes. A search of historical earthquakes that could have generated strong ground shaking in the UCSB campus area was evaluated using the computer program EQSEARCH (Blake 2000).

The historical record indicates that the area has experienced shaking from a number of seismic events over the course of the last 182 years. Some of the seismic events that likely resulted in varying degrees of ground motion at the site are the earthquakes of 1812, 1857, 1862, 1893, 1925, 1929, 1933, 1941, and 1978. The 1812 and 1857 events are thought to have occurred along the Mojave Segment of the San Andreas fault and caused significant damage to developed areas of southern and central California. Those earthquakes were estimated to have had moment magnitudes of approximately M7.0 and M8.0, respectively. The M6.3, 1925 event produced significant ground shaking in the local Santa Barbara region causing damage to much of the downtown area. The strongest historical ground motion at the site was likely produced by either the M5.7, 1862 earthquake; the M7.0, 1812 event; or the 1925, M6.3 Santa Barbara earthquake.

5.3 Probabilistic Seismic Hazard

A site-specific probabilistic seismic hazard evaluation for the project was considered beyond the scope of services of this study. However, probabilistic seismic hazard analyses have been performed for other recent projects on the main campus such as the Seawater System Renewal Project (Fugro 1997), the San Rafael Student Housing Project (Fugro 1998), and the Engineering Science Building project (Fugro 2000). Additional probabilistic seismic data for the entire campus area is summarized in Law/Crandall (1999). On the basis of those studies, probabilistically estimated peak ground accelerations generated at the site corresponding to a 475-year return period event could likely be in the range of about 0.4g. Response spectra data for the main campus of UCSB are provided in Law/Crandall (1999). Those data can be provided to the structural engineer if required.

6.0 GEOLOGIC HAZARDS

6.1 Strong Ground Shaking and Ground Rupture

In our opinion, the proposed project will likely experience strong ground shaking at some point in the design life the project. Peak ground accelerations at the site generated from maximum credible events on local or regional faults could produce peak ground accelerations at the site on the order of 0.3 to 1.0 g or greater. Therefore, we recommend that the proposed project be designed, at a minimum, in accordance with the latest building code provisions.

On the basis of our review of available data, in our opinion the closest mapped fault considered significant to the project is the More Ranch fault mapped about 2,000 feet northwest of the site. Therefore, the potential for ground surface rupture at the site appears to be low.

6.2 Liquefaction

Liquefaction is the loss of shear strength in soil caused by earthquake-generated seismic shaking. Liquefaction typically occurs in loose, saturated granular soil materials and can take place at significant depths. However, for shallow foundations, soil liquefaction is generally not considered to be a significant concern if the soil materials consist of clayey soils or dense granular soils, or groundwater is not present within the upper 40 to 50 feet. Deep foundation elements could be effected by soil liquefaction occurring at greater depths.

Granular soils below the groundwater level encountered at the site generally consist of loose to dense silty fine sand and silt with fines content ranging from about 12 to 36 percent. Standard penetration test blow counts (and approximate equivalent SPT blow counts estimated by multiplying California liner sampler blow counts by 1.6) measured below the level of perched groundwater ranged 8 to 18 blows per foot. Corrected SPT blow counts (blow counts corrected for overburden, energy, sampler type, rod length, fines content etc.) ranged from 15 to 31 blows per foot. Using the blow count data together with the liquefaction analysis procedure described in Youd and Idriss (1997), the terrace deposit materials generally within about 3 to 4 feet of the bedrock contact may be susceptible to liquefaction (assuming a 0.4g peak ground acceleration weighted to an earthquake magnitude of 7.5 from Law/Crandall [1999]).

Based on the current groundwater conditions, consequences of liquefaction occurring in the terrace deposits within a few feet of the bedrock could potentially consist of localized settlements of less than about 3/4 inches to about 1 inch. This evaluation is generally consistent with the findings in Fugro (2001).

The Sisquoc Formation bedrock is not considered susceptible to liquefaction. Thus the potential for liquefaction at the site could be mitigated by overexcavating and recompacting the terrace deposits. Alternatively, the potential impacts associated with liquefaction could be addressed by supporting the proposed structures on deep foundations or a basement level foundation system bearing in the Sisquoc Formation bedrock.

6.3 Lateral Spreading

In our opinion, the consequences of liquefaction are generally anticipated to consist of ground surface settlement, however, past experience suggests that lateral deformation of the ground can also occur as a result of liquefaction. The occurrences of lateral spreading is generally associated with sites where liquefaction is possible and 1) the ground surface is gently sloping or 2) there is a free face conditions such as a road cut or river bank. Existing analytical methods of assessing potential deformations caused by lateral spreading are based on a small number of case histories and generally involve layers of liquefiable soils of greater than a meter. The procedures are generally considered reasonable in assessing risks where significant lateral deformations are possible (deformations of a meter or greater). The ability to reasonably predict small lateral spreading deformations is, however, considered significantly limited.

Considering the current soil and groundwater conditions, the generally limited liquefaction hazard, and the relatively level surface topography of the site, the potential for lateral spreading to occur at the site is considered to be low. However, the risk of lateral spreading deformation should not be considered non-existent.

6.4 Bearing Capacity

Considering the subsurface conditions and the foundation sizes currently proposed for the project, we do not anticipate that bearing capacity failure to be a significant hazard to the foundation system. Settlements resulting from localized liquefaction are considered to be a more significant consequence and should be considered in the foundation design.

6.3 Seismic Settlement and Differential Compaction

Seismically-induced settlement and differential compaction occur when loose to medium dense granular soils densify and undergo a reduction in volume during ground shaking. Such seismically-induced settlement can occur in relatively dry and partially saturated granular soils. Because of the relatively limited thickness of granular soils above the water level, we anticipate the potential for seismically-induced compaction to be low and related settlements are anticipated to be negligible.

6.4 Landsliding

The site is located on relatively level ground about 100 feet from the coastal bluff slope. In our opinion, the potential for landsliding at the project site is considered to be low. However, the potential for long-term bluff retreat should be considered as a possible hazard for project elements, especially in the area east of Lagoon Road.

6.5 Earthquake Induced Flooding

The site is located adjacent to the Pacific Ocean at an elevation of about +42 feet MSL. Studies conducted by U.S. Army Corps of Engineers (Houston and Garcia, 1974) indicate that the Santa Barbara coastal area could be exposed to wave run-ups of 5.5 and 11 feet from a 100- and 500-year tsunami event, respectively. Assuming that the event occurs during a high tide of about 8 feet (MHHW), run-up elevations would extend to about elevation +14 and +19 feet from a 100- and 500-year tsunami event, respectively. Therefore, it appears that the potential tsunami hazard at the site is relatively low.

Because the site is not located adjacent to large bodies of impounded water, the potential for seiches to impact the site is considered to be low. However, seiche impacts could occur adjacent to pool-type habitat structures and should be considered in the planning and design of those facilities.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Foundation Support Alternatives

Our conclusions and recommendations are based on our understanding of the project, our review of relevant existing geotechnical data in the site vicinity, and site-specific exploration and testing programs performed for this study. On the basis of data acquired in this study, the granular terrace deposits below the groundwater appear to be susceptible to liquefaction. As described in Section 6.2, the consequences of liquefaction could possibly consist of ground surface settlement on the order of 3/4 inches to about 1 inch and lateral movement (estimated to be less than a couple of inches).

In our opinion, the proposed structure can be supported on a deep foundation system consisting of the following types of foundation systems:

- Cast-in-drill-hole piers (drilled piers) embedded in the underlying Sisquoc Formation bedrock or a shallow foundation system bearing on compacted fill with a structural slab or slab on grade. A structural slab should be used if the potential for settlement of the slab from liquefaction is not considered acceptable.

- Shallow foundation system supported on compacted fill with a slab on grade floor system. For this case the terrace deposits would be removed down to bedrock and replaced with compacted fill to eliminate the liquefaction potential. Alternatively the project could be designed to include a basement level and support the footings on bedrock materials.

- Shallow foundation system supported on a relatively thin layer of compacted fill (i.e. less than 1 to 2 feet of fill below the footing) with a slab on grade floor system. This system should only be used if the potential impacts from liquefaction of the existing terrace deposits can be accommodated in design and are considered acceptable to the University.

Recommendations for site development and grading and for the design of proposed foundation systems are presented in this section of the report. Design recommendations are provided for both drilled pier and shallow foundation systems.

7.2 Site Development and Grading

7.2.1 General

Fill placement and grading operations should be performed according to the grading recommendations of this report. For the purposes of evaluating the percent relative compaction, the maximum soil density and optimum moisture content should be estimated using standard test method D1557 of the American Society of Testing Materials (ASTM). We recommend that fill materials be compacted to a minimum of 95 percent relative compaction (as determined by ASTM D1557) unless otherwise noted in this report. The in-situ or field moisture content and relative compaction should be evaluated using either ASTM D1556 (sand cone method) or ASTM D2922 (nuclear) as the fill materials are placed.

7.2.2 Clearing and Grubbing

Existing fills, soil containing debris, organics, pavements, or other unsuitable materials should be excavated and removed from the project site prior to commencing grading operations. Demolition areas should be cleared of old foundations, slabs, abandoned utilities, landscaping, and soils disturbed during the demolition process. Depressions or disturbed areas left from the removal of such material should be replaced with compacted fill.

On the basis of our drill holes, the thickness of existing fill materials appears to be less than 5 feet. However, the project specifications should provide for variations in the actual thickness and aerial extent of the existing fill materials. The limits and depths for removal of existing fills materials should be evaluated during grading. We recommend that Fugro be contacted if extensive zones or thicknesses of existing fill material are encountered during grading.

7.2.3 Excavation Conditions

Subsurface materials encountered in our drill holes consisted of artificial fill and terrace deposits underlain by massive, fractured/jointed, poorly indurated, highly to moderately weathered siltstone bedrock. On the basis of our understanding of the site conditions, excavations and site grading can be performed with conventional heavy-duty grading equipment that is in good working order.

7.2.4 Groundwater

In our opinion, there is a potential for excavations to encounter perched groundwater or groundwater seepage. Groundwater was encountered perched on the siltstone bedrock at depths of about 8 to 10 feet below the existing ground surface (elevation of about 35 feet MSL). In our opinion, the potential for encountering groundwater below a depth of about 8 feet (approximate elevation 35 feet) should be considered in the design and construction of the project. However, as described in a previous section of the report, pumping subgrade conditions can develop in excavations extending down to within about 3 feet of the groundwater level.

We recommend that provisions be incorporated into the contract documents that deal with groundwater seepage or pumping subgrade conditions within the proposed excavation depths. This is especially true if a basement level is included in the project design. If groundwater seepage or pumping subgrade conditions are encountered in the proposed excavations, we recommend that an appropriate dewatering system be installed to lower the groundwater to below the lowest excavation level. The contractor should be responsible for the design, installation, and operation of the dewatering system. Dewatering wells or trench drains should be considered for controlling water in excavations in the terrace deposits that extend below or near the groundwater level. On the basis of past experience with excavations in the

Sisquoc Formation, local sumping and pumping of free water at the bottom of the excavation could be suitable for the project.

If the groundwater level has been lowered as recommended and pumping conditions are still encountered, we recommend that the subgrade be stabilized prior to placing fill or construction of foundation elements. A discussion of potential subgrade stabilization measures is provided in Section 7.2.9. Measures to stabilize the subgrade may be required and provisions should be provided for in the contract documents.

7.2.5 Temporary Excavations

The soils encountered at the site generally consist of silty sand terrace deposits underlain by Sisquoc Formation siltstone bedrock. Sisquoc Formation bedrock was generally encountered at a depth of between 12 and 13 feet below the existing ground surface.

The contractor should be responsible for the design of temporary slopes. At a minimum, temporary slopes should be constructed in accordance with OSHA guidelines. As an aid to the design of temporary excavations, we recommend that the existing terrace deposits (above the groundwater level or in a dewatered condition) be classified as Type B material per OSHA guidelines. In our opinion, excavation slopes constructed in the terrace deposits are subject to slumping in areas where groundwater seepage is not controlled. Sisquoc Formation bedrock can be considered as meeting OSHA requirements for Type A material provided groundwater and seepage in the joints and fractures are properly controlled.

As input to the planning of temporary excavations, we suggest that dry or properly dewatered temporary excavations in the terrace deposits be inclined at 1h:1v or flatter and dry temporary excavations in undisturbed Sisquoc Formation bedrock be constructed at an inclination of 0.75h:1v or flatter. We recommend that temporary excavations are monitored for signs of instability and appropriate actions (such as flattening the slope, providing shoring, or controlling groundwater) are undertaken if evidence of potential instability is observed.

7.2.7 Grading Recommendations in Building Areas

Drilled Piers. If a drilled pier foundation system is to be used for structure support, we recommend the existing soils be removed to a depth of at least 3-1/2 feet below the existing ground surface (or greater to remove existing fill soils) or to a depth of equal to the lowest pier cap elevation, whichever results in the larger overexcavation depth. The overexcavation should be uniform and extend throughout the building area. The overexcavation should extend a distance of at least 5 feet beyond the outside edge of the perimeter pier caps. The exposed surface should be scarified and moisture conditioned to within 2 percent of the optimum water content and compacted to at least 95 percent relative compaction. Subsequent fill should be placed in accordance with the recommendations provided in Section 7.2.10.

Shallow Foundations on Compacted Fill. If a shallow foundation system is to be used and the potential for liquefaction is not considered acceptable to UCSB, we recommend the existing terrace deposits be overexcavated to expose Sisquoc Formation bedrock throughout the building area. The overexcavation should extend a distance of 10 feet beyond the building perimeter. Control of groundwater will likely be required to facilitate the excavation. Local sumping and pumping of groundwater from the bedrock surface could potentially be sufficient to control groundwater. If the subgrade at the bedrock surface is subject to pumping, the subgrade should be stabilized as outlined in Section 7.2.9 prior to placing compacted fill. Following subgrade stabilization (if required) subsequent fill material should be placed as described in Section 7.2.10.

Shallow Foundations on Compacted Fill/Terrace Deposits. If the potential for liquefaction is considered acceptable to UCSB and it is desired to use a shallow foundation for structure support, site grading can consist of overexcavating the existing soils to a depth of at least 3-1/2 feet below the existing ground surface (or greater to remove existing fill soils) or to a depth equal to the lowest footing elevation, whichever is greater. The overexcavation should be uniform across the building area (that is at a uniform elevation) and extend at least 5 feet beyond the building footprint (outside edge of perimeter footings). The exposed subgrade should be scarified and moisture conditioned to within 2 percent of the optimum moisture content and compacted to at least 95 percent relative compaction. Subsequent fill placed in the overexcavated area should be placed as outlined in Section 7.2.10.

Groundwater. We note that excavations extending to within about 3 feet of the anticipated groundwater level (about elevation 35 feet) could experience pumping conditions. Excavations extending below about 35 feet could encounter free water in the excavation bottom. If the subgrade is subject to pumping under construction traffic loads, we recommend the subgrade be stabilized as described in Section 7.2.9

7.2.8 Grading Recommendations in Pavement and Hardscape Areas

We recommend that existing soil materials in pavement and hardscape areas be overexcavated to a depth of about 2 feet below the existing ground surface or to a depth of 1 foot below the design section, whichever results in the larger overexcavation depth. The overexcavation should extend laterally at least 3 feet beyond the pavement limits. The exposed subgrade should be moisture conditioned to within about 2 percent of the optimum moisture content and compacted to a minimum of 95 percent relative compaction. If required, the subgrade should be stabilized as described in Section 7.2.9. Subsequent fill should be placed as described in Section 7.2.10.

7.2.9 Subgrade Stabilization Measures

Special stabilization measures may be required if soft or pumping subgrade conditions are encountered during construction, particularly where excavation bottoms are close to the groundwater level. Those measures may be required to provide a firm and unyielding subgrade surface. Where soft or pumping subgrade conditions have been encountered on previous projects, subgrade stabilization measures generally have consisted of one or more of the following:

- Overexcavating a portion of the unstable subgrade and attempting to "bridge" over the pumping soil using compacted aggregate base (for relatively minor pumping conditions only);
- Placing float rock over the pumping subgrade soils in combination with the use of geotextile fabric and geogrids;
- Placing lean-concrete or cement slurry (i.e., mud-mats) over the pumping subgrade; and
- Additional lowering of the groundwater level and reducing the water content of the soil materials by aerating or exposure to sun/wind.

Whether these measures are required or not will depend on the depth of the excavation relative to the groundwater level, nature of the subgrade material, and possibly the contractor's excavation methods. Past experience suggests that float rock/aggregate base thicknesses between 1 to 2 feet may be required to provide a suitable subgrade surface upon which fill materials may be placed and compacted. A geosynthetic filter fabric should be placed beneath and encapsulate the float rock to separate the rock from the adjacent native soils and to prevent migration of native soils into the interstices of the rock.

7.2.10 Fill Placement

Fill and backfill materials should be placed in layers that can be compacted with the equipment being used. Each layer should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer.

We recommend that the fill be placed at a compaction moisture content within about 2 percent of the optimum moisture content. 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 a 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.

We note that the terrace deposits can be sensitive to the compaction moisture content. If the terrace deposits are placed at a moisture content higher than about 2 percent above the optimum moisture content, there is a potential for pumping of the subgrade or fill when subjected to construction traffic or compactive effort. In addition, in-place terrace deposits at a moderate or high water content can become unstable and pump under construction equipment traffic.

Fill placed in structural or pavement areas should be compacted at a moisture content near the optimum. After each layer has been conditioned and placed, it should be spread in lifts no thicker than approximately 8 inches and compacted to a minimum of 95 percent relative compaction unless otherwise noted.

7.2.11 Materials

On-Site Materials. Materials to be used as fill in structural or pavement areas should consist of granular on-site soils having a low potential for expansion and be free of organic and other deleterious materials. Based on our evaluation of the subsurface conditions, we anticipate that the majority of the fill materials and terrace deposits have a low potential for expansion within the expected depths of grading. For preliminary planning purposes, about 75 percent of the fill and terrace deposit materials may be assumed to have a low potential for expansion. During grading operations, the fill and terrace deposits should be evaluated for organic content and expansion potential.

The Sisquoc Formation encountered below the terrace deposits generally consists of highly weathered siltstone (classified as "elastic silt" according to the unified soil classification system) to slightly weathered siltstone. Sisquoc Formation materials are considered to have a medium potential for expansion and we recommend that those materials not be used as compacted fill in structural or pavement areas.

Imported Fill Material. Imported materials to be used for compacted fill should be evaluated prior to being brought to the site. Imported fill should consist of granular material with less than 30 percent passing the No. 200 sieve and have an Expansion Index of less than 20. In addition, imported fill material to be placed within 2 feet of finished grade in pavement areas should have an R-value of at least 20, as determined by California Test 301.

Retaining Wall Backfill. Backfill material for retaining walls should consist of imported granular soils with less than 20 percent passing the No. 200 sieve and a minimum sand equivalent (SE) of 20. We do not anticipate that the excavated on-site soils (existing fill, terrace deposits, or Sisquoc Formation) will have the minimum recommended sand equivalent or be suitable for use as retaining wall backfill.

Float Rock. Float rock should consist of 4-inch minus quarry-run rock having 100 percent of the material passing the 4-inch sieve, 0 to 30 percent passing the 2-inch sieve, 0 to 10 percent passing the 3/4-inch sieve, and less than 5 percent passing the No. 4 sieve. Float rock aggregates should have at least 75 percent fractured faces.

Asphalt Concrete. Asphalt concrete should consist of Type B asphalt concrete conforming to Section 39, "Asphalt Concrete" of the Caltrans Standard Specifications. As an alternative, asphalt concrete can consist of Class B or C2 asphalt concrete conforming to Section 203-6, "Asphalt Concrete" of the Standard Specifications for Public Works Construction (Greenbook).

Aggregate Base. Aggregate base shall be Class 2 conforming to Section 26-1.02A, "Class 2 Aggregate Base" of the Caltrans Standard Specifications. As an alternative, aggregate base can consist of base conforming to Section 200-2.5, "Processed Miscellaneous Base" of the Greenbook.

Filter Fabric. Filter fabric used for underdrains or for separation of contrasting soil material types should consist of Mirafi 140N or equivalent.

Free-Draining Backfill. Free-draining backfill should consist of clean, coarse-grained material with no more than 5 percent passing the No. 200 sieve. Acceptable backfill would be:

- "Pervious Backfill" conforming to Item 300-3.5.2, Standard Specifications for Public Works Construction (Greenbook, 1997);
- "Permeable Material" conforming to Item 68-1.025, Caltrans Standard Specifications; or
- Crushed stone, sized between 1/4 and 1 inch (if crushed stone is used, a filter fabric should be used to separate the rock form the surrounding soil).

7.3 Foundation Design

On the basis of the subsurface conditions encountered during our field exploration and the results of our geotechnical laboratory testing program, in our opinion, the proposed structure can be supported on either cast-in-drilled-hole (CIDH) piers embedded in the Sisquoc Formation or shallow foundations bearing on compacted fill.

7.3.1 Drilled Cast-in-Place Piers

In our opinion, the proposed structures can be supported on drilled pier foundations. We anticipate that the pier foundation system would have good performance in the event that liquefaction of the terrace deposits were to occur. If the potential for settlement of the floor slab of about 3/4 inches to 1 inch is considered acceptable, the floor system could consist of a slab-on-grade. If that potential is not acceptable, the floor system should be structurally supported.

The drilled piers could consist of straight-shaft friction piers, end bearing piers, or as combined friction and end bearing piers. For design, we recommend that friction piers have a minimum embedment of at least 15 feet into moderately weathered Sisquoc Formation. End bearing piers should be embedded at least 5 feet into moderately weathered Sisquoc Formation. As guidance for estimating the depth of drilled piers, the top of the moderately weathered Sisquoc Formation can be assumed to be at about +30 feet MSL. The actual depth of the piers should be evaluated during excavation for the piers. We recommend that piers be designed with a center-to-center spacing of at least 3 pier diameters. If needed, we can provide additional recommendations for piers spaced closer than 3 diameters.

Friction Piers. For design, we recommend that friction piers have a minimum diameter of 2 feet and be embedded in moderately weathered Sisquoc Formation. We recommend using an allowable frictional resistance of 2,000 psf for the portion of the pier embedded in moderately weathered Sisquoc Formation. The portion of the pier above the moderately weathered

Sisquoc Formation surface should be neglected when computing the allowable frictional capacity. A one-third increase in the frictional resistance can be used when considering short-term wind or seismic loads. The uplift capacity of drilled cast-in-place piers can be estimated as equal to the frictional resistance plus the dead weight of the pier.

Groundwater seepage and localized caving may be encountered in drilled pier excavations and casing will be required to control groundwater and minimize caving of the shaft. In our opinion drilled piers can likely be excavated without the use of drilling fluids provided good workmanship is used to install the drill casing. Drilling fluids, if used, should be approved by the design engineer.

We estimate that the total settlement of foundations supported on drilled friction piers designed as described herein should generally be on the order of 1/2 inch total. Differential settlements of about 1/4 to 1/2 inch could occur between similarly loaded friction piers.

End Bearing Piers. We recommend that end bearing piers, if used, be designed using a minimum shaft diameter of three feet to provide room for cleaning the bottom of the pier and downhole observation, as needed. Drilled piers can be designed for combined friction and end bearing provided the bottom of the piers are cleaned prior to placing concrete. The base of the pier can be expanded to form a belled pier to provide additional end bearing capacity. If belled piers are designed for combined friction and end bearing, the bottom one diameter of the pier stem above the pier bottom (or the top of the bell, if used) should not be included in the calculation of frictional resistance. Frictional resistance can be estimated as described in the above section.

We recommend using a maximum allowable end bearing pressure of 8,000 pounds per foot square foot for drilled piers founded in moderately weathered Sisquoc Formation. The combined capacity is the sum of the end bearing capacity and frictional capacity of 2,000 psf for the portion of the pier embedded in Sisquoc Formation bedrock. A one-third increase in the end bearing pressure can be used when considering short-term wind or seismic loads. The uplift capacity of belled piers can be estimated as one-half the frictional resistance plus the dead weight of the pier.

We estimate that the total settlement of foundations supported on drilled friction and end bearing piers designed as described herein are anticipated to be less than 1/2 inch total. Differential settlements of up to about 1/4 inches should be considered between similarly loaded friction piers.

Resistance to Lateral Loads. We estimated the lateral load capacity of a single drilled cast-in-place pier using the computer program LPILE plus (Ensoft 1997). The computer program estimates the lateral load-deflection behavior of the pier using a soil resistance-pile deflection model (p-y analysis). Our analysis used a minimum 28-day compressive strength for concrete of 4,000 pounds per square inch. We have estimated the pier's lateral load capacity and maximum moment for an equivalent 1/4-inch horizontal movement at the top of the pier assuming a full cross section composed of plain concrete. However, the moment of inertia was reduced by 50 percent in an attempt to model a potential cracked section. Residual strengths were used to model the potentially liquefiable terrace deposits below the groundwater level.

Our estimates are based on deflections at the top of the pier (ground surface) and no factor of safety has been applied to the estimated loads. Our estimated lateral capacities and maximum moment for drilled cast-in-place piers of various diameters are provided below. Pile deflection data as a function of depth as well as other lateral capacity data can be provided if needed.

Table – 3 Estimated Lateral Pier Capacity

Pier Diameter	Head Conditions	Estimated Maximum Lateral Load (kips)	Estimated Maximum Moment (kip-feet)
24-inch	Free-Head Fixed-Head	15 35	60 165
30-inch	Free-Head Fixed-Head	25 55	117 305

Resistance to lateral loads can also be provided by passive pressure acting on the sides of piers caps or grade beams if the existing soils within 5 feet of the pier caps and grade beams are replaced with compacted fill. Passive resistance can be provided according to our recommendations presented in Section 7.3.2.

If the design incorporates pier groups, we recommend that additional analyses be performed to model the lateral load capacity and deflection behavior of the group.

Pier Construction. Prior to placing concrete, loose and disturbed materials should removed from the bottom of pier excavations. In general, the bottoms of friction piers can typically be prepared using augering equipment. End bearing piers can be used to provide additional capacity, and should be cleaned more carefully than friction piers by using "special" cleaning buckets. In addition, drilled pier shafts should be reamed to remove material smeared on the side walls. Very hard zones were encountered in drill holes excavated for the Leroy Crandall (1965) study report for the Bio-II building project. In addition, hard zones have been encountered in previous drilled pier foundation excavations (e.g. the Humanities and Social Sciences Building). Typically, the very hard, siliceous zones are less than 1 to 5 feet thick. Localized coring has been used in the past to penetrate those siliceous materials

Drilled piers excavated through the fill and terrace deposits will likely require casing to support the excavation through those materials. The project specifications should provide for shoring or casing as needed to allow for cleaning and observation of the excavations, and for the placement of concrete.

Concrete should be poured under dry conditions and be placed neat against relatively undisturbed Sisquoc Formation. The concrete for piers should be placed through down-hole funnels, or similar provisions and in such a manner that the concrete does not strike the side of the pier shaft. Concrete used for drilled pier construction should have a high level of workability with a slump on the order of at least 6 inches. Concrete should be placed the day the drilling is completed. A pier excavation should not be allowed to stand open overnight. In general, a minimum of 24 hours should be allowed between placing concrete in one pier shaft and the drilling of nearby pier shafts within four pier diameters, center to center, or within three bell diameters, edge to edge.

We recommend geotechnical observation of pier construction be made to verify that the pier excavations extend the minimum required depth into moderately weathered Sisquoc Formation, and to review the bearing materials. All applicable safety requirements, including the use of casing if necessary, is the contractors responsibility. The project specifications should provide for variations in the depth of the pier excavations and for expected variations in the depth and hardness of the Sisquoc Formation.

7.3.2 Shallow Foundations

As an alternative to using drilled piers, the proposed structures can be supported on a shallow foundation system consisting of continuous, individual, or combined spread footings. If the potential for liquefaction and the estimated settlement-related effects of liquefaction are considered acceptable, proposed footings can be supported on compacted fill underlain by native terrace deposits. However, if the potential for liquefaction is not considered acceptable and UCSB desires to utilize a shallow foundation system, we recommend that the terrace deposits be removed down to the Sisquoc Formation bedrock and be recompacted. Footings can then be supported on the compacted fill material. Alternatively, the building could incorporate a basement level with the footings supported on Sisquoc Formation bedrock.

Allowable Bearing Pressures. Footings founded on compacted fill can be designed assuming a maximum net allowable bearing pressure of 2,000 psf. Footings supported on Sisquoc Formation bedrock can be designed assuming a maximum net allowable bearing pressure of 6,000 psf. The maximum net allowable bearing pressure can be increased by one-third when considering short-term wind or seismic loads. Footings should be embedded at least 2 feet below the lowest adjacent grade or floor slab, whichever is deeper.

Resistance to Lateral Loads. Resistance to lateral loading can be provided by sliding friction acting on the base of spread footings combined with passive pressure acting on the sides of foundations. We suggest a coefficient of friction of 0.35 be used to estimate the resistance to sliding for footings bearing on compacted fill or bedrock.

For resistance to lateral loads derived from passive pressure, we recommend using a value of 350 pounds per cubic foot (equivalent fluid weight) to estimate the passive resistance acting on the sides of foundations, stem walls, and grade beams bearing against compacted fill or Sisquoc Formation. Passive resistance should not be used for the upper one foot of soil that is not constrained at the ground surface by a slab-on-grade or pavement. A one-third increase in the passive value can be used when considering short-term wind or seismic loads. The friction resistance and passive resistance provided above are ultimate values and appropriate factors of safety should be used when evaluating foundation overturning or sliding.

Settlement Estimate. We estimate that total foundation settlements under static loading conditions should be less than about 1 inch, provided that footings are constructed as recommended herein. We estimate that differential settlements should be less than 1/2 inches between similarly loaded isolated footings or between adjacent columns supported by continuous footing elements. The above settlements represent estimates of static settlement and do not include potential settlements from liquefaction.

7.4 Slab-on-Grade and Exterior Concrete Slabs

The floor slab for the proposed structures may consist of a slab-on-grade supported on compacted fill. Floor slabs that will have moisture-sensitive floor coverings should be underlain by at least 4 inches of imported clean sand (e.g. washed concrete sand). A vapor barrier should be provided at the mid-depth of the sand layer. From a geotechnical standpoint, we suggest the slab-on-grade have a minimum thickness of 4 inches. Alternatively, concrete thicknesses can be estimated using a modulus of subgrade reaction of 150 pci for compacted fill. Samples of the subgrade soils in the concrete pavement areas should be evaluated for expansion potential at the time of grading. This will allow for the soil conditions to be reviewed and allow for modifying reinforcing requirements if necessary.

Concrete slabs for walkways and patio areas should be at least 4 inches thick and placed on a subgrade prepared as described in Section 7.2.8. The expansion potential of the terrace deposits is considered to be in the very low to low range, however, reinforcing steel should be provided. In addition, the design of exterior slabs should provide for control joints and expansion joints.

7.5 Building Code Criteria

The project location is within Uniform Building Code (ICBO, 1997) Seismic Zone 4 (Z factor of 0.4). In accordance with the Uniform Building Code descriptions, the soil profile at the site can be approximated as one with dense or stiff soil conditions exceeding a depth of 100 feet. For use with the code provisions, we recommend that the proposed structures be designed using a seismic coefficient for very dense soil/soft rock conditions (Type S_C). Near-source factors should be determined assuming that the significant local or regional faults are Type B seismic sources with the closest distance to known seismic sources of less than 2 km.

Selected code coefficients recommended for the project are listed below:

- Na 1.3
- Nv 1.6
- Ca 0.52
- Cv 0.90
- Ts 0.689
- To 0.138

7.6 Retaining Walls

7.6.1 Static Earth Pressures

Retaining walls that are free to rotate or translate laterally (e.g. cantilevered retaining walls) through a horizontal distance to wall height ratio of no less than about 0.004 are referred to herein as unrestrained or yielding retaining structures. Such walls can generally move enough to develop active earth pressure conditions. Retaining structures that are unable to rotate or translate laterally (e.g. restrained basement walls) are referred to as restrained or non-yielding walls. We have assumed that walls proposed for the project could consist of both unrestrained walls and restrained walls.

Backfill material placed behind retaining structures should conform to the recommendations for retaining wall backfill outlined in Section 7.2.11. Backfill behind retaining structures should be placed within a wedge extending up from the heel of the footing at an angle of at least 45 degrees from vertical. Retaining wall backfill material that supports foundations, floor slabs, or pavements should be compacted to at least 95 percent relative compaction.

Active and at-rest earth pressures for the design of proposed retaining structures can be estimated using the equivalent fluid weights provided below assuming horizontal backfill behind the wall.

Table -4 Earth Pressures

Wall Loading Condition	Lateral Earth Pressure Condition	Equivalent Fluid Pressure (pcf)
Unrestrained	Active	35
Restrained	At-rest	65

The tabulated values are based on a soil unit weight of 125 pounds per cubic foot (pcf). The values do not provide for hydrostatic forces (e.g. standing water in the backfill materials) or for surcharge conditions resulting from vehicle traffic or heavy compaction equipment.

The equivalent fluid weights should be applied to a vertical plane passing through the back most part of the heel. The height of the vertical plane should extend from the point where the vertical plane intersects the ground surface down to the elevation of the lowest retaining wall foundation element (e.g. bottom of shear key).

We recommend the retaining structures be designed for a factor of safety of 1.5 for sliding and overturning. Frictional and passive resistance can be combined when evaluating the potential for sliding or overturning of the foundation.

7.6.2 Surcharge Pressures

Surcharge loads induce additional pressures on earth retaining structures. Uniform area surcharge pressures for below grade walls may be assumed equal to one half of the applied horizontal surcharge pressure. Lateral pressures for other surcharge loading conditions can be provided, if required.

7.6.3 Drainage Measures

If drained lateral earth pressures are used for permanent structures, then drainage measures should be implemented to reduce the buildup of hydrostatic pressures behind below grade walls. To reduce the buildup of hydrostatic pressures behind below grade walls, we recommend that a free draining backfill, at least 2 feet in thickness, be placed behind the wall. The free draining material should conform the requirements of free draining backfill described in Section 7.2.11. A nonwoven filter fabric should be placed between the free draining material and the retaining wall backfill to limit migration of fines into the drainage material. The filter fabric should conform to the requirements for filter fabric described in Section 7.2.11.

In lieu of providing a 2-foot layer of granular drainage material and filter fabric behind the wall, prefabricated drainage structures (e.g. Miradrain, manufactured by Marif, Inc., or similar) can be used behind the retaining structures. Manufacturer recommendations for product installation should generally be followed, although those recommendations should be reviewed by the design engineer.

7.6.4 Compaction Adjacent To Walls

Backfill within 5 feet, measured horizontally, behind the retaining structures should be compacted with lightweight, hand-operated compaction equipment to reduce the potential for creation of large compaction-induced stresses in the walls. If large or heavy compaction equipment is used, compaction-induced stresses can result in increased lateral earth pressures on the retaining walls in addition to those presented above. If anything but lightweight, hand-operated compaction equipment is to be used, further evaluation of the potential for compaction-induced stresses is recommended.

Backfill material should be brought up uniformly around the below-grade or retaining walls (i.e., the backfill should be at about the same elevation all around the wall as the backfill is placed). That is, the elevation difference of the backfill surface around the wall should not be greater than about 2 feet, unless the wall is designed for those differences.

7.7 Pavement Design

Two R-value tests were performed as by Fugro part of the work for the Marine Science Research Building (2001) and the pavement recommendations provided in Fugro (2001) are considered applicable to the proposed Ocean Science Education Building project. A summary of those pavement recommendations for assumed traffic indices of 4 to 8 is provided below. The final pavement design sections should be evaluated on the basis of additional R-value tests performed during rough grading in the pavement areas.

Table – 5 Pavement Sections

Pavement Area	Recommended Pavement Sections				
	TI=4	TI=5	TI=6	TI=7	TI=8
Roadway Areas	- -	- -	0.30' AC 0.8' AB	0.35' AC 1.0' AB	0.40' AC 1.1' AB
Parking Areas	0.25' AC 0.4' AB	0.25' AC 0.6' AB	0.30' AC 0.8' AB	- -	- -

Prior to placing pavement materials, the existing subgrade should be overexcavated at least 1 foot below the bottom of the pavement section or 2 feet below the existing ground surface whichever results in the larger overexcavation depth. We note that additional overexcavation may be required to remove unsuitable soils. The exposed subgrade should be moisture conditioned as necessary and compacted to a minimum of 95 percent relative compaction. Following compaction of the overexcavation subgrade, the overexcavated material can be replaced. Additional fill placed over the prepared overexcavation subgrade (including the 1 foot of overexcavated material) should be compacted to at least 95 percent relative compaction.

Pavement materials should conform to Sections 26 and 39 of the Caltrans Standard Specifications (or equivalent) for aggregate base and asphalt concrete, respectively. Subgrade and pavement materials placed in the pavement areas should be compacted to at least 95 percent relative compaction.

7.8 Corrosion

Selected samples of the terrace deposits and Sisquoc Formation were tested for corrosion potential in Fugro (2001). The results of those tests are provided below.

Table - 6 Summary of Corrosion Test Results

Drill Hole No.	Sample Depth (ft)	Material Type	Resistivity (ohms/cm)	pH	Chloride (ppm)	Sulfate (ppm)
1	3.5	Terrace Deposits	13,794	7.01	1.6	23.6
2	13-1/2	Sisquoc Fm	1,379	7.41	102	589

The data acquired for Fugro (2001) and Law/Crandall (1994) suggest that the terrace deposits above the groundwater level have a low to moderate potential for corrosion of buried ferrous metals and a low potential to affect concrete. However, the terrace deposits below the groundwater and the underlying Sisquoc Formation materials have a moderate to high potential for corrosion of buried ferrous metals and a moderate potential to affect buried concrete. We recommend that the chemical test data be reviewed by a corrosion specialist for the design of underground piping composed of ferrous metal.

The sulfate content of the terrace deposits above the groundwater are considered to be low and generally below the range where sulfate resistant cement is required. However, the sulfate content of the terrace deposits below the groundwater level and the Sisquoc Formation materials is relatively high (over 2,000 ppm reported in Law/Crandall, 1994). Thus we recommend that sulfate resistant cement be used for concrete that will be in contact with those materials.

7.9 Utility Trenches

Excavation of utility trenches can likely be accomplished with a backhoe. Trenches over 5 feet in depth should be braced or sloped in accordance with the requirements of (Cal) OSHA. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction recommendations. In general, backfill for service lines extending inside of the property should be compacted to at least 90 percent relative compaction. Where utility trench backfill is placed in pavement or building areas, backfill should be compacted to at least 95 percent relative compaction or backfilled with 2-sack (minimum) sand-cement slurry.

7.10 Surface Drainage Control

Site grading should be such that positive drainage away from the structure and pavements is provided, and so that water will not pond near the structures or run over slopes. We recommend that roof gutters or drainage systems be installed to collect roof water and to carry it away from the foundations. Surface drainage swales should be positioned to allow for rapid removal of rain and irrigation water away from the foundations.

7.11 Additional Services

We recommend that Fugro be provided the opportunity to review the civil and foundation plans and the specifications for the project. In addition, some of the conclusions and recommendations presented herein are based on assumptions made during our geotechnical studies and evaluations. To verify or disprove those assumptions, we recommend that a representative of our firm be present to observe subsurface geotechnical conditions as they are exposed. Therefore, we recommend that Fugro be retained during grading and foundation construction to observe compliance with the design concepts and geotechnical recommendations and to allow for design changes in the event that subsurface conditions or methods of construction differ from those anticipated. Our representative should test and/or observe all excavations, fill and backfill placement and compaction, and the construction of all foundation systems.

8.0 CLOSURE

Fugro prepared the conclusions and professional opinions presented in this report according to generally accepted geotechnical engineering practices at the time and in the region that this report was prepared. This statement is in lieu of all warranties, express or implied.

This report has been prepared for the exclusive use of the University of California at Santa Barbara, and its authorized agents, for the design of the proposed Ocean Science Education Building project. This report may not contain sufficient information for other parties or other uses. If any changes are made in the project described in this report, the conclusions and recommendations contained in this report should not be considered valid. Fugro should review any changes in the project, and modify and approve in writing the conclusions and recommendations of this report for those changes. This report and the figures contained in this report are intended for design-input purposes; they are not intended to act as construction drawings or specifications.

Soil and rock deposits vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions vary seasonally or for other man-induced and natural reasons. Therefore, we do not and cannot have complete knowledge of subsurface conditions underlying the site. The criteria presented in this report are based upon findings at the points of exploration and on interpolation and extrapolation of information obtained at the points of observation.

The scope of our services did not include the assessment of the presence or absence of hazardous/toxic substances in the soil, groundwater, surface water or atmosphere. Any statements in this report regarding odors or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic substances.

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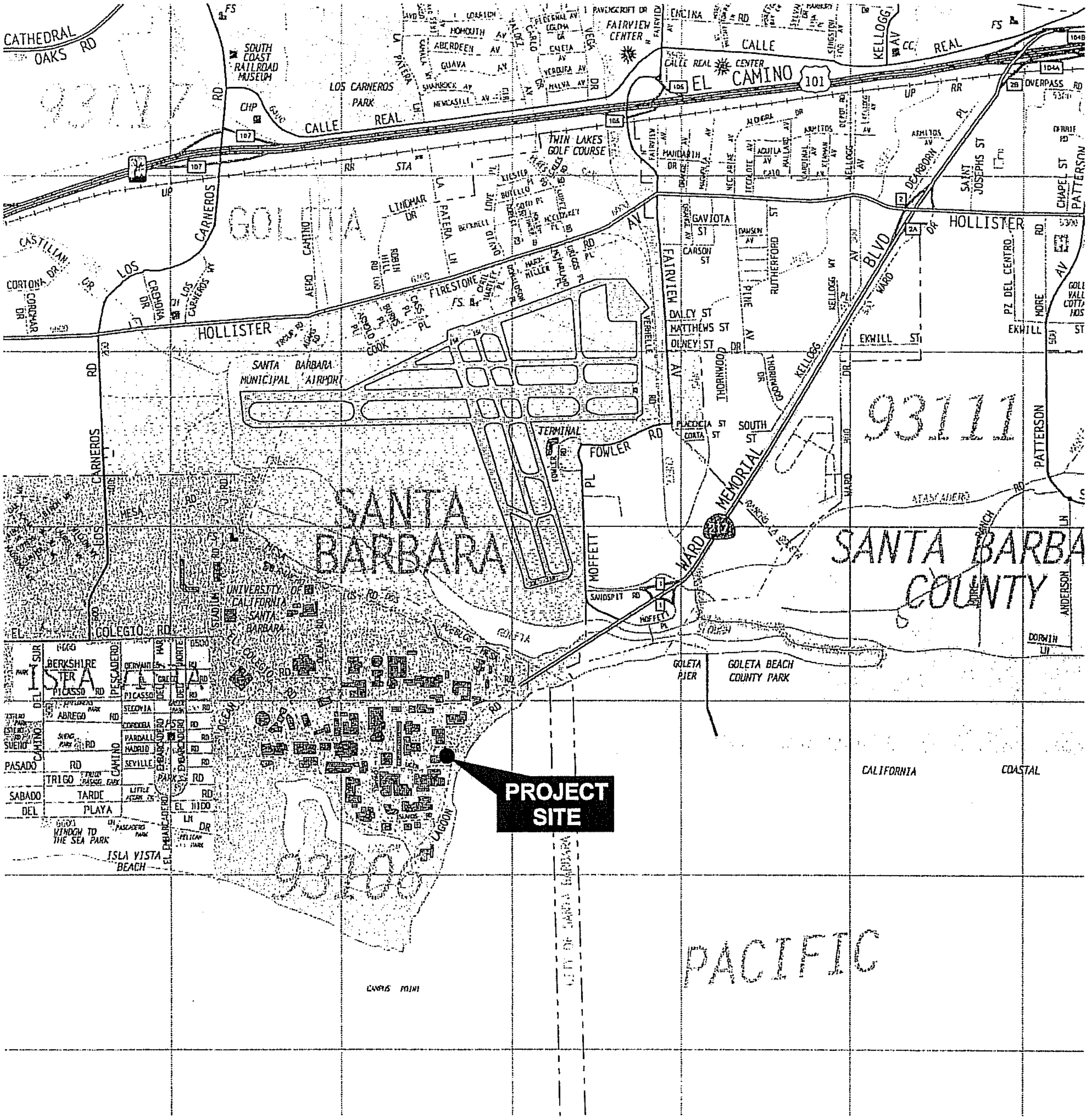

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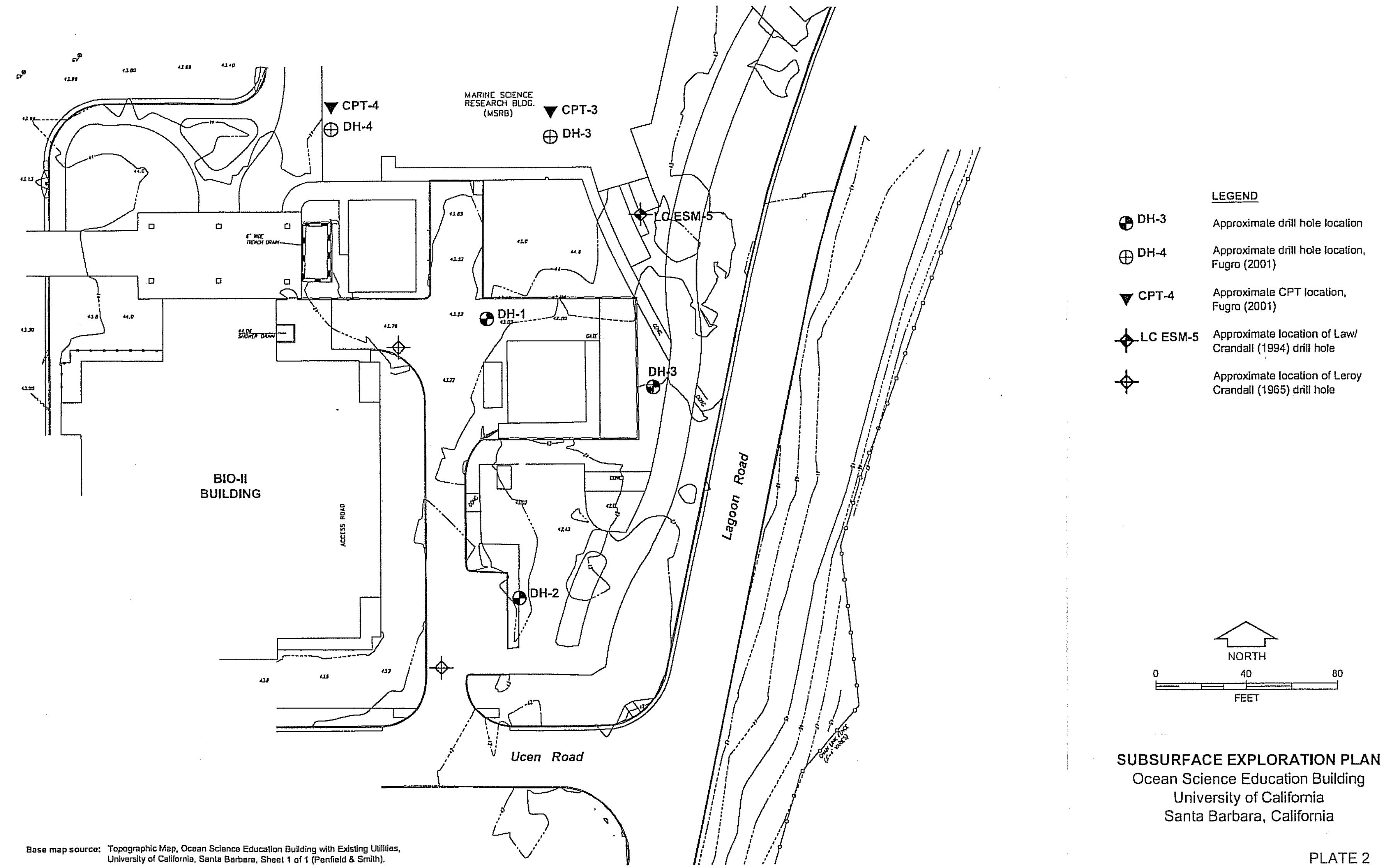
VICINITY MAP
Ocean Science Education Building
University of California
Santa Barbara, California



Base map source: Topographic Map, Ocean Science Education Building with Existing Utilities, University of California, Santa Barbara, Sheet 1 of 1 (Penfield & Smith).

SUBSURFACE EXPLORATION PLAN
Ocean Science Education Building
University of California
Santa Barbara, California

PLATE 2

APPENDIX A
FIELD EXPLORATION

APPENDIX A
FIELD EXPLORATION

The field exploration program for this geotechnical study consisted of drilling three exploratory drill holes on December 12, 2005. The field exploration program for this project was conducted in general conformance with our proposal dated July 22, 2004.

Drilling. The drilling subcontractor used for the project was S/G Testing Laboratory of Lompoc, California. The drilling subcontractor used a CME 75 hollow-stem-auger drill rig to drill the hollow-stem-auger drill holes. The drilling was performed under the observation of a Fugro staff engineer who prepared logs of the soil conditions and obtained soil samples for laboratory observations and testing. The soils were classified in the field according to the Unified Soil Classification System. The hollow stem auger drill holes were advanced to depths of about 20 feet below the existing ground surface. Drill holes were located in the field by pacing and siting from existing topography and features located at the site and as shown on the University Campus Atlas Sheets. Drill hole locations were reviewed by UCSB personnel and by Underground Service Alert for utility clearance.

Soil samples were obtained at approximately 2-1/2 to 5-foot intervals in the drill holes using either a driven modified California ring sampler or a standard penetration test sampler (SPT). The modified California ring sampler has a 3-inch outside diameter and a 2.37-inch inside diameter that contains 1-inch high rings. The SPT sampler has a 2-inch outside diameter and a 1-1/2-inch inside diameter. Samplers were driven using a 140-pound automatic trip hammer with a 30-inch drop. The number of blows needed to drive the sampler the last 12 inches into the soils was recorded, and is shown on the Log of Drill Holes. Bulk samples were collected from the cuttings. Recovered samples were placed in transport containers and returned to the laboratory for further classification and testing. The drill holes were backfilled with soil cuttings and capped with cold patch asphalt concrete after the completion of drilling.

Logs of the drill holes showing the depths and descriptions of soils encountered, geologic structure where applicable, vertical locations of samples, sampler blow count data, and results of density and moisture content tests, are presented on Plates A-1 through A-3 - Log of Drill Hole. A legend of symbols typically used on the drill hole logs is given on Plate A-4 - Key to Terms and Symbols Used on Logs. The logs represent the interpretation of field logs and tests, interpolation between samples, and the results of laboratory observation and tests. The stratification lines are approximate boundaries between soil types; the transitions can be gradational.

FUGRO

LOCATION: Approx. 170' N of Ucen Rd & 110' W of Lagoon Rd

SURFACE EL: 43 ft +/- (rel. MSL datum)

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
		3" Asphalt Concrete							
		3" Base							
		ARTIFICIAL FILL (af) Silty SAND (SM): medium dense, light to medium yellowish brown, moist							
1	(34)		116	108	8				
2	(35)		120	104	15				
		ALLUVIAL TERRACE DEPOSITS (Qt) Silty Fine SAND (SM): medium dense, light yellowish brown, orange oxidation staining							
3	19				18	32			
4	14				39	24			
5	9	- grades to dark greenish black, increased silt content, very moist, loose, at 9.5'			37	27			
6	12				47	36			
		- shell fragments, at 12.5'							
		SISQUOC FORMATION (Tsq) SILTSTONE (Rx) Elastic SILT (MH): moderately weathered, soft to moderately hard, poorly indurated, dark greenish gray, moist							
7	30								
8	(69)		108	73	49				

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: 20.5 ft
DEPTH TO WATER: Not Encountered Upon Completion of Drilling

DRILLING DATE: December 12, 2005

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

LOG OF DRILL HOLE NO. DH-1

Ocean Science Education Building
University of California, Santa Barbara, California

PLATE A-1

LOCATION: Approx. 50' N of Ucen Rd & 70' W of Lagoon Rd

SURFACE EL: 42.2 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
42				3" Asphalt Concrete							
				8" Base							
				ARTIFICIAL FILL (af) Silty SAND (SM): medium dense, medium brown, moist							
40	2	1	(29)		116	106	10				
38	4	2	(38)		116	102	14				
36	6			**ALLUVIAL TERRACE DEPOSITS (Qt)** Silty Fine SAND (SM): medium dense, light yellowish brown, orange oxidation staining							
		3	(40)		110	97	13				
34	8	4	18	- dark grayish black, silty fine sand, very moist, increased silt content			7	31			
32	10	5	11				34	21			
30	12	6	8	- loose sandy silt, at 11.5'			38	82			
		7	33	- shell fragments, at 13'							
28	14			**SISQUOC FORMATION (Tsq)** SILTSTONE (Rx) Elastic SILT (MH): moderately weathered, soft to moderately hard, poorly indurated, dark greenish gray, moist							
26	16										
24	18										
		8	50-5.5"								
22	20										
20	22										
18	24										
16	26										
14	28										
12	30										
10	32										
8	34										
6	36										
4	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: 20.5 ft
DEPTH TO WATER: Not Encountered Upon Completion of Drilling
DRILLING DATE: December 12, 2005

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

LOG OF DRILL HOLE NO. DH-2
Ocean Science Education Building
University of California, Santa Barbara, California

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LOCATION: Approx. 140' N of Ucen Rd & 30' W of Lagoon Rd

SURFACE EL: 42.7 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
42	0			**ARTIFICIAL FILL (af)** Silty SAND (SM): medium dense, medium brown, moist							
38	4	1	16	- hand auger to 4'			16	18			
37	5			**ALLUVIAL TERRACE DEPOSITS (Qt)** Silty Fine SAND (SM): medium dense, light yellowish brown, moist, orange oxidation staining							
35	7.5	2	17				19	12			
33	9.5	3	14				32	21			
32	10.5	4	8	- dark greenish black, increased silt, content very moist, at 10' - loose, at 10.5' - shell fragments from 11.5'-12'			36	32			
30	12	5	28	**SISQUOC FORMATION (Tsq)** SILTSTONE (Rx) Elastic SILT (MH): moderately weathered, soft to moderately hard, poorly indurated, dark greenish gray, moist							
23	19.5	6	42								

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: 20.5 ft
DEPTH TO WATER: Not Encountered Upon Completion of Drilling

DRILLING DATE: December 12, 2005

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

LOG OF DRILL HOLE NO. DH-3

Ocean Science Education Building
University of California, Santa Barbara, California

PLATE A-3

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

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-4

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

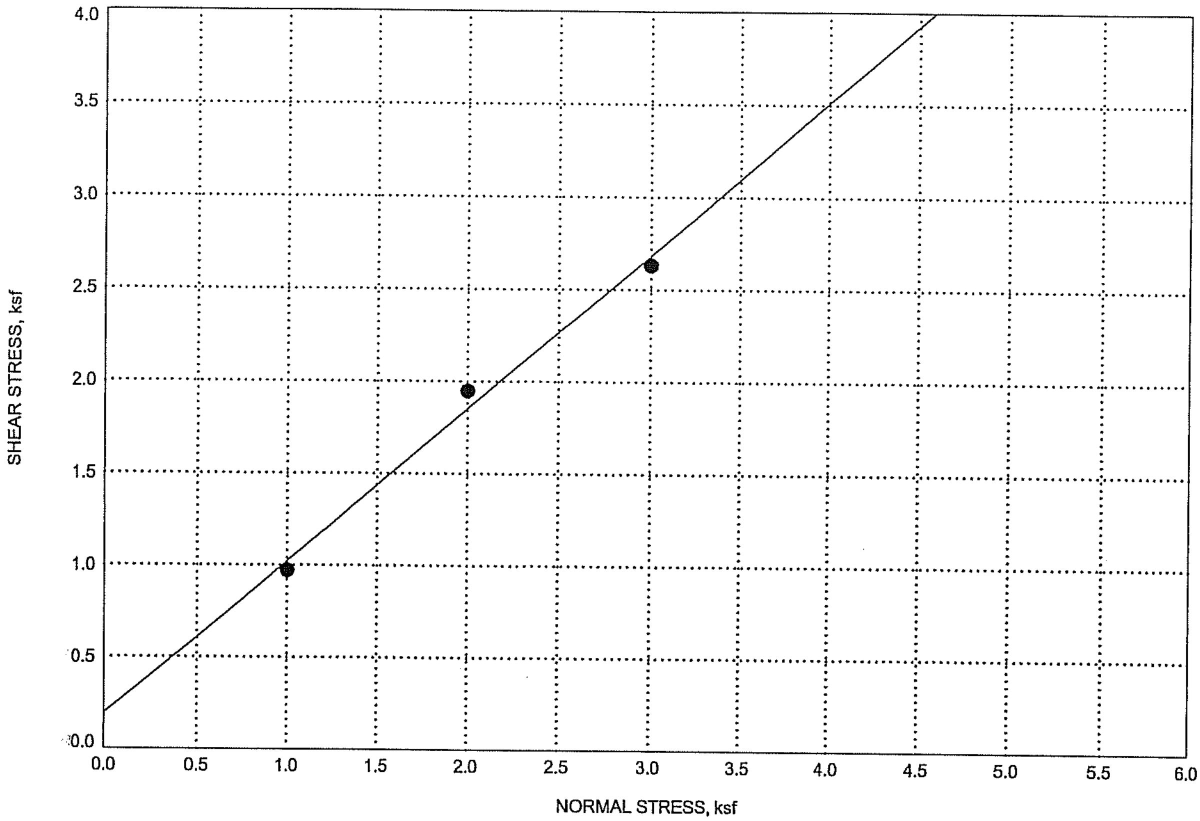
This appendix discusses the results of the laboratory testing program performed for this geotechnical study. Laboratory tests were performed on selected samples obtained from the field to help classify the soils encountered and to estimate some of their engineering properties. The program was carried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

Driven-ring 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.
- **Direct Shear Tests.** A direct shear tests was performed on a selected driven ring sample in general accordance with ASTM D3080 The results of the direct shear test is presented on Plate B-2 - Direct Shear Test Results.
- **Consolidation.** A one-dimensional consolidation test was performed on a selected driven-ring sample of terrace deposit materials. The sample was 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 sample at a loading pressure of 0.25 ksf. Rebound was measured under reverse alternate loading. Results of the consolidation tests are presented on Plate B-3 - Consolidation Test Results.
- **Expansion Index.** One expansion index test was performed on a selected sample of the siltstone bedrock material. The test was performed in general accordance with the Uniform Building Code Test Method 29-A. The sample was remolded and submerged in water, and the amount of expansion was recorded with a dial indicator. The results of the expansion index test is 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-1	2.0	1	Silty SAND (SM)	116	108	8																	
DH-1	3.9	B	Silty SAND (SM)																		0		
DH-1	4.0	2	Silty SAND (SM)	120	104	15																	
DH-1	7.0	3	Silty Fine SAND (SM)			18	32																
DH-1	8.5	4	Silty Fine SAND (SM)			39	24																
DH-1	10.0	5	Silty Fine SAND (SM)			37	27																
DH-1	11.5	6	Silty Fine SAND (SM)			47	36																
DH-1	19.0	8	SILTSTONE (Rx)	108	73	49																	
DH-2	2.0	1	Silty SAND (SM)	116	106	10																	
DH-2	4.0	2	Silty SAND (SM)	116	102	14																	
DH-2	7.0	3	Silty Fine SAND (SM)	110	97	13						0.2	40										
DH-2	8.5	4	Silty Fine SAND (SM)			7	31																
DH-2	10.0	5	Silty Fine SAND (SM)			34	21																
DH-2	11.5	6	Silty Fine SAND (SM)			38	82																
DH-3	4.0	1	Silty SAND (SM)			16	18																
DH-3	7.0	2	Silty Fine SAND (SM)			19	12																
DH-3	9.0	3	Silty Fine SAND (SM)			32	21																
DH-3	10.5	4	Silty Fine SAND (SM)			36	32																

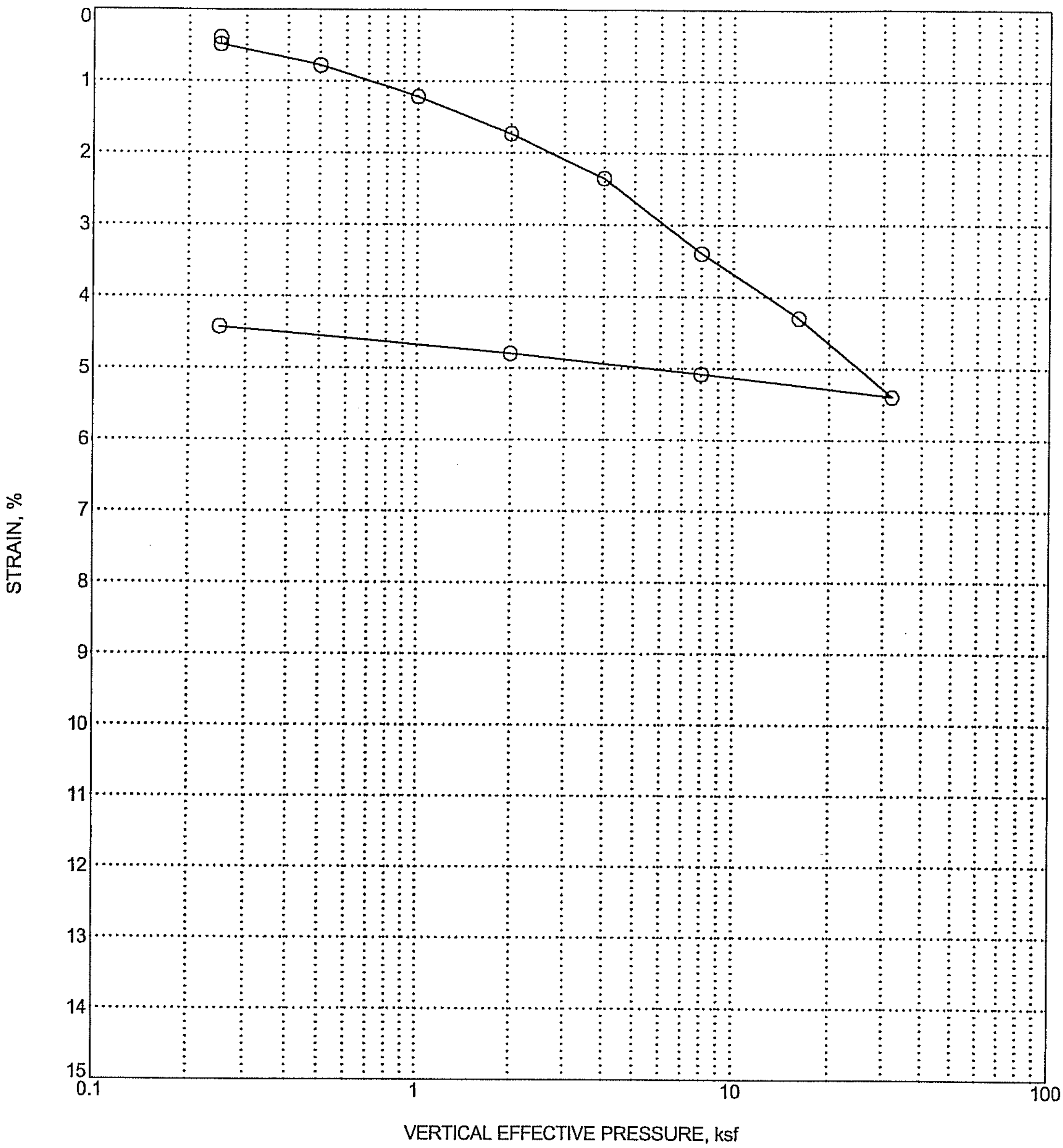
SUMMARY OF LABORATORY TEST RESULTS
Ocean Science Education Building
University of California, Santa Barbara, California



COHESION, ksf	0.2
ANGLE OF INTERNAL FRICTION, deg	40
LOCATION	DH-2
DEPTH, ft	7
MOISTURE CONTENT, %	20
UNIT DRY WEIGHT, pcf	91
MATERIAL DESCRIPTION	Silty Fine SAND (SM)
SAMPLE CONDITION	Ring Sample Driven Ring

DIRECT SHEAR TEST RESULTS

Ocean Science Education Building
University of California, Santa Barbara, California



LOCATION	DH-2
DEPTH, ft	7
INITIAL MOISTURE CONTENT, %	13
UNIT DRY WEIGHT, pcf	97
MATERIAL DESCRIPTION	Silty Fine SAND (SM)
SAMPLE CONDITION	Driven Ring

CONSOLIDATION TEST RESULTS
Ocean Science Education Building
University of California, Santa Barbara, California

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