APPENDIX F

GEOTECHNICAL REPORTS

APPENDIX F1: ***Geotechnical Study, Geologic and Seismic Hazards Evaluation, New Hospital Replacement Project***

APPENDIX F2: ***Geotechnical and Geohazard Evaluation, Outpatient Center No. 2***

APPENDIX F3: ***Phase-1 Geotechnical and Geohazard Evaluation, Central Utility Plant***

APPENDIX F4: ***Supplemental Geotechnical Recommendations, Bulk Liquid Oxygen Storage Tank Project***

APPENDIX F5: ***Preliminary Geotechnical Design Report, Caltrans Right-of-Way Decertification***

APPENDIX F1

Geotechnical Study, Geologic and Seismic Hazards Evaluation, New Hospital Replacement Project

FUGRO WEST, INC.

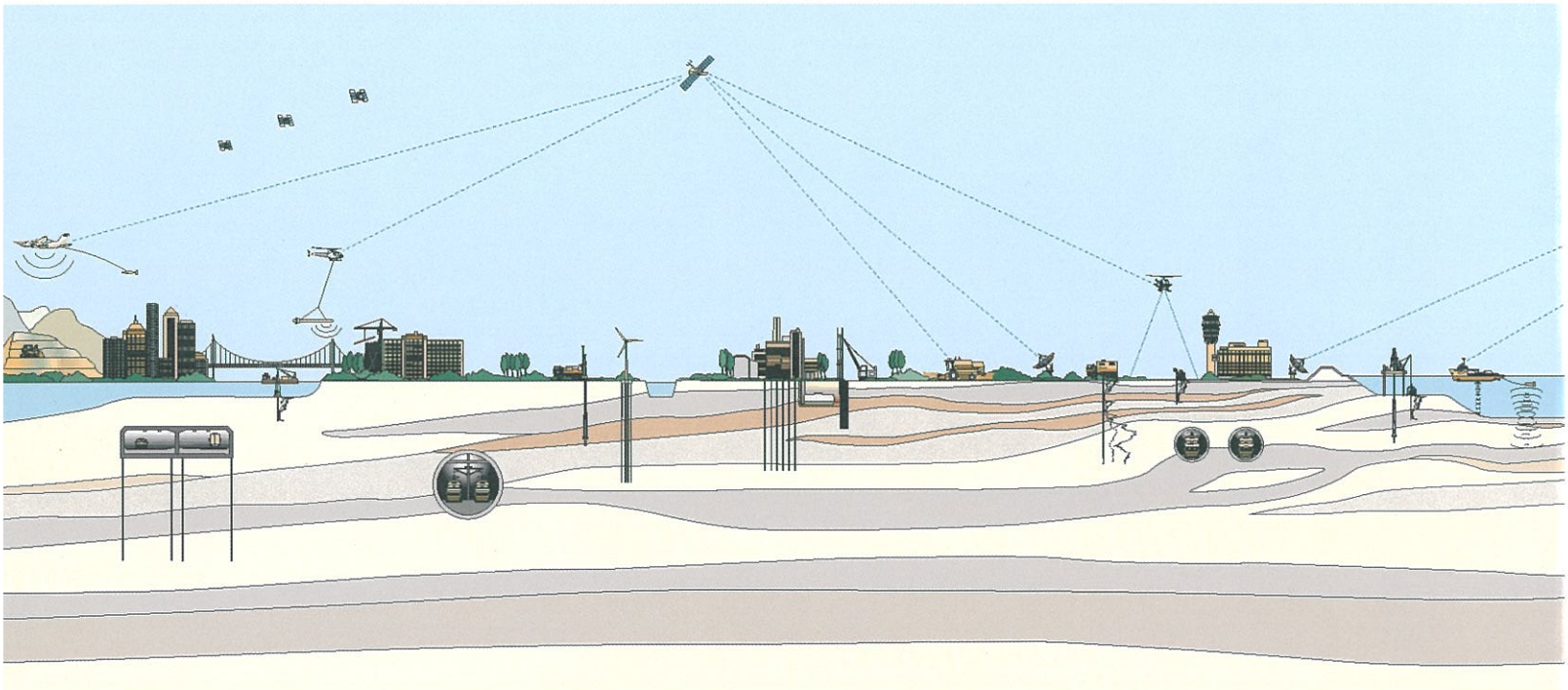
GEOTECHNICAL STUDY, GEOLOGIC AND SEISMIC HAZARDS EVALUATION NEW HOSPITAL REPLACEMENT PROJECT CHILDREN'S HOSPITAL & RESEARCH CENTER OAKLAND, CALIFORNIA

Prepared for:

CHILDREN'S HOSPITAL & RESEARCH CENTER AT OAKLAND
C/O HARRIS & ASSOCIATES

May 2008

Fugro Project No. 1595.002



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May 14, 2008
Project No. 1595.002

Children's Hospital & Research Center at Oakland
Doug Nelson c/o Sharon Knowlton
747 52nd Street
Oakland, California 94609

Attention: Mr. Doug Nelson

Subject: Geotechnical Study, Geologic and Seismic Hazards Evaluation
New Hospital Replacement Project, Oakland, California

Dear Mr. Nelson:

Fugro is pleased to submit this geotechnical study, geologic and seismic hazards evaluation report presenting the results of our field explorations, laboratory-testing program, and engineering analyses for the New Hospital Replacement Project, located in Oakland, California.

We appreciate this opportunity to be of service to you. Please contact us if you have any questions regarding the information presented in this report.

Sincerely,

FUGRO WEST, INC.

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CONTENTS

Page

1.0 INTRODUCTION 1
1.1 Project Description 1
1.2 OSHPD Compliance 1
1.3 Scope of Services 2
2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING 3
2.1 Review of Existing Data 3
2.2 Aerial Photograph Review 4
2.3 Field Exploration 5
2.4 Laboratory Testing 6
3.0 GEOLOGIC SETTING 6
3.1 Regional Geology 6
3.2 Regional Seismicity 8
3.2.1 Historic Seismicity 10
4.0 SITE CONDITIONS 11
4.1 Surface Conditions 11
4.1.1 New Hospital 11
4.1.2 Parking Structure and CUP 11
4.2 Site Geology 11
4.3 Subsurface Conditions 12
4.3.1 New Hospital 12
4.3.2 Central Utility Plant (CUP) and Parking Garage 13
4.4 Groundwater 14
5.0 GEOLOGIC HAZARDS EVALUATION 15
5.1 Surface Fault Offset Hazard 15
5.2 Shaking Hazards 16
5.2.1 Ground Motions 16
5.2.2 Liquefaction 17
5.2.3 Dynamic Densification 19
5.2.4 Lateral Spreading 20
5.2.5 Tsunami and Seiche 20
5.2.6 Landsliding 21
5.2.7 Other Shaking Hazards 21
5.3 Other Geologic Hazards and Complicating Site Conditions 22
5.3.1 Flooding and Inundation Due to Dam or Embankment Failure 22
5.3.2 Debris Flow 22
5.3.3 Land Subsidence 22
5.3.4 Soil Expansion 22
5.3.5 Corrosion Potential 23
5.3.6 Land Subsidence 25
5.3.7 Naturally Occurring Asbestos (NOA) 25

CONTENTS – CONTINUED

Page

5.3.8 Radon-222 26
5.4 Conclusions - Geologic Hazards 26
5.5 Other Hazards and Complicated Site Conditions Not Addressed In This Report 26

6.0 DISCUSSIONS AND CONCLUSIONS 27
6.1 Expansive Soils 27
6.2 Undocumented Fill 27
6.3 Groundwater 27
6.4 Foundation Support 28
6.4.1 New Hospital 28
6.4.2 Central Utility Plant (CUP) 28
6.4.3 Parking Structure 28
6.5 Construction Considerations 29
6.5.1 Existing Structures and Foundations 29
6.5.2 Excavation Shoring and Dewatering 29
6.5.3 Earthwork 30
6.5.4 Construction Quality Control 30

7.0 RECOMMENDATIONS 30
7.1 Seismic Design Criteria 31
7.2 Site Preparation and Grading 32
7.2.1 Site Preparation 32
7.2.2 Subgrade Preparation 32
7.2.3 Reworking Undocumented Fill 33
7.2.4 Engineered Fill Materials 34
7.2.5 Fill Placement and Compaction 34
7.2.6 Trench Backfill 34
7.2.7 Surface Drainage 35
7.3 Temporary Shoring and Construction Slopes 35
7.3.1 General 35
7.3.2 Temporary Construction Slopes 35
7.3.3 Temporary Shoring 36
7.3.4 Construction Dewatering 36
7.4 Foundations 37
7.4.1 Spread Footings 37
7.4.2 Mat Foundations 37
7.4.3 Drilled Pier Foundations 38
7.5 Lateral Loads 40
7.5.1 Long-Term Lateral Loads 40
7.5.2 Temporary Seismic Lateral Loads 41
7.6 Waterproofing and Designing for Hydrostatic Pressures 41
7.7 Concrete Slabs-on-Grade 41

CONTENTS – CONTINUED

Page

7.7.1 Interior Slabs-on-Grade 41
7.7.2 Exterior Slabs-on-Grade 42
7.8 Retaining/Below Grade Walls 43
7.8.1 Unrestrained Walls 43
7.8.2 Restrained Walls (Basement Walls) 44
7.9 Pavements 45
7.10 Additional Geotechnical Services 47

8.0 LIMITATIONS 48

9.0 REFERENCES 49

TABLES

Page

1. Aerial Photographs Examined 4
2. Large Magnitude (M≥ 6.0) Regional Earthquakes Within 100 Miles of the Site 10
3. Summary of Corrosion Test Results for Buried Concrete 24
4. Summary of Corrosion Test Results for Buried Ferrous Metal 25
5. 2007 CBC Seismic Design Parameters 31
6. Site Specific Seismic Design Parameters 31
7. Allowable Bearing Pressures for Spread Footings 37
8. Drilled Pier Design Skin Friction 39
9. Passive Soil Resistance (P) versus Displacement (Δ) Relationship 41
10. Recommended Pavement Design Alternatives 46

PLATES

Plate

Vicinity Map 1
Topographic Site Map 2
Site Plan 3
Geologic Map 5
Regional Fault Map 6a
Vicinity Fault Map 6b
Regional Epicenter Map 7
Cross-Section A-A' 8a
Cross-Section B-B' 8b
Cross-Section C-C' 8c
Cross-Section D-D' 8d
Cross-Section E-E' 8e
CGS Seismic Hazard Zone Map 9

CONTENTS – CONTINUED

Plate

Static Lateral Pressures on Unrestrained Walls 10a
Seismic Lateral Pressures on Unrestrained Walls 10b
Static Lateral Pressures on Restrained Walls 11a
Seismic Lateral Pressures on Restrained Walls 11b

APPENDICES

APPENDIX A FIELD EXPLORATIONS

Key to Soil Classification and Test Data ..... Plate A-1
Boring Logs ..... B-101 through B-112
Historic Boring Logs

APPENDIX B LABORATORY TESTING PROGRAM

Summary of Laboratory Test Results ..... Plate B-1
Plasticity Chart and Data ..... Plate B-2a
Plasticity Chart and Historic Data ..... Plate B-2b
Gradation Test Data ..... Plate B-3
Cerco Corrosion test results and Evaluation

APPENDIX C SEISMIC HAZARD ANALYSIS

APPENDIX D LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

1.0 INTRODUCTION

This report presents the results of the geotechnical study, geologic and seismic hazards evaluation conducted by Fugro West, Inc. (Fugro) for the New Hospital Replacement Project for Children's Hospital & Research Center at Oakland (CHRCO). The new hospital site is located on Dover Street, between 52nd Street and 53rd Street, in Oakland, Alameda County, California as shown on Plate 1 – Vicinity Map. The site coordinates are:

Latitude: 37.8380 º

Longitude: -122.2665 º

The project site is plotted on a 7 ½-minute United States Geologic Survey (USGS) quadrangle topographic map on Plate 2 – Topograpic Site Map.

1.1 PROJECT DESCRIPTION

As currently envisioned, the project includes construction of a new hospital facility at the site, and the demolition of the existing residential structures. The new structure will likely consist of a 12-story, approximately 250-bed hospital building. The footprint of the new hospital will be approximately 55,000 square feet. One basement level is currently planned beneath the hospital footprint, about 16 feet below grade. The new hospital structure will be located to the east of the existing Outpatient Center, as shown on Plate 3 – Site Plan. The site is currently developed as several residential houses, and other houses that have been converted to offices for the existing hospital. Dover Street, which bisects the location of the proposed hospital structure, will be removed (i.e., de-mapped) between 52nd Street and 53rd Street as part of the development.

The development plans also include construction of a Central Utility Plant (CUP) and a new parking garage. The CUP and parking garage will be located in a triangular shaped area on the south end of the CHRCO campus, as shown on Plate 3 – Site Plan. The current plan calls for a 5-story parking garage with about 500 spaces. The site is currently occupied by the hospital helipad and several support facilities for the hospital.

1.2 OSHPD COMPLIANCE

California hospitals are classified as critical structures according to California Code of Regulations, Title 24, of the California Building Code. The office of Statewide Health Planning and Development (OSHPD) reviews proposed new construction of hospital facilities. California Geological Survey (CGS) serves under contract as the advisor to OSHPD for engineering geology and seismology issues, which includes the review of geologic hazard reports.

This report was prepared with regard to satisfying the guidelines presented in *California Geological Survey – Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.* The most recent revision of Note 48 is October 2007.

One of the CGS requirements is that approximately one subsurface exploration point be completed for every 5,000 square feet of proposed building footprint area. Previously completed subsurface explorations and laboratory data can be used to partially satisfy this requirement. New borings, which are a part of this study, represent the additional borings necessary to meet the 5,000 square foot criteria. The locations of the borings were chosen based on accessible areas of the site to adequately characterize subsurface conditions at the location of the new hospital building.

The new hospital and CUP are considered critical structures, and as such need to satisfy the criteria of Note 48, and are subject to review by OSHPD and CGS. We note the parking structure is not considered critical, and does not fall under the regulations of Note 48.

1.3 SCOPE OF SERVICES

The purpose of our geotechnical field exploration and laboratory-testing program was to obtain information on subsurface conditions to evaluate the geotechnical aspects of the various portions of the site where structures will be located. The scope of our services included:

- Compiling and reviewing available geotechnical and geologic data, that is contained in our files and is pertinent to the project vicinity. This includes a series of previous reports prepared for various developments at the site;
- Conducting a field exploration and laboratory-testing program to supplement the available information on subsurface conditions at the proposed development locations;
- Evaluating the geologic hazards at the site (a stability evaluation of the existing Caltrans Highway 24 entrance ramp is beyond this scope of work);
- Performing engineering analyses of the collected data and developing geotechnical recommendations and design criteria for the proposed hospital redevelopment;
- Conducting a site-specific probabilistic seismic hazard analysis to generate horizontal response spectra, modifying the results to incorporate the near source effects, and applying factors to generate fault normal and fault parallel motions; and
- Preparing this geotechnical report presenting the results of our geotechnical field exploration, laboratory testing program, discussion of geotechnical issues, geologic hazards, seismic hazards and our geotechnical recommendations.

2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING

The exploration and laboratory-testing program described herein was developed to provide general characterization of the subsurface materials.

2.1 REVIEW OF EXISTING DATA

Prior to conducting our field exploration and laboratory testing, Fugro reviewed relevant information relating to geotechnical, geologic, and seismic issues at the site. Fugro's local predecessor companies have completed several geotechnical investigations at the Children's Hospital site since 1974. Historic boring locations from our previous work at the site are shown on Plate 3 – Site Plan. Data from these borings was used in preparing the cross sections presented in this report, and for our engineering analyses. We reviewed the results of these previous studies performed at the site, including the following reports that contain information directly related to the current project:

- Fugro (2002) "Western Expansion": Fugro performed a geotechnical study for the western expansion of the hospital. Fugro also completed a supplemental field exploration program including four soil borings and extensive lab testing. The results of this investigation were used to evaluate the liquefaction potential at the site as part of OSHPD Permit Number HS-002307-01. This report was used to supplement our current liquefaction evaluation.
- Harza (1997) "Children's Hospital Helistop": Harza Consulting Engineers (Harza) (local predecessor company acquired by Fugro) completed two soil borings for the hospital helistop. The helistop is located at the proposed parking garage and CUP area for the current project. The information in this report was used as a primary source of subsurface information for the proposed parking garage and CUP structures.
- Harza (1994) "Proposed Patient Services Pavilion, Phase I": Harza completed six borings as part of a geotechnical and seismology study for the proposed patient services pavilion. These borings extended to depths up to 51 feet below grade. Laboratory testing was completed on samples collected from these borings, including index and strength tests. Two borings were converted to monitoring wells to observe groundwater levels. This report was used to supplement our current investigation.
- Kaldveer (1990) "Proposed Medical Office Building and Parking Structure": Kaldveer Associates, Inc. (Kaldveer) (predecessor company acquired by Harza) completed four borings as part of a geotechnical study for the parking structure and office building on 52nd Street. Two of the borings (PB-2(90) and PB-4(90)) were located beneath the footprint of the proposed hospital structure for this 2008 study and were applied to satisfy CGS exploration requirements. This report was used as a primary source of data in our current investigation.
- Kaldveer (1987) "Foundation Investigation for Nursing Tower": Kaldveer completed two borings and related laboratory testing for the nursing tower on the south side of the Children's Hospital campus. These borings were drilled near the proposed

location of the new parking garage and CUP. We used these borings as a primary source of data in our current investigation.

- Kaldveer (1986) "Foundation Investigation, Family Center": Kaldveer completed four borings as part of a geotechnical study for the Family Center on Dover Street. These borings were located beneath the footprint of the proposed hospital structure for this 2008 study and were applied to satisfy CGS exploration requirements. This report was used as a primary source of data in our current investigation.

A complete list of references used in this study, including all previous studies performed by Fugro at the site, is included in Section 9.0 – References. Pertinent previous boring logs used to interpret subsurface conditions at the proposed structures are included in Appendix A.

2.2 AERIAL PHOTOGRAPH REVIEW

Six sets of vertical angle, black and white aerial photographs taken between 1930 and 1990 were examined both in stereo pairs and as single photos. Stereo views are used to determine relief and shape of landforms, which are commonly controlled by bedrock type and tectonic movement. Single photographs are reviewed for tonal variations and patterns that often go unnoticed in stereo because of the high relief. The historic photos were obtained from the Pacific Aerial Surveys library. A list of the photographs reviewed is provided in Table 1 below.

Table 1. Aerial Photographs Examined

Flight Date	Approximate Scale	Serial Number
6/12/90	1:12,000	AV3845-7-21/22
5/2/69	1:12,000	AV 902-07-17/18
7/25/63	1:36,000	AV 550-08-19/20
5/3/57	1:12,000	AV 253-08-21/22
3/24/47	1:20,000	AV 11-04-11/12
1930	1:9,500	GY-30-38/39/69/70

Each photoset was examined for changes in vegetation and geologic conditions related to soil movements and bedrock conditions. Interpretations are generally based on three factors: tone, texture and trend. Tonal and textural variations, if present, may be directly related to 1) the rocks or soils underlying the site, 2) water content of the underlying material, 3) natural vegetation (commonly controlled by local geology and water content), or 4) human activities (surface modification) including agriculture. Directional trends are observed as linear features.

The examination for this investigation was focused on the identification of landslides and active faulting. Landslides are identifiable through such features as headscarps, concave slopes, lateral drainages, hummocky terrain and lobate toes. Faults are expressed by offset

tonal or textural bands, tonal lineaments, isolated lines of springs or trees, abrupt changes in slope or drainage direction, and by streams with anomalously straight or curved courses.

No geologic hazards were observed in the aerial photos that could affect the proposed development. Our photograph review indicates no presence of historic landslides and no features indicative of active faulting at the site. Linear features associated with the Hayward fault are visible (1963 photo) about 2 miles east of the site. No other features relating to active faulting were observed in the vicinity of the project site.

The earliest aerial photographs reviewed for the site were from 1930. At that time, the area surrounding the hospital was developed as a predominately residential neighborhood. The site appears to be developed with an early hospital tower. However, the tower in the photo is not in the configuration of today. To the south of the site, in the vicinity of the intersection of Martin Luther King Way and 47th Street, Temescal Creek is visible. The location of Temescal Creek is shown on Plate 4 – Creek Location Map, and discussed in Section 3.1 below. The close proximity of this creek implies the presence of paleochannels beneath the site, created from a meandering path. No additional features indicative of geologic hazard are visible within or trending towards the subject property. No significant changes were made before the 1947 photos.

The 1957 aerial photograph shows the existing Children's Hospital tower building as the structure appears today. The hospital is surrounded on all sides by residential property. Temescal Creek is visible on the same path as earlier photos. No features indicative of any geologic hazard are visible within or trending towards the subject property.

By 1963 the main u-shaped hospital tower has been constructed. This structure is in the same configuration today, and residential neighborhoods continue to surround the site. Temescal Creek is visible to the south; however, more of the creek path has been diverted into underground culverts.

By 1969 additional hospital towers have been constructed directly northwest of the main tower. Caltrans has constructed Highway 24 in the right-of-way directly east of the hospital. The elevated roadway includes retaining wall and embankment slope with an approximate inclination of 2:1 (horizontal to vertical).

The 1990 photograph was the earliest photograph reviewed after the 1989 Loma Prieta Earthquake. No features were readily observed as being related to earthquake-induced damage, such as faulting, liquefaction, or lateral spreading.

2.3 FIELD EXPLORATION

Fugro's field investigation included drilling 12 new geotechnical borings. The borings were located in the field by our engineer based on existing site features, and to provide adequate coverage beneath the proposed hospital structure.

Taber Consultants, of West Sacramento, California, drilled the geotechnical borings between February 11 and 15, 2008. The borings, designated B-101 through B-111 (hospital area) and B-112 (CUP area), were drilled with a track-mounted drill rig equipped with 4-inch diameter solid flight augers and mud rotary drilling capabilities. The borings extended to depths of 25 to 80 feet below existing grade. All borings were backfilled with cement grout in accordance with our Alameda County drilling permit.

A Fugro engineer observed the drilling operation and logged the soil encountered. Our engineer recorded SPT N-values and estimated shear strength of the soils using a Pocket Penetrometer. Recovered soil samples were visually examined and classified in accordance with the Unified Soil Classification System (USCS) and ASTM D-2488 (visual method). Soil samples were transported to our office and laboratory for further examination, confirmation of classification and laboratory testing.

Detailed descriptions of the soils encountered in each of the borings are presented on the boring logs in Appendix A. The boring logs depict location specific subsurface conditions. The approximate locations of the borings were estimated by taping from existing landmarks and the locations should be considered accurate only to the degree implied by the method used. Elevations shown on the boring logs are in reference to the North American Vertical Datum of 1988 (NAVD88), and were estimated from contours on the project topographical survey provided by Chaudhary & Associates, Inc. The subsurface conditions encountered in the soil borings are summarized in Section 4.3.

2.4 LABORATORY TESTING

Geotechnical laboratory testing was conducted on the soil samples collected from the borings at Fugro's soil mechanics laboratory in Oakland, California. The geotechnical laboratory test program included: classification tests (gradation, fines content, Atterberg limits, water content, unit weight), and miniature vane strength tests. The results of the laboratory tests are presented on boring logs (Appendix A) at the appropriate sample depths, and in Appendix B - Laboratory Test Results.

Corrosion testing was conducted on three soil samples at CERCO Analytical, Inc., in Pleasanton, California. Corrosion potential is discussed further in Section 5.3.5 – Corrosion Potential. Results of the corrosion testing are provided in Appendix B.

3.0 GEOLOGIC SETTING

3.1 REGIONAL GEOLOGY

The site is located within the San Francisco Bay alluvial plain. This area, along with the Diablo Mountain Range, and the Santa Cruz Mountains are within the Coast Range Geologic Province, a belt of sedimentary, volcanic, and metamorphic rocks that extend from southern California to Oregon. The structural geology of the Coast Range is complex and dominated by transpressive stress caused by the interaction between the Pacific and North American plates. This stress, concentrated along faults within the San Andreas fault system, results in the series

of northwest-trending ridges and valleys that characterize the area. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates, subduction of the Pacific Plate beneath the North American Plate, and subsequent strike-slip faulting along the Proto-San Andreas, and San Andreas fault system. Strike-slip motion along the plate boundaries replaced subduction several million years ago (Atwater, 1970).

Geologic formations in the San Francisco Bay region range in age from Jurassic (190 to 135 million years ago) to recent Holocene (less than 11 thousand years ago). The oldest rocks are deformed Mesozoic sedimentary and volcanic rocks of the tectonically accreted Mesozoic Franciscan Complex and the contemporaneous Great Valley Sequence. During the last glacial maximum, significant relief of the bedrock was formed as a result of incision of creeks and streams to reach the elevation of the global sea level; approximately 140 feet lower than sea level today. As sea level rose with the onset of the current interglacial, lowland areas in the San Francisco Bay Area were subject to the deposition of a transgressive sequence of alluvial sediments, ranging in age from Pleistocene to the present. Younger alluvial, marine and marsh deposits have accumulated in the valleys in the region as a result of weathering in the surrounding mountains and sea level rise.

The regional geology, as mapped by Graymer (2000) is presented on Plate 5 – Geologic Map. Geologic units that are exposed within a mile of the site include the sandstone and quartz diorite members of the Novato Quarry terrane of the Franciscan Complex (Kfn, Kfgm), and surficial alluvial sediments (Qhaf, Qhl, Qpaf). Holocene aged alluvial fan deposits (Qhaf) underlie the site. Graymer (2000) described this formation as "alluvial fan and fluvial deposits, [a] brown or tan, medium dense to dense, gravelly sand or sandy gravel that generally grades upward to sandy or silty clay." An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site as shown on Plate 5 – Geologic Map. The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. The fault does not appear to impact the Pleistocene aged alluvial deposits mapped in the vicinity. The fault location is poorly constrained, and is not considered active, nor has the fault been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

Radbruch and Case (1967) map the site as being underlain by the Temescal Formation (Qtc). The Temescal Formation fills old channel meanders (of uncertain location) that were cut in the underlying San Antonio Formation (Qsu), and as such do not uniformly underlie the area. The thickness of the Temescal Formation varies from 3 to 18 feet. The nomenclature "Temescal Formation" was first used by Lawson (1914) to comprise several presumably contemporaneous alluvial units of different origin, lithology, and physical properties. The source material in the vicinity of the project location is assumed to be San Antonio Formation, which consists of "clay, silt, sand, and gravel, some pebbles". Most beds contain flakes or pebbles of white Claremont chert. Clays of this formation are often montmorillonite, and highly plastic. The maximum thickness of the San Antonio Formation is unknown.

Sowers (1993, rev. 2000) mapped modern day hydrology of Oakland and its vicinity, and where available, historic creek paths were included. Temescal Creek was mapped as being contained in a culvert, passing directly south of the project location (Plate 4 – Creek Location Map). As described in Section 4.3, Subsurface Conditions, paleochannels resulting from the meandering path of the creek underlie portions of the project site.

The regional geomorphology is generally controlled by the active Hayward fault, which is located approximately 2 miles east–northeast of the project. Local geomorphology is controlled by fluvial deposition associated with Temescal Creek.

3.2 REGIONAL SEISMICITY

Geologists and seismologists recognize the San Francisco Bay Area as one of the most active seismic regions in the United States. Dominated by the San Andreas fault system, the Bay Area is comprised of mostly northwest-trending strike-slip faults driven by the interaction of the Pacific and North American Plates. Movement between these two plates is predominantly accommodated on the San Andreas, Hayward-Rogers Creek, Calaveras, San Gregorio, and Concord-Green Valley faults. The major fault in the system is the San Andreas fault, a major rift in the earth's crust that extends for at least 450 miles.

In 2003, the Working Group on California Earthquake Probabilities (WGCEP), in conjunction with the United States Geological Survey (USGS), published an updated report evaluating the probabilities of significant earthquakes occurring in the Bay Area over the next three decades (2002-2031). WGCEP found a 62 percent probability that at least one magnitude 6.7 or greater earthquake will occur in the San Francisco Bay region before 2031. This probability is an aggregate value that considers seven principal Bay Area fault systems and unknown faults (background values). The San Francisco Bay region continues to be seismically active. The principal active faults in the Bay Area include the San Andreas, Hayward, Calaveras, and the San Gregorio faults. Earthquakes occurring along these faults are capable of generating strong ground shaking at the project site. The locations of significant Bay Area faults are shown on Plate 6a – Regional Fault Map, and Plate 6b – Vicinity Fault Map.

The closest active fault to the site is the Hayward fault. The Hayward fault was originally mapped by Lawson (1908), as part of his report to the State Earthquake Investigation Commission regarding the April 18, 1906 San Francisco earthquake. This report included a "comparison with other severe earthquakes in the same region", where he compiles data obtained from newspapers and eyewitness accounts. The fault was named after the city of Haywards (now Hayward), the site of the greatest damage.

The Hayward fault is the southern part of an extensive fault system that includes the Hayward, Rodgers Creek, Healdsburg, and Maacama faults. This system of faults extends over 280 kilometers (km) from San Jose to Mendocino County (Steinbrugge and others, 1987). The Hayward fault extends from the southwestern margin of the East Bay Hills for a distance of 105-km from San Pablo Bay in the north to an area near Mount Misery, east of San Jose, where slip is transferred to the Calaveras fault via a complex system of oblique slip and reverse faulting. The fault is divided into three segments: the 42-km long northern segment which

extends from the northwestern margin of San Pablo Bay to the vicinity of San Leandro, the 34-km long southern segment, which extends from San Leandro to the Warm Springs district of Fremont, and the 28-km long southeast Extension segment which extends from the Warm Springs District to the vicinity of Mount Misery (Bryant and Cluett, 2000). The segment closest to the project site is the northern Hayward fault. Based on earthquake focal mechanisms, the Hayward fault is near vertical. However, secondary fault traces are common and exhibit west-vergent structures that may merge at depth with the Hayward fault. Geomorphic evidence for the northern segment of the Hayward fault includes, offset stream channels, sag ponds, and distress to cultural features as a result of fault creep.

A robust recurrence interval has not been developed for this segment, but Lienkaemper and others (1997) concluded there have been at least four surface rupturing earthquakes in the last 2,250 radiocarbon years. The most recent event has been dated as after 1640 AD, but before 1776 AD. The 1836 magnitude 6.8 earthquake was originally believed to have occurred on this segment of the Hayward fault, however, recent work by Toppozada and Borchardt (1998) re-evaluated this earthquake to locate the epicenter further south, east of Monterey Bay.

The latest assessment of the U.S. Geological Survey Working Group on California Earthquake Probabilities (2003) has given the northern segment of the Hayward fault an 11 percent probability of exceedance, and the Hayward-Rodgers Creek fault in its entirety a 27 percent probability of exceedance, in the 30 year period ending in 2031 for a magnitude 6.7 or greater earthquake. Holocene displacement on the Hayward fault is predominantly right lateral, although varying degrees of vertical slip are indicated by the presence of escarpments along the fault's surface trace. Lienkaemper and Borchardt (1992) evaluated the aseismic slip rate for the Hayward fault. They found the area of the highest aseismic slip, 9 mm/yr (the southern Hayward fault segment in the vicinity of Fremont, California), and assumed that represented the minimum slip rate of the fault at depth. The historic surficial slip rate measured in the Oakland area only accounts for approximately 4 mm/yr of this total. It is assumed, therefore, the Hayward fault in this area is locked at depth and has the potential to rupture in large events.

The approximate distances of the site to the closest known mapped active faults are summarized in Table C-1 in Appendix C. Table C-1 summarizes the seismic sources and their pertinent characteristics. These data were used as part of the site-specific seismic hazard analyses discussed in Appendix C. The distances to faults were estimated using Boore et al. (1997). Fault parameters were adapted from the California Geologic Survey (CGS) fault database (2003).

Earthquakes on these or other active faults (including unmapped faults) could cause strong ground shaking at the site. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site, and other factors.

The project site is not located within an Alquist-Priolo Earthquake Fault Zone (discussed in further detail in Section 5.1 below).

3.2.1 Historic Seismicity

Since the first historical reports from the late 1700s, major earthquakes have been recorded along the San Andreas fault system in the Bay Area. Table 3 presents large magnitude ($M_w \geq 6.0$) earthquakes that have occurred within 100 km of the site between 1800 and 2007. The locations of the closest historic earthquakes to the project are shown on Plate 7 – Regional Epicenter Map. The computer program EQSEARCH, v3.00 (Blake, 2001) was used to compile the seismicity information presented in Table 2.

Table 2. Large Magnitude (M≥6.0) Regional Earthquakes Within 100 Miles of the Site

Epicenter Location	Date	Moment Magnitude	Distance (mi)	Distance (km)	Compass Direction to Epicenter
San Francisco	June 21, 1808	6.0	12.9	20.7	Southwest
Hayward	October 21, 1868	7.0	13.2	21.3	South
San Francisco	April 18, 1906	8.25	15.8	25.4	Southwest
San Francisco Peninsula	June 1, 1838	7.0	17.9	28.8	Southwest
Antioch	May 19, 1889	6.25	23.0	37.0	East
Mare Island	March 31, 1898	6.5	26.0	41.9	Northwest
San Jose	December 26, 1858	6.25	30.8	49.6	Southwest
Vacaville	April 19, 1892	6.5	41.5	66.8	Northeast
South Santa Cruz Mountains	October 8, 1865	6.5	42.2	68.0	South
1984 Morgan Hill	April 24, 1984	6.1	47.5	76.4	Southeast
1911 Morgan Hill	July 1, 1911	6.5	49.5	79.7	Southeast
Winters	April 21, 1892	6.25	49.9	80.4	Northeast
Loma Prieta	October 18, 1989	7.1	59.2	95.3	South
Gilroy	June 20, 1897	6.25	71.6	115.2	Southeast
Pajaro Gap	April 24, 1890	6.25	74.4	119.7	South
Pajaro Gap	October 18, 1800	7	79.4	127.8	South
Pacific Ocean	October 22, 1926	6.1	84.9	136.6	South

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The topography of the area surrounding Children's Hospital is shown on Plate 2 – Topographic Site Map. Note Plate 2 presents a USGS topographic map, with elevations referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). The local topography and surface conditions at each proposed structure are discussed below.

4.1.1 New Hospital

The proposed hospital structure is surrounded by 53rd Street to the north, 52nd Street to the south, an entrance ramp for westbound Highway 24 to the east and the existing parking structure and hospital office buildings to the west. The site is relatively flat, with a gentle slope increasing from about Elevation[1] 99 feet at the western end of the proposed hospital to about Elevation 105 feet at the eastern end of the proposed hospital. Directly east of the proposed hospital, the Highway 24 ramp abutment rises sharply to about Elevation 120 feet. Several houses (residential and hospital offices) and associated yards and driveways currently occupy the site. Many of the yards have small to medium size trees and various shrubberies. Dover Street, an asphalt-paved road, runs north/south between 52nd Street and 53rd Street through the center of the proposed structure. Dover Street will be removed between 52nd Street and 53rd Street as part of the development.

4.1.2 Parking Structure and CUP

The proposed parking structure and CUP are located in the southern portion of the Children's Hospital campus, and are surrounded by the existing hospital to the north, the Highway 24 entrance ramp to the east and Martin Luther King Jr. Way to the west. Several hospital laboratory and office buildings, as well as the hospital helipad structure currently occupy this site. In general, the surface in this area is asphalt paved, except for a few localized landscaped areas. The site is relatively flat, with a gentle slope increasing from about Elevation 100 feet at the southern tip of the proposed parking garage, to about Elevation 103.5 feet at the northern tip of the proposed CUP.

4.2 SITE GEOLOGY

The Children's Hospital site is located on a gradually sloping outwash plain located between the Berkeley Hills to the east and San Francisco Bay to the west. The native soils at the site consist of Quaternary age alluvial deposits likely including materials from both the San Antonio and Temescal Formations, according to Radbruch and Case (1967) and Helley, Lajoie and Burke (1972). Graymer (2000) maps the site as Holocene age alluvial and fluvial deposits, as shown on Plate 5 – Geology Map.

[1] Unless otherwise noted, all elevations in this report are in reference to the North American Vertical Datum, 1988 (NAVD88)

4.3 SUBSURFACE CONDITIONS

The alluvial stratigraphy encountered across the site typically consisted of a layer of lean sandy clay fill, over a layer of stiff lean clay, over interlayered clayey sands to sandy clays with varying amounts of gravel over stiff lean clay. The clayey sands encountered within the interbedded layer are interpreted to be the paleochannels associated with a meandering paleo-Temescal Creek. These deposits appear to be at various stages within the rubification process (e.g., the reddening of soils through the release and precipitation of iron as an oxide during weathering). In general, there appears to be two paleochannel deposits that may or may not be laterally continuous (see Plates 8a through 8e). One is shallow and appears to be relatively laterally continuous with a greater percentage of fines, than the deeper, more poorly graded deposit. These deposits may or may not connect at depth. Based on the relatively common high blow-counts (i.e., stiffness of the soils) for the underlying deposits, the near-surface material appears to have some antiquity. One previous 150-foot deep boring on 52nd Street, PB-1 (74), shows lean clay below 50 feet to a depth of 150 feet with two clayey sand layers encountered between 102 feet and 108.5 feet and 118 feet and 121 feet. The subsurface conditions at the site of each proposed structure are discussed below. The geologic units are described in order of increasing depth. Detailed descriptions of the deposits encountered in each of the borings are presented on the boring logs in Appendix A. Interpreted subsurface profiles are shown on Plates 8a through 8e.

The attached boring logs and related information depict location specific subsurface conditions, encountered during our field exploration. The passage of time could result in changes in the subsurface conditions.

4.3.1 New Hospital

The proposed hospital area north of 52nd Street is generally characterized by four layers, described below in order of increasing depth: I) lean clay fill; II) lean clay alluvium; III) interlayered clayey sand and sandy clay alluvium with varying amounts of gravel and IV) lean clay alluvium to the maximum depth explored of 150 feet.

The lean clay fill was encountered below the surface topsoil and asphalt paving and extended to depths ranging from 4 feet to 8 feet. This fill is generally characterized as dark brown to black, firm to stiff, lean clay with trace organics. The fill has a moderate to high expansion potential, with measured Plasticity Index ranging from about 22 to 28 percent. Fugro does not have documentation indicating that these fills were properly compacted during placement.

Below the fill layer, we observed a layer of stiff lean clay. This clay layer varied in thickness, ranging from about 6 feet to greater than 15 feet. We often observed a gradual transition into the interlayered clays and sands at the bottom of the layer. This unit is generally characterized as brown, stiff to very stiff, lean clay to lean clay with sand. Sand and gravel content was observed to increase with depth. Water content measured on samples in this layer ranged from about 18 to 27 percent, with an average of about 22 percent. Atterberg limits show the Plasticity Index in the stiff clay ranges from about 16 to 29 percent, indicating a moderate to

high expansion potential. Undrained shear strength (Su) measurements in this layer from pocket penetrometer and miniature vane tests ranged from about 1.5 ksf to 4.0 ksf.

Below the stiff clay, we encountered interlayered sandy clays and clayey sands with varying amounts of gravel. The individual clayey sand and sandy clay layers range in thickness from about 5 feet to over 10 feet. In B-105 (36 feet deep), the sandy clay layer was not readily distinguished from the upper stiff lean clay. This unit is generally characterized as grayish brown, medium dense to dense, clayey sand with gravel to clayey gravel with sand. Gravel content typically increased with depth. Laboratory testing showed the material to be well graded, with sandy material typically containing a significant amount of clay (at least 10 percent, and often higher than 35 percent). The sandy units were typically medium dense to dense, and the clay units were typically firm to stiff. Most borings terminated in this layer.

Below the interlayered sandy clay and clayey sand, we encountered very stiff to hard lean clay. This layer was typically encountered at about 45 to 50 feet below grade in the deeper borings at the proposed hospital site. This layer was observed to extend to a depth of 150 feet in a previous boring on 52nd Street (PB-1 (74)). Two clayey sand layers were encountered in PB-1 (74) between the depths of 102 and 109 feet and 118 and 121 feet. Recorded SPT N-values in this layer were very high, typically in excess of 30 blows per foot.

4.3.2 Central Utility Plant (CUP) and Parking Garage

Similar to the hospital site, the area at the southern end of the CHRCO campus is generally characterized by four layers, described below in order of increasing depth: I) undocumented sandy and clayey fill; II) lean clay; III) interlayered clayey sands and sandy clays with gravel and IV) lean clay.

Beneath the asphalt at the CUP and parking garage site is a layer of fill ranging from firm to stiff sandy lean clay to dense to very dense clayey sand. The fill was observed to range from black to yellowish-brown and contains varying amounts of gravel. This fill layer was typically observed to be about 4 to 6 feet thick beneath the CUP and the northern portion of the proposed garage. The thickness of the fill increases to about 11 to 18-feet thick towards the southern end of the proposed parking garage. The thicker fill is likely associated with an existing buried concrete box culvert at the southernmost portion of the site. Atterberg limits show the Plasticity Index in the fill ranges from about 12 to 21 percent, indicating a moderate expansion potential.

Beneath the fill at the CUP and parking garage site is a layer of stiff to very stiff lean clay with varying amounts of sand and gravel. This clay layer was consistently observed to be about 10-feet thick in the borings at and near the proposed CUP and parking garage. The thickness of the layer decreases to about 5 feet or less near the south end of the site, where the material was likely excavated and replaced with fill during construction of an existing culvert. Water content measured on samples in this layer ranged from about 14 to 24 percent, with an average of about 20 percent. Undrained shear strength (Su) measurements in this layer from miniature vane tests and unconfined compression tests range from about 3 ksf to over 4 ksf.

Below the stiff clay layer we observed interlayered sandy clays and clayey sands with varying amounts of gravel. The individual clayey sand and sandy clay layers range in thickness from about 5 to 10 feet. Gradual transitions were often observed between the sandy material and the clayey material. Laboratory testing indicates the sandy material typically contains a significant amount of clay (at least 10 percent and often higher than 35 percent) and the clay material contains significant amounts of sand. Both the sand and the clay layers contain a significant amount of gravel. The sandy units were typically medium dense to dense, and the clay units were typically firm to stiff.

Below the interlayered sands and clays, we observed a very stiff to hard lean clay layer. This layer was typically encountered about 40 to 45 feet below grade at the parking garage and CUP site. Recorded SPT N-values in this layer were very high, typically in excess of 30 blows per foot.

4.4 GROUNDWATER

Groundwater levels were observed to vary significantly across the site during our subsurface investigation. Groundwater was encountered as shallow as about 8.5 feet below grade in Boring B-101 and as deep as about 40 feet below grade in Boring B-104. Groundwater was typically measured within 30 minutes of completing the boreholes because the borings were backfilled immediately after drilling with cement grout in accordance with our Alameda County drilling permit. Note that the borings may not have been left open for a sufficient period of time to establish equilibrium ground water conditions.

As part of the geotechnical investigation for the Patient Services Pavilion, Harza (1994) installed two monitoring wells at the existing hospital. One monitoring well, EB-1 (OW), is located near the southern courtyard and the other monitoring well, EB-6 (OW), is located in the physicians parking lot near the entrance gate (approximate locations are shown on Plate 3 – Site Plan). Water levels were recorded in September of 1992, approximately 50 days after drilling, and in March of 2008 as part of this study. Groundwater in EB-1 (OW) was measured at 22 feet (Elevation 79 feet) in 1992, and at 21 feet (Elevation 80 feet) in 2008. For EB-6 (OW), groundwater was measured at 16 feet (Elevation 86.5 feet) in 1992 and 12.5 feet (Elevation 90 feet) in 2008.

Historically, groundwater levels at the Children's Hospital site have varied considerably. Observed levels range from 6 feet below grade in boring PB-3 (86b) (K876-3, 1986), at the Family House east of Dover Street, to 23 feet below grade in boring PB-5(86a) (K928-1, 1986), located in the current parking garage area. Boring PB-5 (86a) was converted into a temporary observation well and groundwater was recorded at a depth of 17 feet below grade three weeks after drilling. Near the Ambulatory Services Center, north of 52nd Street and to the west of Dover Street, groundwater was recorded at depths of 12 to 23 feet during investigations in 1986, 1990 and 1991.

The CGS Seismic Hazard Zone Report for the Oakland West Quadrangle (1991) presents a map that shows approximate depths to historic high ground water. Groundwater

depth contours presented on this map suggest historic high ground water at the Children's Hospital site to be on the order of about 7.5 feet below grade.

, Based on the depths of observed shallow groundwater in borings at the proposed hospital, CUP and parking garage sites, we have estimated a design ground water depth of 8 feet below ground surface for use in our analyses. This depth corresponds to an approximate elevation of 93 feet at the proposed location of the hospital. Fluctuations in the groundwater level may occur from changes in seasons, variations in rainfall, and other factors.

5.0 GEOLOGIC HAZARDS EVALUATION

Geologic hazards can be grouped into seismic and non-seismic categories. Seismic hazards can be further subdivided into surface fault offset, ground shaking, seismic ground deformation and water movements (tsunamis & seiches). Seismic ground deformation includes liquefaction, dynamic densification and landsliding. Certain types of ground deformation and water movement, however, can also occur under non-seismic conditions, so there is some overlap between seismic and non-seismic hazards. In general, the list of potential geologic hazards can also include such rare occurrences as mine collapse, hydrocompaction and peat oxidation. These hazards are rare and do not warrant discussion in connection with the project site as peat deposits were not encountered in the explorations. In addition, there is no mention of mines underlying the site in the literature we reviewed.

Historical evidence and the results of current seismological research indicate that at least one moderate to severe earthquake will occur sometime during the design life of the proposed development. Geologic hazards in this region typically become more critical during strong earthquakes. Detailed discussions of these hazards with respect to the site are presented below.

5.1 SURFACE FAULT OFFSET HAZARD

The potential for surface fault rupture at the site is low. No known active fault traces are mapped crossing the Children's Hospital Oakland site, based on existing geologic maps and reports including Graymer (2000) and Helley and Graymer (1997). The site is not located within a Fault-Rupture Hazard Zone as designated by the Alquist-Priolo Earthquake Fault Zoning Act (1972), as mapped on the official Earthquake Fault Hazard Zone Maps issued by the State of California (1982). The site is approximately 2 miles west of the closest active fault zone (Hayward fault) depicted on the Special Studies Zones map (CGS, 1982), as shown on Plate 9 – CGS Seismic Hazard Zone Map. The Fault Activity Map from the CGS digital database (CGS, 2000), as shown on Plates 6a (regional view) and Plate 6b (vicinity view), shows the Hayward fault to the east of the site.

An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site (Plate 5 – Geologic Map). The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. Based on Graymer (2000), this fault does intersect the ground surface within Pleistocene aged alluvial deposits near the site vicinity. The fault location is poorly constrained, and is not considered

active, nor has it been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

No indications of surface fault displacement were found in our reconnaissance of the site. While the possibility of new fault breaks developing during future earthquakes cannot be precluded, historic occurrences of surface fault rupture have generally followed pre-existing active fault traces.

5.2 SHAKING HAZARDS

Strong ground shaking at the Children's Hospital site will likely occur during a moderate to severe earthquake occurring on one of the active Bay Area faults. Strong ground shaking can cause the structures to shake and also has the potential to induce other phenomena that can indirectly cause damage to structures. These phenomena include: soil liquefaction, seismically induced waves such as tsunamis and seiches, inundation from dam or embankment failure, landsliding, lateral spreading, differential compaction, and ground cracking. Detailed discussions of these phenomena with respect to the site are presented in the subsequent paragraphs.

Youd and Hoose (1978) compiled data related to damage caused by historic earthquakes in the San Francisco Bay region, including the 1906 San Francisco and 1868 Hayward Earthquakes. To the south, in the vicinity of the Alameda and Oakland border, "the [1906] earthquake did not produce, relatively speaking, much destruction in these [Oakland, Alameda, and Berkeley water systems] works." Damage was limited to minor settlement and pipeline breaks, stream bank failures, and minor lateral displacements on the order of 8 inches. No features were noted in the project area as a result of the 1906 San Francisco Earthquake. In 1868, "portions of the wharves were carried away in some instances, while walls were cracked in almost every house". No features were noted in the Children's Hospital vicinity or surrounding areas as being related to liquefaction.

Liquefaction data is also available for the 1989 Loma Prieta Earthquake. Tinsley and others (1998) mapped liquefaction features nearby in Berkeley and Emeryville. Damage included sand boils, cracks, ground settlement, lateral spreading and pipeline breaks. "Lateral spreading and settlement caused extensive pavement damage to the freeway road surface. Pavement cracking was oriented parallel to the shoreline, with a total lateral movement of 30 to 120 mm. Several cracks were more than 50 m long." No other features indicative of liquefaction or lateral spreading were identified in the vicinity of the Children's Hospital site as a result of the 1989 Loma Prieta Earthquake.

5.2.1 Ground Motions

Site-specific probabilistic seismic hazard analyses were conducted to generate horizontal response spectra. The spectra were modified to incorporate the near source effects, and factors were applied to generate fault normal spectra. Detailed information regarding our site-specific seismic hazard evaluations are presented in Appendix C.

The design response spectrum and Maximum Credible Earthquake (MCE) events were developed per the 2007 CBC requirements and per the analysis procedures outlined in the design guidelines prepared by the American Society of Civil Engineers (ASCE), ASCE 7-05. Site-specific response spectra were developed for this project. The development of site-specific spectra included conducting probabilistic hazard analyses for 2475-year return period, as well as conducting deterministic checks that are required by ASCE 7-05. Detailed information regarding the development of site-specific spectra is presented in Appendix C. The design peak horizontal ground acceleration (PGA) for the site is 0.60g, based on our site-specific analyses (Table C-2).

The hazard analyses and ground motion evaluations were performed based on our understanding that the hospital structure will include one basement level and will be founded either on conventional spread footings, a mat foundation, or drilled pier foundations. If the foundation system is significantly changed, the applicability of the results presented in Appendix C should be re-evaluated with the selected alternate foundation system. The results presented in Appendix C are applicable only at the foundation level, based on the understandings mentioned above.

5.2.2 Liquefaction

The potential for liquefaction occurring at the site is considered low. Settlement can occur as a result of seismic shaking caused by liquefaction of subsurface soils. Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers located close to the ground surface. As very loose to medium dense saturated granular soils are shaken, their tendency to contract and compress may lead to the development of excess pore pressures. If the seismic shaking is strong enough and long enough, the buildup in pore pressure can produce a significant loss of shear strength.

Liquefaction is said to occur when the excess pore pressure equals the initial effective stress in the soil. If the shaking continues after the onset of liquefaction, ground distress may occur (e.g., sand boils, settlement, lurching, and lateral deformation). Liquefaction may also cause a loss of capacity of shallow or deep foundations, loss of lateral capacity or stiffness, and lateral ground spreading.

The susceptibility of a soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in a) distribution of grain size, b) soil density, c) aging, d) cementation, e) fines content, and f) plasticity characteristics of the fines. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface, a depth usually considered to be less than about 50 feet.

The liquefaction susceptibility at the project site is considered to be moderate, based on the Association of Bay Area Governments (ABAG) Liquefaction Susceptibility Map (2000). Witter and others (2006) mapped the quaternary deposits and evaluated the liquefaction susceptibility for the San Francisco Bay Area. The map shows the soils at the site as Holocene aged alluvial fan deposits, and as such, the liquefaction susceptibility is dependent on the depth

to groundwater in the vicinity. The liquefaction susceptibility is considered moderate for groundwater depths of 10-30 feet below ground surface and high for groundwater less than 10 feet below ground surface. The site has a moderate to high liquefaction susceptibility as mapped by Witter and others (2006). The site is located in an area designated as a zone of required investigation for liquefaction by California Geological Survey (CGS), as shown on the Seismic Hazard Zones Maps, West Oakland Quadrangle (CGS, 2003), included as Plate 9 – CGS Seismic Hazard Zone Map.

Our liquefaction evaluation was conducted using the procedures recommended by the National Center for Earthquake Research, and summarized by Youd et al (2001). The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.60g. This PGA represents the design basis earthquake (DBE) ground motion at the site, based on our probabilistic seismic hazard analyses, as discussed in Appendix C. An anticipated earthquake magnitude (maximum moment magnitude) of 7.4, representing the upper bound M_{max} for a full rupture of the Hayward-Rogers Creek fault, was used in conjunction with the PGA value assigned in the liquefaction analyses. The liquefaction-induced settlements were estimated using the procedure of Tokimatsu and Seed (1984).

The calculations of liquefaction potential and corresponding settlement are included in Appendix D. Our liquefaction analyses show that the materials beneath the proposed structures are generally cohesive and/or dense enough to have a low potential for liquefaction. This low potential for liquefaction in the new borings agrees well with previous liquefaction studies around the CHRCO campus that also found low potential for liquefaction. Our previous liquefaction studies included a detailed study for the western expansion of the hospital (OSHPD Permit Number HS-002307-01). This previous study included four soil borings and extensive lab testing to evaluate the liquefaction potential at the site. This detailed study showed the clayey sand layers to be "non-liquefiable" according to the procedure by Youd et al (2001). Additionally, liquefaction analyses of the borings performed as part of the geotechnical investigation for the medical office and parking structure by Kaldveer (1990) indicate the sand layers are medium dense to dense and liquefaction potential is remote.

However, our analyses identified two isolated borings (B-109 and B-112), with relatively thin layers of clayey sand that may be susceptible to liquefaction and subsequent densification and settlement following a design level earthquake. Boring B-109 (near proposed hospital) encountered a 4-foot thick layer of medium dense sand with clay at a depth of 36 feet. This thin sand layer is between two soft clay layers, and the clay material may have influenced the lower blow count. In addition, Atterberg limits show the fines within this sample behave primarily as clay (not silt) with sufficient plasticity (PI of about 16) to reduce the potential for liquefaction in this layer. Blow counts within the same layer in nearby borings are higher, and the potential for liquefaction is likely limited to a small portion of the site. Our analysis indicates less than ½ inch of liquefaction induced settlement could occur in this layer during a design level earthquake. Because this is likely an isolated layer, and the liquefaction would occur at a depth of about 35 feet, surface manifestation of settlement is expected to be negligible because of soil arching effects. If a mat foundation is used at the hospital, the mat should be designed to include an additional ½ inch of differential settlement caused by liquefaction within the potentially liquefiable area. Nearby borings about 75 feet away from B-109 do not show liquefiable

material, therefore the portion of the mat located within a 75-foot radius of B-109 should be designed for liquefaction settlement. If a drilled pier foundation is used, we believe the magnitude of settlement is not enough to produce significant downdrag loads and the impact of isolated liquefaction will not significantly impact the pier foundation system.

Boring B-112 (at the proposed CUP site) also encountered a potentially liquefiable layer of soil. We observed about 8 feet of clayey sand with gravel, beginning at about 14 feet below grade in Boring B-112. This layer had an average SPT N-value of about 17 blows per foot. Sieve tests show the fines content of this layer is about 14 to 15 percent. Atterberg limits run on the fine-grained portion of these samples behave primarily as clay (not silt) with sufficient plasticity (PI of about 17) to reduce the potential for liquefaction in this layer. Nearby borings within about 50 feet of B-112 show the clayey sand layer to be significantly denser than observed in B-112. The liquefaction potential at the CUP is likely an isolated area within what is typically a dense layer. We believe the potential for significant liquefaction in this layer is low. However, we conservatively estimated up to 1 inch of liquefaction-induced settlement could occur during a design level earthquake. Because nearby borings do not indicate a potential for liquefaction, the settlement should be treated as a differential settlement of 1 inch over a distance of 50 feet. The foundation system for the CUP should be chosen and designed to incorporate the possible liquefaction settlement.

The saturated clayey sand materials with the potential for liquefaction identified above are likely part of paleochannels associated with Temescal Creek. The isolated loose zones are within what are typically dense granular layers with low potential for liquefaction. The potential for differential settlement occurring between the transition from stiff clay to the clayey sand paleochannels is considered low because of the low potential for liquefaction in the paleochannels. The potential for significant liquefaction and large liquefaction-induced settlements or other ground failures at the site is considered low, based on the evaluation of the borings and our review of previous liquefaction studies at this site.

5.2.3 Dynamic Densification

Volumetric compression, or soil densification, is settlement caused by earthquake-induced ground shaking that causes soil particles to compress together. Dry cohesionless soils, such as sands and gravels, are susceptible to this type of settlement. Earthquake induced settlements of dry cohesionless soils depends on three main factors, including the relative density of the soil, the maximum shear strain induced by the design earthquake and the number of shear strain cycles which can be related to earthquake magnitude.

We used the methodology proposed by Tokimatsu and Seed (1987) to estimate the degree of dynamic densification potential in the unsaturated alluvial clay and clay fill above groundwater at the Children's Hospital site. This methodology is outlined below:

1. Estimate the shear stress in the soil induced by the earthquake (peak horizontal ground acceleration value);
2. Estimate the volumetric strain of the soil based on this shear stress value using the chart developed by Tokimatsu and Seed (1987);

3. Adjust the unidirectional volumetric strain to account for the multidirectional shaking effects of an earthquake;
4. If the moment magnitude earthquake being considered is different than 7.5, adjust the volumetric strain by a ratio value based on the earthquake magnitude; and
5. Based on the thicknesses of the dry cohesionless soil layers and the adjusted volumetric strain values, compute the total predicted settlement.

The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.60g. This PGA represents the design basis earthquake (DBE) ground motion at the site, based on our probabilistic seismic hazard analyses, as discussed in Appendix C. An anticipated earthquake magnitude (maximum moment magnitude) of 7.4, representing the upper bound M_{max} for a full rupture of the Hayward-Rogers Creek fault, was used in conjunction with the PGA value assigned in the dry dynamic densification analyses.

The calculations for the soil dry densification evaluation are included in Appendix D. The soils above groundwater at our site were primarily clay. No dynamic settlement was identified in any of our 12 borings, based on our evaluation. The potential for dynamic densification of unsaturated soils above groundwater at this site is considered low.

5.2.4 Lateral Spreading

Lateral Spreading occurs where the contact between a layer of liquefiable material and the material below is sloped. Saturated sands lose their strength during an earthquake and become fluid-like and mobile. As a result, the ground may undergo large permanent displacements that can damage underground utilities and well-built surface structures. Lateral spreading involves displacement of large blocks of ground down gentle (greater than 0.1 percent) slopes or towards the open face of slopes such as a stream channel. In areas where there is no open face, buckling of the overburden is often observed.

Historic earthquakes in the Bay Area have produced lateral spreading features. However, large-scale lateral spreading is considered unlikely at this site because the site is essentially level and the probability for liquefaction at the site is considered low, except in localized areas (discussed in Section 5.2.2). In addition, the transitions between cohesive layers and granular layers were observed to be gradual and occur at varying depths. Liquefaction-induced lateral spreading along the contact between cohesive materials and granular materials is unlikely. Liquefaction induced lateral spreading into buried paleochannels is unlikely because the paleochannels are typically dense and have a low potential for liquefaction, as discussed above.

5.2.5 Tsunami and Seiche

During a major earthquake, strong waves such as tsunamis or seiches may be generated in large bodies of water and may cause damage to structures affected by them. A tsunami (or seismic sea wave) is an open ocean phenomenon caused by faulting, volcanism or other abrupt movements on the ocean floor. A seiche is a wave that occurs in an enclosed

basin as a result of fault displacement in the basin bottom, large landslides into the basin, or from periodic oscillation or sloshing of the water in the basin in response to ground shaking.

The potential for tsunami and seiche hazards at this site is considered remote. The site is located at an elevation of about 100 feet according to the project survey, and about 1.8 miles east of San Francisco Bay. There are no large enclosed bodies of water in the vicinity of the site. Damaging tsunamis are not common on the California coast. Most California tsunamis are associated with distant earthquakes (often those in Alaska or South America), not with local earthquakes. Devastating tsunamis have not occurred in historic times in the Bay Area. Because of the lack of reliable information about the kind of tsunami run-ups that have occurred in the prehistoric past, there is considerable uncertainty over the extent of tsunami run-up that could occur at the site. However, the Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region (Ritter & Dupre, 1972) does not indicate that the site is in an area that may be inundated by a tsunami with a run-up of 20 feet at the Golden Gate Bridge.

5.2.6 Landsliding

A landslide is a movement of a mass of soil down a steep slope when the soil cannot support the weight of overlying soil or rocks. Landslides vary in size and rate of movement. Slides can occur slowly over time or suddenly. Areas susceptible to landslides are those where masses of soils are weakly supported because of natural erosion, changes in ground water or surface water patterns, or human activities such as undercutting. Heavy rains or seismic shaking can also trigger landslides.

A large earthquake in the San Francisco Bay Region may cause movement of active slides and could trigger new slides. However, no significant slopes are present at the site, and no evidence of previous or active slides were observed during our site investigation. The site is not within a mapped area of existing or potential slope instability. The site is not located within a State of California designated seismically induced landslide hazard zone, as shown on Plate 9 – CGS Hazard Zone Map. The site is not located within a City of Oakland landslide Hazard Zone (Miles and others, 2001). The potential for landsliding at the site is considered low.

The nearby Highway 24 entrance ramp to the east of the site includes retaining walls and embankment slopes with an approximate inclination of 2:1 (horizontal to vertical). This roadway is about 25 to 30 feet higher than the surrounding relatively flat terrain. We have assumed for purposes of this study that the Caltrans structures were engineered and constructed such that a low potential for sliding exists at the slope. An evaluation of the stability of the Caltrans Highway 24 entrance ramp embankment is beyond the scope of work for this study.

5.2.7 Other Shaking Hazards

Ground shaking during an earthquake could cause objects within buildings that are not rigidly attached to the building structure (such as desks and bookshelves) to undergo some differential movements with respect to the structure. Building construction should include

designs that reduce such potential differential movements and the adverse effects of such movements where they cannot be prevented.

We have also considered the possibility of the occurrence of other seismic hazards. Differential compaction is considered unlikely because of the abundance of very stiff clayey soils and bedrock and low potential for liquefaction. Ground cracking and lurching can be caused by any of the phenomena discussed above.

5.3 OTHER GEOLOGIC HAZARDS AND COMPLICATING SITE CONDITIONS

5.3.1 Flooding and Inundation Due to Dam or Embankment Failure

The site is located at an elevation of approximately 100 feet (relative to NAVD88) and about 1.5 miles west of San Francisco Bay. The potential for flooding from San Francisco Bay is remote. The site is not within the flood plain of any nearby water body. The National Flood Insurance Program designates flood prone areas. The site is designated as Zone C, an area of minimal flooding (FEMA, 1982). Flooding resulting from ponding of locally heavy rainfall was not included in our evaluation.

Dams and reservoirs, which hold large volumes of water, represent a potential hazard from failure caused by ground shaking. The site is in the path of inundation from failure of Lake Temescal dam (owned by the East Bay Regional Park District). The state's Division of Safety of Dams last inspected this dam in July 2002. At the time, no issues necessitating corrective action were presented and the dam was "judged satisfactory for continued operation" (City of Oakland General Plan, 2004).

5.3.2 Debris Flow

The project area is about 2 mile from the nearest zone designated as a debris-flow source area (Oakland Hills) in the Principal Debris-Flow Sources Areas in Alameda County map (USGS, 1997), and is therefore not likely to be impacted by debris flows.

5.3.3 Land Subsidence

Land subsidence is a gradual settling or sudden sinking of the Earth's surface caused by subsurface movement of earth materials. The principal causes of subsidence are aquifer-system compaction, drainage of organic soils, underground mining, hydrocompaction, sinkholes, and thawing permafrost. The conditions needed for these hazards are not known to exist within the project area.

5.3.4 Soil Expansion

The near surface clay fill soils observed at the proposed hospital location have Plasticity Index (PI) ranging from about 15 to 28 percent, based on laboratory testing from current and previous borings at the site. These PI values indicate a moderate to high expansion potential. In addition, an expansion index test was performed as part of the previous study for the adjacent office building and parking structure (Kaldveer, 1990). This test showed an Expansion

Index of 115 in the fill material, indicating a high expansion potential. These results suggest that the clayey soils can be classified as "expansive" as defined in Section 1802A.3.2 of the 2007 California Building Code.

The moderate to high expansion potential of the clayey surface soils encountered at the site is a primary consideration for foundation and floor slab design. These materials could be subjected to volume changes during seasonal fluctuations in moisture content. In order to reduce the potential impact of expansive soils on the proposed building resulting from swelling and shrinkage of these materials, we recommend that buildings be supported on deepened footings. In addition, we recommend that all interior slabs-on-grade be supported on a layer of imported non-expansive fill. The amount of required non-expansive fill can be reduced if reinforcement is provided in the slab to minimize the impact of expansion pressures.

Expansive materials surrounding the structures may be subjected to volume changes during seasonal fluctuations in moisture content, which could affect minor slabs-on-grade and landscaping hardscape such as sidewalks and exterior slabs.

5.3.5 Corrosion Potential

Corrosion testing was conducted on samples of the surficial material from B-104 at a depth of 2.5 feet, B-105 at a depth of 15.5 feet and B-112 at a depth of 4 feet. The corrosion testing was conducted at CERCO Analytical Laboratory in Pleasanton, California. The corrosivity testing included resistivity analysis, chloride ion concentration, sulfate ion concentration, pH testing, and redox potentials. The corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. The results of the laboratory tests and details of the results are presented in the letter reports provided by CERCO Analytical, Inc., attached in Appendix B.

The results of our corrosion tests generally show slightly more corrosive soils than identified in previous corrosion studies performed at the nearby hospital office and parking structure site by Kaldveer (1990). The Kaldveer study included two resistivity traverses and one sulfate content test. The historic resistivity tests indicated resistivity values ranging from 3,066 to 11,975 ohm-cm; indicating slightly to moderately corrosive conditions. The sulfate test indicated a sulfate content of 0.005 percent; suggesting negligible sulfate attack potential.

Tests for Buried Concrete

Soluble sulfate concentration, chloride ion concentration, and pH tests were performed on the three samples. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures. The table below presents the depth at which the samples were collected and the laboratory test results.

Table 3. Summary of Corrosion Test Results for Buried Concrete

Test No.	Boring	Depth	Sulfate (mg/kg)	Chloride (mg/kg)	pH
1	B-104	2.5 ft	110	52	6.8
2	B-105	15.5 ft	N.D.	N.D.	7.5
3	B-112	4 ft	33	N.D.	5.6

N.D. = none detected

CERCO's evaluation detected sulfate ion concentrations in tested samples ranging from none detected (N.D.) to 110 mg/kg. The sulfate concentrations are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel. Section 1904.3 of the 2007 California Building Code requires that concrete exposed to sulfate containing soils comply with the provisions of ACI 318, Section 4.3. According to ACI 318 sulfate exposure may be considered negligible if water-soluble sulfate in soil is less than 0.1 percent by dry weight. The water-soluble sulfate was measured to be about 110 mg/kg (ppm) or less (0.0011 percent by dry weight). Therefore, the test results suggest Type II Portland cement is suitable for onsite below-grade concrete structures.

CERCO's evaluation detected chloride ion concentrations in tested samples ranging from none detected (N.D.) to 52 mg/kg. The chloride concentrations are determined to be insufficient to attack steel embedded in a concrete mortar coating. Section 1904.4 of the 2007 California Building Code requires that reinforcement in concrete be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.4. Table 4.4.1 of ACI 318 suggests commencing mitigative measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06 percent by dry weight. The chloride content measured in soil samples was about 52 mg/kg (ppm) or less (.00052 percent), which does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures.

CERCO's evaluation indicates the pH of the soils tested ranges from 5.6 to 7.5, which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Soils with a pH less than 6.0 are considered corrosive, and corrosion prevention measures need to be considered for structures placed in this acidic soil.

For specific long-term corrosion control design recommendations, we recommend that a corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures and steel pipe coated with cement-mortar.

Tests for Buried Ferrous Metals

In addition to the pH tests described above, the resistivity and redox tests were performed on the soil samples discussed above. These tests may be evaluated together to provide an indication of the corrosion potential of the soil environment on buried ferrous metals such as steel or cast-iron pipes. The results of these tests are presented in the table below.

Table 4. Summary of Corrosion Test Results for Buried Ferrous Metal

Test No.	Boring	Depth	Resistivity (ohm-cm)	Redox Potential (mV)	pH
1	B-104	2.5 ft	1,300	420	6.8
2	B-105	15.5 ft	1,700	450	7.5
3	B-112	4 ft	1,800	410	5.6

ND = none detected

CERCO classified the soil samples as "corrosive" based on the resistivity measurements. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron pipeline should be protected against corrosion as well.

The measured soil reduction-oxidation (redox) potentials range from 410 to 450 mV and are considered indicative of aerobic soil conditions.

Appendix B presents the complete results of corrosion tests and recommendations provided by CERCO. For specific long-term corrosion control design recommendations, we recommend that a corrosion engineer evaluate the corrosion potential of the soil environment on buried metal.

5.3.6 Land Subsidence

Land subsidence is a gradual settling or sudden sinking of the Earth's surface owing to subsurface movement of earth materials. The principal causes are aquifer-system compaction, drainage of organic soils, underground mining, hydrocompaction, sinkholes, and thawing permafrost. The conditions needed for these hazards are not known to exist within the project area.

5.3.7 Naturally Occurring Asbestos (NOA)

Inhalation of asbestos fibers may cause cancer. Most commonly, asbestos occurrences are associated with serpentinite and partially serpentinized ultramafic rocks. Asbestos occurs naturally in certain geologic settings in California. Exposure and disturbance of rock and soil that contains asbestos can result in the release of fibers to the air and consequent exposure to the public. Asbestos most commonly occurs in ultramafic rock that has undergone partial or complete alteration to serpentine rock (proper rock name serpentinite) and often contains chrysotile asbestos. In addition, tremolite, another form of asbestos, can be found associated with ultramafic rock, particularly near faults. Sources of asbestos emissions include: unpaved roads or driveways surfaced with ultramafic rock, construction activities in ultramafic rock deposits, or rock quarrying activities where ultramafic rock is present.

The bedrock underlying the site has the potential of containing asbestos fibers; however, bedrock is anticipated to be at a depth of about 300 feet below grade and construction activities should not penetrate through the Quaternary alluvium overburden. The potential for NOA hazards at the site is low.

5.3.8 Radon-222

Radon gas is a naturally occurring radioactive gas that is invisible and odorless. Radon forms from the radioactive decay of small amounts of uranium and thorium naturally present in rocks and soils. Some rock types, such as black shales and certain igneous rocks, can have uranium and thorium in amounts higher than is typical for the earth's crust. Increased amounts of radon will be generated in the subsurface at locations where these rocks are found. Because radon is a gas, it can easily move through soil and cracks in building slabs or basement walls and concentrate in a building's indoor air. Areas with higher amounts of radon in the underlying rocks and soil are likely to have higher percentages of buildings with indoor radon levels in excess of U.S. Environmental Protection Agency guidelines, and incidences of very high indoor radon levels are more likely in these areas. Long term exposures to elevated levels of radon can increase the risk of lung cancer.

In California, the Department of Public Health (CDPH), Radon Program, collects radon test data for buildings throughout the state. The data are maintained in a digital database. The database is used to help CDPH determine areas with excessive indoor radon levels, determine areas that may need testing, and inform the public of the results. This database is organized by zip code, and is **not** to be considered a site specific investigation. The United States Environmental Protection Agency (EPA) recommends that individuals avoid long-term exposures to radon concentrations above 4 picocuries per liter (pCi/L). The Oakland zipcode 94118 has had three tests conducted, none returning with levels exceeding 4 pCi/L.

5.4 CONCLUSIONS - GEOLOGIC HAZARDS

The geologic hazards at the Children's Hospital site appear to include the high expansion potential of the clay soils, a corrosive soil environment, the potential for moderate to strong ground shaking and localized areas of potential liquefaction based on the results of our review and evaluation. Details regarding specific geologic hazards are presented above. Practically all structures within the San Francisco Bay Area will experience similar shaking effects during a moderate to strong earthquake.

5.5 OTHER HAZARDS AND COMPLICATED SITE CONDITIONS NOT ADDRESSED IN THIS REPORT

Under the "Exceptional Geologic Hazards and Complicated Site Conditions" of CGS Note 48 are a number of exceptional items not typically applicable statewide including:

- Phase I & II Environmental Site Assessment;
- Hazardous Materials;
- California Environmental Quality Act;

- Groundwater Quality;
- Onsite Septic Systems; and
- Non Tectonic Faulting and Hydrocollapse of Alluvial Fan Soils;

These exceptional items have not been evaluated in detail as part of this report, and are considered outside of the current scope of Fugro's work.

6.0 DISCUSSIONS AND CONCLUSIONS

We believe that redevelopment of the Children's Hospital is feasible from a geotechnical and geological standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections:

6.1 EXPANSIVE SOILS

As discussed in Section 5.3.6, the moderate to high expansion potential of the clayey surface soils encountered onsite is a primary consideration for foundation design. These materials could be subjected to volume changes during seasonal fluctuations in moisture content. In order to reduce the potential impact of expansive soils on the proposed buildings resulting from swelling and shrinkage of these materials, we recommend that the buildings be supported on deepened footings/mats or drilled piers. We recommend that all interior slabs-on-grade be supported on a layer of imported non-expansive fill. In addition, both slabs-on-grade and grade beams, if used, should be reinforced to account for the expansion pressure of the soils. Note that special design considerations will apply for the design of exterior slabs, as discussed in Section 7.5.2 - Exterior Slabs on Grade.

6.2 UNDOCUMENTED FILL

Our subsurface investigation indicates that about 4 to 8 feet of clay fill with varying amounts of sand and gravel exists at the proposed development locations. This information agrees well with the historic borings drilled at this site that indicate a fill layer. Because this is an undocumented fill, the consistency and density of the material cannot be relied upon for foundation support. Variations in consistency and density could result in differential settlement of structures founded on this material. Therefore, we recommend the fill be reworked to conform to engineered fill requirements as discussed below, or foundations be deepened to extend through the undocumented fill layer.

6.3 GROUNDWATER

Based on our recent borings, as well as previous borings, groundwater was encountered at depths as shallow as 8.5 feet below grade. We anticipate that groundwater will be encountered in the excavation for the hospital, requiring dewatering to establish a stable working base for construction.

The lowest level floor slab at the hospital will be about 16 feet deep and will be below the recommended design groundwater depth of 8 feet. Therefore, the below-grade walls and subfloors should be designed to resist lateral and uplift hydrostatic pressures and appropriately waterproofed to help prevent the potentially damaging migration of water into the structure. Permanent groundwater lowering systems such as subdrains should not be employed because lowering the water table in this area would induce consolidation settlement in the surrounding clay soils and cause settlement of adjacent structures.

6.4 FOUNDATION SUPPORT

Suitable foundation options for each proposed structure are discussed below. Our design recommendations for each foundation system are discussed in Section 7.

6.4.1 New Hospital

The new hospital can be supported on a structural mat foundation, or a drilled pier foundation system with a structural slab designed to resist hydrostatic uplift, depending on the needs of the structural engineer. A spread footing foundation system is not feasible, because of the magnitude of the structural loads and because the basement level will be below groundwater. Liquefaction induced settlement must be included in the design of a mat foundation, as discussed in Section 5.2.2. Isolated drilled piers could be used in combination with the mat foundation to provide additional uplift resistance. If the design lateral and uplift loads are significantly high and a large number of piers are required, the hospital can be supported entirely on drilled piers. We do not believe that driven piles are a feasible foundation option because of the active hospital and residential neighborhood that are sensitive to noise and vibration.

6.4.2 Central Utility Plant (CUP)

We recommend that the new CUP be supported either on a mat foundation or on a drilled pier foundation system, because of the potentially liquefiable soils observed at the proposed CUP location. If a mat foundation is used, the mat should be designed to incorporate 1 inch of differential movement over 50 feet caused by liquefaction induced settlement in addition to long-term settlements. The mat should bear entirely on engineered fill or be deepened to bear on undisturbed native soils. If a drilled pier foundation system is used, the piers must be designed to include a downdrag load caused by settlement of the soil surrounding the piers post liquefaction.

6.4.3 Parking Structure

We recommend the proposed parking structure be supported on conventional spread footings with drilled piers for uplift loads, or if spread footings become too large, completely on a drilled pier foundation system. If spread footings are used, the footings should bear entirely on properly compacted engineered fill or be deepened to bear on undisturbed native soils. The footings should also be deepened because of the highly expansive and undocumented fill material.

6.5 CONSTRUCTION CONSIDERATIONS

6.5.1 Existing Structures and Foundations

Several existing structures, associated foundation systems and buried utilities exist at the proposed development site and will be demolished as part of the construction. The contractor should be aware that buried obstructions are likely to be encountered during construction of the new hospital, parking structure and CUP.

- The new hospital site is currently occupied by several 1-to 2-story high hospital and residential structures. Fugro has not been provided any detailed information regarding the type and locations of the existing foundation system for the structures. However, the structures are likely supported on shallow foundations such as spread or continuous footings. In addition, basement levels may be encountered that will need to be cutoff or removed and backfilled.
- The proposed parking garage site is located in the footprints of the existing Hem/Onc building, an elevated hospital helipad structure, nursing tower and other hospital structures. Foundations for the existing Hem/Onc building presumably consist of drilled piers. Foundations at the helipad and nursing tower likely consist of spread footings. Fugro does not have documentation regarding foundations for the other structures at the proposed parking garage location.
- The proposed CUP site is partially located in the footprint of the existing hospital helipad. The helipad is supported on spread footing foundations.

In order to prevent "hard spots" beneath any future development at the site, all foundations (including basement walls and deep foundations) should be removed to a depth at least 5 feet below the planned final grade. Holes should be drilled in any basement slabs left in place to prevent water from becoming trapped in the buried concrete structure. The contractor should prepare detailed documentation of the location, size and depth of all foundations that are not completely removed.

6.5.2 Excavation Shoring and Dewatering

Excavations will be required for below grade portions of the structures, to install utilities, to rework undocumented fill and to remove locally weak or unsuitable soils, if encountered. Based on the currently proposed location and 16-foot depth of the new hospital basement, excavations will likely extend below the foundation influence of nearby structures. Underpinning of neighboring structures may not be needed; however, shoring will likely needed to limit the size of the excavation because of the depths of excavation required and spatial constraints. The shoring system must be designed to resist loads from foundations of neighboring structures. All excavations that will be deeper than 5 feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards.

Groundwater was observed to vary significantly in our borings and monitoring wells, and should be anticipated during excavations deeper than about 8 feet. To help reduce the impacts of water drawdown on surrounding facilities, a relatively impervious shoring system, such as an interlocking steel sheet pile system, should be installed. The shoring system should penetrate deep enough into the relatively impermeable clay soils below the clayey sand and gravel layer beneath the base of the excavation such that the shoring wall could act as a water cutoff. Given the relatively large size and depth of excavation that will be required, we anticipate that a dewatering system that includes a series of wells may be required in addition to sump pumps. In addition, grading of the excavation subgrade to direct water toward sump pumps or to perimeter or interior ditches may be required to help drain remaining surface water. We anticipate that with a properly installed sheet pile and dewatering system, groundwater within the excavation can be effectively dewatered and controlled. The clayey sand and gravel soils above the clay encountered in our investigation are generally medium dense to dense; therefore, we anticipate that sheet piles could be driven through these materials. Localized areas of very dense clayey sand and gravel were also encountered across the site that could make driving sheet piles more difficult.

The performance of the shoring and dewatering systems are highly dependent on the construction methods and procedures employed. The design of the necessary shoring and dewatering systems, as well as the protection of existing facilities, site improvements and utilities should be the responsibility of the contractor. The project geotechnical and structural engineers should review the proposed shoring system prior to installation.

6.5.3 Earthwork

If earthwork is performed during the dry season, moisture conditioning will be required to raise the in-situ moisture contents to near optimum moisture content (per ASTM D1557). If earthwork is performed during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

6.5.4 Construction Quality Control

Careful quality control should be maintained during construction so that the recommendations presented in this report are achieved. Construction observation is necessary to check that the quality control measures are being properly implemented.

7.0 RECOMMENDATIONS

The recommendations presented in this report should be incorporated in the design and construction of the project to help minimize soil or foundation related problems. Detailed earthwork and foundation recommendations for use in design and construction of the project are presented below. We recommend that our firm review the final design and specifications to check that the earthwork and foundation recommendations presented in this report have been

properly interpreted and implemented. Fugro can assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications.

7.1 SEISMIC DESIGN CRITERIA

The proposed structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practice. This section presents seismic design parameters for use with the 2007 California Building Code (CBC). The CBC is based on the 2006 International Building Code (IBC). The CBC refers to the design code by American Society of Civil Engineers (ASCE 7-05) for the development of site-specific response spectra. Design values based on the CBC are tabulated below. Because of the critical nature of the structures, site-specific response spectra were developed in accordance with guidelines mentioned in Section 21 of ASCE 7-05. The values of S_{DS} and S_{D1} from of our site-specific analyses are included in the table below.

Table 5. 2007 CBC Seismic Design Parameters

Parameter/Coefficient Description	2007 CBC Reference	Parameter/ Coefficient	Value
MCE for 0.2 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(3)	Ss	1.73
MCE for 1.0 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(4)	S1	0.64
Soil Profile Type	Table 1613A.5.2	Site Class	D
Site Coefficient	Table 1613A.5.3(1)	Fa	1.0
Site Coefficient	Table 1613A.5.3(2)	Fv	1.5
Code Based Design Spectral Acceleration Parameters	Equation 16-39	SDS	1.16
	Equation 16-40	SD1	0.64

**Site Located at: Latitude: 37.8380, Longitude: -122.2665

Table 6. Site Specific Seismic Design Parameters

Site-Specific Design Spectral Acceleration Parameters	Section 21, ASCE 7-05	**S_{DS}**	**1.00**
	Section 21, ASCE 7-05	**S_{D1}**	**0.70**

The new hospital, CUP and parking garage should be assigned to Seismic Design Category "D", according to Section 1613.5.6 of the CBC.

7.2 SITE PREPARATION AND GRADING

7.2.1 Site Preparation

The site should be cleared of all obstructions, including concrete, asphalt pavement, buried foundations, slabs, utility lines, trees and associated root systems, and debris. Removed concrete, asphalt concrete, and baserock may be reused as fill, provided the material is broken up to meet the requirements in Section 7.2.4 - Engineered Fill Materials. Holes resulting from the removal of root systems of larger trees could extend to depths of 3 feet, and laterally to the drip line of each tree. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with suitable material compacted to the requirements in Section 7.2.5 - Fill Placement and Compaction. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

As previously discussed, existing structures will be demolished as part of the development. The existing residential structures at the proposed hospital are likely founded on shallow foundations. At the parking garage and CUP location, the existing Hem/Onc building south of the helipad is assumed to be founded on drilled piers, the helipad is founded on spread footings and the nursing tower is believed to be founded on spread footings with one basement level. Several other structures exist at the parking garage location which Fugro does not have details regarding foundation type and depth. These structures are likely also founded on shallow foundations, and may have basement structures. We recommend all foundations and below grade walls be removed to a minimum depth of 5 feet below final planned grade. The contractor should prepare detailed documentation of the location, size and depth of all foundations that are not completely removed.

Prior to backfilling any basement level, holes should be drilled in the floor slab of the basement to prevent water from becoming trapped in the buried concrete structure. If the basement level is not completely removed, the buried concrete will act as a "bathtub" and will collect water. In order to prevent this water buildup, we recommend that holes be created in the floor slab to allow water to pass through. We recommend the holes be installed on a grid pattern with a maximum center-to-center spacing of 10 feet. The holes should be a minimum of 6-inches in diameter.

After clearing, the portions of the site containing surface vegetation or organic laden topsoil should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by the Geotechnical Engineer at the time of construction. Stripped materials should be removed from the site, or stockpiled for later use in landscaping, if approved by the owner.

7.2.2 Subgrade Preparation

Following excavation to the required grades, soil subgrades in areas to receive engineered fill (as defined in Section 7.2.4) or slabs-on-grade should be scarified to a depth of at least 6 inches, moisture conditioned to at least slightly above optimum moisture content, and

compacted to at least 95 percent of the soil's maximum dry density. The top 6 inches of subgrade in areas to receive pavements should be moisture conditioned and compacted to at least 95 percent of the soil's maximum dry density. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. If the subgrade is allowed to become dry, it should be moisture conditioned to eliminate shrinkage cracks.

In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry, or that scarification and aeration be performed in any soils that are too wet. There was no indication of weak or unstable soils in our borings; however, after the removal of existing buildings and pavements, the exposed subgrade materials may be above their optimum moisture content, and may be locally unstable. If areas of unstable soils are encountered at the time of construction, the geotechnical engineer should review conditions in the field and provide recommendations for stabilization procedures.

7.2.3 Reworking Undocumented Fill

As previously discussed, about 4 to 8 feet of undocumented fill was observed in our borings at the site. If footings or mat foundations are designed to bear within this layer, we recommend that this material be removed to within 12 inches of the native material and replaced with engineered fill in accordance with the recommendations in this report. Prior to placing engineered fill, the bottom of the excavation should be scarified to a depth of 12 inches, moisture conditioned to slightly above optimum moisture content, and compacted to al least 95 percent of the soil's maximum dry density as determined by ASTM D1557 (modified proctor test.) However, the exact depth of the excavation should be determined in the field by a geotechnical engineer.

Over excavation and backfilling with engineered fill should be performed to a lateral extent corresponding to a point where an imaginary plane extending outward and downward at a 1:1 inclination (horizontal to vertical) from the outside bottom edge of the perimeter foundation intersects the bottom of the undocumented fill. For example, based on the 6 feet of fill observed in our borings and an estimated footing depth of 1 foot, the fill should be removed to a lateral extent of 5 feet beyond the perimeter of the footing. The exact dimensions of the excavation should be determined in the field by a Geotechnical Engineer. The material removed may be reused as engineered fill, provided the material is placed in accordance with the requirements of Section 7.2.5 – Fill Placement and Compaction.

Shoring or underpinning may be required in portions of the excavations where existing site features prevent the use of construction slopes. Additionally, if site constraints do not provide space for extending the footprint of the excavation, the over excavated zone can be replaced with controlled-low strength material (CLSM). CLSM is significantly stiffer than the surrounding soil and will essentially lower the bearing layer of the foundations. With this alternative, the lateral extent of over excavation can be reduced to correspond to the footprint of the structure. CLSM is typically cast in-situ; therefore, no compaction is necessary. Periodic density testing is completed during placement to ensure the project specifications are met.

7.2.4 Engineered Fill Materials

All fill placed at the site should consist of engineered fill meeting the requirements presented in this report, except for landscaping materials which are placed on level ground. On-site soil below the stripped layer and having an organic content of less than 3 percent by volume can be used as fill except where "non-expansive" import is required beneath slabs-on-grade. All engineered fill placed at the site, including on-site soils, should not contain rocks or lumps larger than 4 inches in greatest dimension and contain no more than 15 percent larger than 2.5 inches.

"Non-expansive" fill should be predominantly granular, have an organic content of less than 3 percent by volume, should have a liquid limit less than 40 percent, have a plasticity index not exceeding 15, and should contain no environmental contaminants or debris. All imported fill should consist of "non-expansive" fill.

7.2.5 Fill Placement and Compaction

All engineered fill should be compacted to at least 95 percent of the soil's maximum dry density as determined by ASTM Designation D1557 (latest edition). The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent of the soil's maximum dry density. Fill material should be spread and compacted in lifts not exceeding 8 inches in pre-compacted thickness. The moisture content of the natural on-site expansive clayey soils reused as fill should be 2 to 3 percent above the optimum moisture content for the soil at the time of compaction. In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet.

7.2.6 Trench Backfill

Pipeline trenches should be backfilled with materials satisfying the criteria described above for fill, placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts may be used provided the method of compaction is approved by the project geotechnical engineer and the required minimum degree of compaction is achieved. Onsite soil used for trench backfill should be compacted to at least 90 percent of the soil's maximum dry density by mechanical means only (jetting should not be permitted). Sand can be used for trench backfill if the sand is compacted to at least 95 percent of the soil's maximum dry density and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill below slab and pavements should be compacted to at least 95 percent of the soil's maximum dry density.

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts can be used, provided the method of compaction is approved by the Geotechnical Engineer, and the required minimum degree of compaction is achieved.

Where utility trenches backfilled with sand enter building areas, they should be backfilled by an impermeable soil plug that is at least 2-feet thick. The plug should be located at the perimeter of the foundation. The plug should be keyed at least 12 inches into the sides and bottom of the trench and should extend to within 12 inches of the finished grade. This should help to minimize moisture change in the moderately to highly expansive clays beneath the slabs. Where sand backfilled utility trenches cross planter areas and pass below pavements or concrete sidewalks, they should be plugged as described above to minimize soil volume change below asphalt and concrete areas.

7.2.7 Surface Drainage

Positive surface gradients should be provided adjacent to the buildings to direct surface water away from foundations and slabs toward suitable discharge facilities. Water from roof downspouts should be carried away from the buildings in solid pipes and discharged into suitable drainage facilities. Surface water should not be allowed to pond adjacent to the structures or on pavements.

7.3 TEMPORARY SHORING AND CONSTRUCTION SLOPES

7.3.1 General

Construction may require either temporary slopes or shoring for excavations as needed. If site constraints are not present, temporary construction slopes may be used. However, where site constraints are present, or at the contractor's option, temporary shoring should be used. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary construction slopes and shoring system. In selecting the temporary slope, the contractor should evaluate the potential of shallow soil movement, which may potentially jeopardize the existing nearby facilities.

7.3.2 Temporary Construction Slopes

If temporary slopes are used, the contractor should be aware that in no case should slope height, inclination, and excavation depths exceed those specified in local, state, or federal safety regulations. Specifically, one needs to be aware of the current OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P. In areas next to adjacent existing foundations, we recommend that open excavations not be performed within 1.5:1 (horizontal: vertical) slope extended downward from the bottom edge of adjacent foundations.

Fugro recommends that the contractor, or his specialty subcontractor, design temporary construction slopes to conform to the OSHA "Guidelines for Excavations and Temporary Sloping." The contractor, or responsible subcontractor, should determine temporary slope inclinations based on the subsurface conditions exposed at the time of construction.

For planning purposes, the onsite soils should be categorized as OSHA Type C with temporary slope inclinations of no steeper than 1.5:1 (horizontal to vertical) for excavations less than 20 feet deep.

If temporary slopes are left open for extended periods of time, exposure to weathering and rain could have detrimental effects such as sloughing and erosion. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes and that the temporary slopes be protected from excessive drying or saturation during construction.

7.3.3 Temporary Shoring

If open-excavation is not possible because of site constraint and proximity of existing or new buildings or streets, we recommend that a temporary shoring system be used during any needed excavations. Such shoring should be sufficiently rigid to minimize damage to the adjacent facilities (streets, utilities, neighboring buildings, etc.) caused by deflection of the sides of the excavation. The temporary shoring system must be designed to include loads from foundations of neighboring structures.

We recommend a relatively impervious shoring system, such as an interlocking steel sheet pile system, be installed as discussed in Section 6.3.2. The shoring system should penetrate deep enough into the relatively impermeable clay soils beneath the base of the excavation such that the shoring wall acts as a water cutoff and to provide adequate factor of safety against base heave.

Fugro recommends that the contractor, or his specialty subcontractor, design and install the temporary shoring. We recommend that the geotechnical and structural engineers review the final temporary shoring plan to ensure compliance with the anticipated soil conditions encountered at the site. In addition, we recommend that a geotechnical engineer's representative observe the installation of the temporary shoring systems. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary shoring system.

7.3.4 Construction Dewatering

The excavation for the basement will extend below the groundwater level and will require dewatering to provide dry working conditions and help prevent instability of the bottom of the excavation during construction. We recommend that the groundwater be drawn down at least 2 feet below the bottom of the excavation. We acknowledge that in areas underlain by more clayey soil, it may be difficult to draw the groundwater down to these depths. In these cases, other measures such as grading and directing the water to sump pumps and/or drainage ditches within the excavation may be more appropriate.

The dewatering system should be designed and installed by a specialty contractor who is familiar with similar subsurface conditions. The Geotechnical Engineer should review and check if the proposed dewatering system and approach may negatively impact the adjacent structures. The main considerations that must be factored into the dewatering design are settlement and decreased stability of foundation for the surrounding structures that could be induced by lowering the groundwater table through dewatering.

7.4 FOUNDATIONS

7.4.1 Spread Footings

Spread footings may be used to support the parking garage. Conventional continuous and isolated spread footings should bear on properly compacted engineered fills or on undisturbed native soils. No footings should bear in the undocumented fill material, where the consistency and density is likely highly variable. The undocumented fill should be reworked and placed as engineered fill according to the requirements in this report, or the footings should be deepened to extend through the undocumented fill layer.

In addition, footings should be founded at least 36 inches below lowest adjacent finished grade, because of the highly expansive nature of the onsite surface soils. Footings located adjacent to other footings (i.e. existing structures) or buried utilities should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trench.

Footing dimensions should be based on structural requirements. However, we recommend continuous footings be a minimum of 18-inches wide and spread footings be a minimum of 24-inches wide for ease in construction.

The following allowable bearing values may be used for spread footings. These bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Table 7. Allowable Bearing Pressures for Spread Footings

Load Condition	Factor of Safety	Allowable Bearing Pressure (psf) Parking Garage
Dead Load	3	3,000
Dead plus Live Loads	2	4,500
Total Loads (including wind or seismic)	1.5	6,000

Footings designed with the above bearing pressures should have long-term settlement limited to about 1 inch, and differential settlement limited to about ½ inch over a typical column spacing of 30 feet.

7.4.2 Mat Foundations

If a mat foundation is used at the hospital site, the mat should bear on undisturbed native soils at the basement level. If a mat is used at the CUP location, the mat should bear on either properly compacted engineered fill or on undisturbed native soils (depending on the design depth of the finished floor). No mat foundation should bear on the undocumented fill

material at the site. The undocumented fill should be reworked and placed as engineered fill to support the mat, or the mat should be deepened to extend through the fill layer.

A number of considerations are involved in the design of mat-type foundations. In addition, there are a variety of mat foundation types ranging from a uniformly thick slab, to slabs with various locally-thickened features such as waffles, thickened column plates or pedestals. The technical problems associated with mat foundation design are complex and a number of design methods have evolved. The following geotechnical criteria are provided for use in mat foundation design.

Modulus of Subgrade Reaction

The modulus of subgrade reaction (k_v) reflects the conceptual relationship between soil deflection and soil pressure. We recommend a modulus of subgrade reaction of 150 pounds per cubic inch (pci), based on the stiff clayey nature of subsurface soils encountered at the site during current and previous studies. This modulus value corresponds to a 1 foot by 1 foot bearing area and needs to be adjusted to account for the dimensions of the mat foundation. We recommend the modulus of subgrade value be divided by the width of the mat, B, in inches.

Allowable Soil Bearing

For a mat foundation located at the bottom of a basement excavation, the dead plus live loads of the structure should not exceed the weight of the soil removed during the excavation. The weight of the removed soil can be calculated using an average value of 125 pcf above groundwater (assumed to be 8 feet below existing grade) and 63 pcf below groundwater. The calculated allowable bearing is the bearing pressure averaged over the entire mat area to minimize long-term settlement.

7.4.3 Drilled Pier Foundations

Drilled pier foundations may be used in combination with spread footings or a mat to resist seismic and hydrostatic uplift loads, or may be used as an alternative foundation system. The drilled piers should be cast-in-place, straight piers that develop their load carrying capacity through skin friction in the stiff clay soils and interlayered clayey sands and sandy clays. Piers located in a group should have a minimum center-to-center spacing of three times the shaft diameter. The minimum pier diameter should be 18 inches. Pier reinforcing should be based on structural requirements. If piers are used at the CUP location, they must include downdrag loads, as discussed below.

The upper 8 feet of the pier below existing ground surface should be neglected because of the undocumented consistency of the fill material and to account for seasonal moisture variations in the highly expansive material that could result in shrinkage of the material away from the piers. The design depth of the piers should be determined using an ultimate skin friction of 1,400 pounds per square foot (psf) in the upper stiff clay and the interlayered clayey sand and sandy clay to a depth of 50 feet. An ultimate skin friction value of 2,000 psf may be used in the very stiff to hard clay below a depth of 50 feet. We recommend a minimum factor of safety of two be used for design of friction piers.

Table 8. Drilled Pier Design Skin Friction

Depth Below Ground Surface (feet)	Soil Type	Ultimate Skin Friction (psf)	Allowable Skin Friction (psf)
0 to 8	Undocumented Fill	0	0
8 to 50	Stiff Clay and Interlayered Clayey Sand and Sandy Clay	1,400	700
> 50	Very Stiff to Hard Clay	2,000	1,000

Our recommended allowable skin friction values may be increased by one-third for all load combinations that include wind or seismic loads. Uplift loads may be resisted using 80 percent of the allowable skin friction values.

Total and differential settlements of drilled pier supported structures should be minimal (less than ½ inch) and within tolerable limits for the proposed structures.

Drilled Pier Downdrag (CUP Location only)

Downdrag occurs where soils settle around a relatively rigid deep foundation system and pull the pier downward. Significant downdrag loads should not occur on piers at the hospital location because of the relatively small amount of anticipated liquefaction settlement and soil arching effects.

If drilled piers are used at the CUP structure, the piers must be designed to include the downdrag load that could be imposed on the piers by settlement of the surrounding soil post liquefaction during a major seismic event. As the soils settle with respect to the piers, the side friction force in the upper zone acts downward instead of upward and becomes a load instead of a resistance. Our boring at the CUP location indicates the top of the liquefiable layer is at about Elevation 89. For design purposes, we recommend applying an ultimate negative skin friction of 700 psf to the portion of the pier above Elevation 89. The liquefiable sand layer will not contribute significantly to the downdrag load. Negative skin friction should be applied to the pier based on final design grade, not existing site grade. We recommend negative skin friction be subtracted from the ultimate capacity of the pier, not the allowable capacity, as shown below:

$$\frac{Q_{ULTIMATE} - NegativeSkinFriction}{Factor\ of\ Safety = 2} = Q_{ALLOWABLE\ (dead+live)}$$

Drilled Pier Construction

Pier excavations extending below a depth of about 8 feet will be below the groundwater table and will require special considerations during construction. Excavations may need to be cased to protect the holes from caving. All pier excavations adjacent to existing structures should be cased to prevent caving of the hole caused by nearby footing loads. In addition, we recommend that concrete be placed in the bottom of the drilled holes by the tremie method. Concrete should be designed with a high slump, equal to or greater than 6 inches, to facilitate

construction and reduce the potential for free water to be trapped in the pier excavation. Concrete should be placed in all piers the same day that the excavations are completed. We recommend that the excavation of all piers be performed under the direct observation of the geotechnical engineer to confirm that the pier foundation is constructed in general accordance with the requirements presented above.

Pier Caps and Grade Beams

Pier caps and grade beams should be used to tie together the foundation piers. Pier caps and grade beams within 3 feet of the ground surface should be designed to resist an uplift pressure of 2,500 pounds per square foot (from highly expansive soils), with uplift resistance provided by the drilled piers. Pier caps and grade beams beneath the proposed 16-foot hospital basement do not need to be designed for expansion pressures. Grade beam reinforcement should be designed to meet structural requirements.

7.5 LATERAL LOADS

7.5.1 Long-Term Lateral Loads

Long-term resistance to lateral loads may be provided by friction along the base of spread footings and mat foundations. A friction coefficient of 0.30 times the dead load may be used to evaluate the allowable frictional resistance along the bottom of foundations in clay. Friction resistance should not be used on the bottom of grade beams or pier caps.

In addition to friction, a long-term passive pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for lateral load resistance against the face of footings, grade beams and pier caps perpendicular to the direction of loading where the foundation is poured neat against engineered fill or undisturbed native material. However, a reduced passive pressure of 150 pcf should be used on any portion of the footings, grade beams or pier caps within the undocumented fill layer (upper 8 feet below existing grade).

The equivalent fluid pressure of 300 pcf can also be assumed to act against twice the project diameter of the drilled pier shaft on the upper 10 feet of the piers. *However, a reduced passive pressure of 150 pcf should be used on any portion of the drilled pier within the undocumented fill layer (upper 8 feet below existing grade).* Additionally, any portion of pier caps and grade beams within the upper 3 feet below grade should be neglected in the passive resistance design because of the expansive nature of the soils. If piers are spaced closer than six diameters on center, the recommended passive pressure should be reduced to account for group effects. We can provide group reduction factors during design once the estimated pier spacing and load requirements are better known.

Note that lateral deflections corresponding to a Δ/H of 0.04 (where Δ equals horizontal deflection and H equals the height of the footing, pier cap or grade beam) are required to mobilize full passive resistance. Compatible deflection-resistance relationships must be considered when evaluating the relative contributions of passive resistance from foundation elements. The table below presents the recommended load-deflection curves to mobilize the passive resistances for different lateral deflections.

Table 9. Passive Soil Resistance (P) versus Displacement (Δ) Relationship

Deflection Ratio (Δ/H)	Passive Soil Resistance Ratio (P/P_{max})
0.00	0.0
0.01	0.5
0.04	1.0
1.00	1.0

7.5.2 Temporary Seismic Lateral Loads

For temporary dynamic loads, a maximum passive resistance of 1,500 psf can be used instead of the passive pressures given above. Passive resistance in the upper 1 foot should be ignored unless the soil is confined by a pavement or slab.

For lateral resistance to seismic loads, friction along the sides of the footing, pier caps and grade beams parallel to the direction of the imposed lateral force may be combined with passive resistance. An allowable uniform frictional resistance of 300 psf, corresponding to a lateral displacement of about ½ inch, may be used to resist lateral seismic loads. *However, a reduced frictional resistance of 150 psf should be used on any portion of the footings, grade beams or pier caps within the undocumented fill layer (upper 8 feet).* Frictional resistance should only be considered applicable to the sides of the footing, pier cap and grade beam parallel to the direction of the imposed lateral seismic force. Friction resistance along the bottom of the pier caps and grade beams should be ignored in evaluating the lateral seismic resistance. Frictional resistance along the bottom of spread footings or mat foundations can be calculated using a friction coefficient of 0.30 times the dead load.

7.6 WATERPROOFING AND DESIGNING FOR HYDROSTATIC PRESSURES

The lowest floor level at the hospital (and any slabs below water at the CUP or parking structure, if used) should be waterproofed and designed to resist hydrostatic uplift pressures. Appropriate slab reinforcement should be provided in accordance with the anticipated loading of the slab, including hydrostatic uplift pressures. We recommend that hydrostatic pressure be calculated assuming a design groundwater depth of 8 feet below existing grade. Hydrostatic uplift pressure can be resisted by the dead load of the structure or by drilled piers. Drilled piers may be used in combination with a mat slab, if additional uplift resistance is needed.

7.7 CONCRETE SLABS-ON-GRADE

7.7.1 Interior Slabs-on-Grade

We recommend that interior slabs-on-grade be supported on a minimum of 18 inches of select, predominantly granular "non-expansive" fill meeting the requirements discussed above in Section 6.2.3. Where slabs-on-grade are located in excavations more than 5 feet below surrounding grades, "non-expansive" materials are not required beneath the slabs. Slab

reinforcing should be provided in accordance with the anticipated use and loading of the slab. However, the slab should be reinforced with a minimum of No. 4 bars on 18-inch centers, both ways, because of the expansive soils at the site. Additionally, interior slabs should be separated from the pier foundations and grade beams, where used, so that expansion pressure on the foundations does not cause additional damage to the slab. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

If migration of moisture through the floor slab is undesirable, a moisture retarder system should be provided between the slab and subgrade. We recommend that the system consist of 4 inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than 3 percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder that is placed between the subgrade soil and the mat. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class A, B, or C Underslab Vapor Retarders.

If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., .012 perms) per ASTM E 96 (e.g., 15-mil thick "Stego Wrap Class A") may be used in place of the retarder. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam. The vapor retarder or barrier should be placed directly under the slab foundation, or at the structural engineer's option, the retarder may be covered with 2 inches of sand. If used, sand should be lightly moistened just prior to placing the concrete.

7.7.2 Exterior Slabs-on-Grade

As previously discussed, the onsite moderate to highly expansive surface soils could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs, sidewalks, and pavements should be anticipated. This movement could result in damage to the slabs, sidewalks, and pavements that might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

Exterior slabs such as sidewalks could be reinforced with steel reinforcing bars in lieu of wire mesh to minimize the impact of expansion pressures.

Walkways and pavement curbs and gutters should be supported directly on properly prepared native soils. Eliminating rock base beneath slabs will reduce the potential for migration of landscape irrigation water into pavement and walkway subgrade. Curbs should extend to the bottom of the pavement and baserock layer. One to two days prior to placing concrete, subgrade soils should be soaked to increase their moisture content to 3 to 5 percent above laboratory optimum moisture (ASTM D1557). The water content of subgrade soils should be verified by field-testing by the Geotechnical Engineer prior to placing concrete.

To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and/or a "drip" irrigation watering system be used. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the tree from slabs or pavements to reduce the effects of tree roots on these improvements.

7.8 RETAINING/BELOW GRADE WALLS

Below grade walls must be designed to resist lateral earth pressures and any additional lateral loads caused by surcharging or seismic pressure. If the walls are designed to resist lateral loads through passive pressure, we recommend they be designed for the equivalent fluid pressures discussed in Section 6.3.4 – *Lateral Loads*. If below grade walls are not designed to resist passive lateral loads, they should be designed as discussed below.

The following recommendations were developed based on the clayey soils encountered at the new hospital location. Retaining wall backfill less than 5 feet deep should be compacted using light compaction equipment to at least 90 percent of the soil's maximum dry density. Backfill greater than 5 feet deep should be entirely compacted to at least 95 percent of the soil's maximum dry density. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

7.8.1 Unrestrained Walls

Unrestrained walls (walls that are free to move/rotate) should be designed for active earth pressure. We recommend unrestrained walls be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot (pcf). This equivalent fluid pressures is based on:

- Walls backfilled with onsite, reworked lean clay fill compacted to the requirements in this report;
- The walls are fully back drained to prevent the development of hydrostatic pressure;
- The walls are capable of moving/rotating sufficiently for active pressure conditions to develop; and
- Level backfill. Walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.

In addition to soil pressure, a uniform seismic pressure of 30*H (psf) should be applied to the entire wall height, where H is the height of backfill above the top of the wall footing, in feet.

Unrestrained walls should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load. The design surcharge should include the anticipated surcharge caused by vehicular traffic and construction equipment. Surcharge loads from adjacent structures need to be considered if the proposed walls extend below the zone of

influence of adjacent foundations. The zone of influence of adjacent foundations can be defined as the area below an imaginary 1.5:1 (horizontal to vertical) line extending downward from the bottom of footings near the wall.

Wall Drainage

The recommended lateral pressures for unrestrained walls herein assume walls are fully back drained to prevent the buildup of hydrostatic pressures. Adequate drainage could be provided by means of either weep holes with permeable material installed behind the walls or by means of a system of subdrains.

For the subdrain system, the top of the perforated pipe should be below the bottom of the adjacent slab or grade at the toe of the wall. Drains should consist of a drain rock layer at least 12 inches thick that extends to within 2 feet of the ground surface. Four-inch diameter perforated plastic pipe should be installed (with perforations down) along the base of the walls on a two-inch-thick bed of drain rock. The pipe should be sloped to drain by gravity to a suitable drainage facility. Drain rock should conform to Caltrans specifications for Class 2 permeable material. A more open-graded material, such as ¾-inch crushed rock, could be used provided the rock is wrapped in a geotextile filter fabric (Mirafi 140 N or equivalent) to reduce the migration of fine-grained soils into the drain rock. Paving or a 2-foot thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration. Drainpipes should outlet to an appropriate drainage facility.

Alternatively, wall back-drainage can be provided by prefabricated drainage material (such as Miradrain 6000 or an approved alternative). The drainage material can be installed on the back (soil) face of the basement wall and should terminate at a 4-inch diameter perforated plastic pipe surrounded by at least 6 inches of drain rock as defined above.

Recommended static design lateral pressures for unrestrained walls are presented on Plate 11a. Recommended seismic design lateral pressures for unrestrained walls are presented on Plate 11b.

7.8.2 Restrained Walls (Basement Walls)

Restrained walls (walls that are prevented from moving/rotating) should be designed for at-rest earth pressure as well as hydrostatic pressure. We recommend restrained walls be designed to resist an equivalent fluid pressure of 60 pounds per cubic foot (pcf). These equivalent fluid pressures are based on:

- Walls backfilled with onsite, reworked lean clay fill compacted to the requirements in this report;
- The walls are prevented from moving/rotating such that at-rest pressure conditions develop; and

- Level backfill. Walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.

For seismic loading conditions only, we recommend that soil pressures be reduced to the active (unrestrained) condition plus the seismic increment. For the seismic case, restrained walls should be designed to resist an equivalent fluid pressure of 40 pcf for soil loading, plus a uniform seismic pressure of 38*H psf, where H is the height of backfill above the top of the wall footing, in feet.

Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.50 times the anticipated surcharge load. The design surcharge should include the anticipated surcharge caused by vehicular traffic and construction equipment. Surcharge loads from adjacent structures need to be considered if the proposed walls extend below the zone of influence of adjacent foundations. The zone of influence of adjacent foundations can be defined as the area below an imaginary 1.5:1 (horizontal to vertical) line extending downward from the bottom of footings near the wall.

Hydrostatic Pressure

Below-grade walls should be waterproofed and designed to resist hydrostatic pressures. The pressures recommended above can be used for the portion of the wall above the design groundwater level (8 feet below grade); below the designed groundwater level, restrained walls should be designed for an equivalent fluid pressure of 95 pcf.

We recommend all basement walls be waterproofed. If the permanent wall will be constructed immediately against a temporary wall, the waterproofing must be applied on the face of the temporary wall.

Recommended static design lateral pressures for restrained walls are presented on Plate 12a. Recommended seismic design lateral pressures for restrained walls are presented on Plate 12b.

7.9 PAVEMENTS

We have developed the following pavement design recommendations based on an R-value of 5, based on the critical expansion potential of the clayey soils at the site and R-value tests performed on bulk samples collected during our investigation. We developed the following alternative preliminary pavement sections based on Chapter 630 of the State of California Department of Transportation Highway Design Manual and assumed traffic indices. Pavement designs for pavement lives of 1 to 5 years, 6 to 10 years, and 11 to 20 years are presented below.

Table 10. Recommended Pavement Design Alternatives

Location	Anticipated Pavement Life (years)	Pavement Components		Total Thickness (inches)
		Asphalt Concrete (inches)	*Caltrans Class 2 Aggregate Base (inches)*	
Automobile Parking (T.I. = 4.5)	20	2.5	9.0	11.5
Automobile Access (T.I. = 5.5)	20	3.0	12.0	15.0
Heavy Truck Access (T.I. = 6.5)	20	3.5	15.0	18.5

The traffic indices used in our design were established assuming a typical mix of automobile and "delivery or garbage" truck type of use in the proposed development once construction has been completed. However, if the pavements are planned to be placed prior to, or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. Therefore, if the pavement sections will be used for construction access, our firm should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use. If requested, we could provide recommendations for a phased placement of the asphalt concrete to minimize the potential for mechanical scars caused by construction traffic on the finished grade.

The traffic indices should provide the indicated pavement lives with only a normal amount of pavement maintenance. Selection of the design traffic parameters, however, was based on engineering judgment, and not on an equivalent wheel load analysis developed from a traffic study or furnished to us.

In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils and trench backfill should be protected against saturation. Planned concrete sidewalks, driveways, and curb and gutters should be supported directly on the properly compacted native soils. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas and the pavement section. In addition, a compacted impermeable soil plug should be constructed within any lateral or other trench backfill that passes beneath the curb and gutter and under the adjoining pavement. In addition, water should never be allowed to pond behind the curb and gutter during or after the completion of construction.

The Aggregate Base for use in flexible pavements should conform to Caltrans Standard Specification Section 26-1.02A for Class 2 Aggregate Base. The Aggregate Base used in the pavement sections should be compacted to 95 percent of the soil's maximum dry density (ASTM D1557) and should be firm and unyielding.

7.10 ADDITIONAL GEOTECHNICAL SERVICES

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations. The analyses, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from the subsurface explorations conducted for the project, and upon the conditions existing when services were conducted. Variations of subsurface conditions from those analyzed or characterized in the report are possible, as may become evident during construction. In that event, it may be advisable to revisit certain analyses or assumptions.

We recommend that Fugro be retained to provide geotechnical services during site grading and foundation installation to observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered. During construction, our field engineer should observe and/or test the following:

- Soil conditions exposed by site grading and foundation excavations, to check that they are consistent with those encountered during the field exploration,
- Installation of temporary shoring,
- Pier foundation excavations,
- Pavement subgrade preparation, and
- Fill placement and compaction, including backfill of utilities and compaction of aggregate base.

8.0 LIMITATIONS

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The analyses and recommendations contained in this report are based on the data obtained from the subsurface explorations conducted for this study and relevant previous explorations at the site. These explorations indicate subsurface conditions only at specific locations and times, and only to the depths explored. Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our analyses of the observed conditions. If conditions other than those described in this report are encountered, we should be notified so that we can provide additional recommendations, if warranted.

This report has been prepared for the exclusive use of Children's Hospital and Research Center Oakland and their consultants for specific application to the Children's Hospital development as described herein. In the event that there are any changes in the ownership, nature, design, or location of the proposed project, or if any future additions are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) the project changes are reviewed by Fugro, and 2) conclusions and recommendations presented in this report are modified or verified in writing. Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

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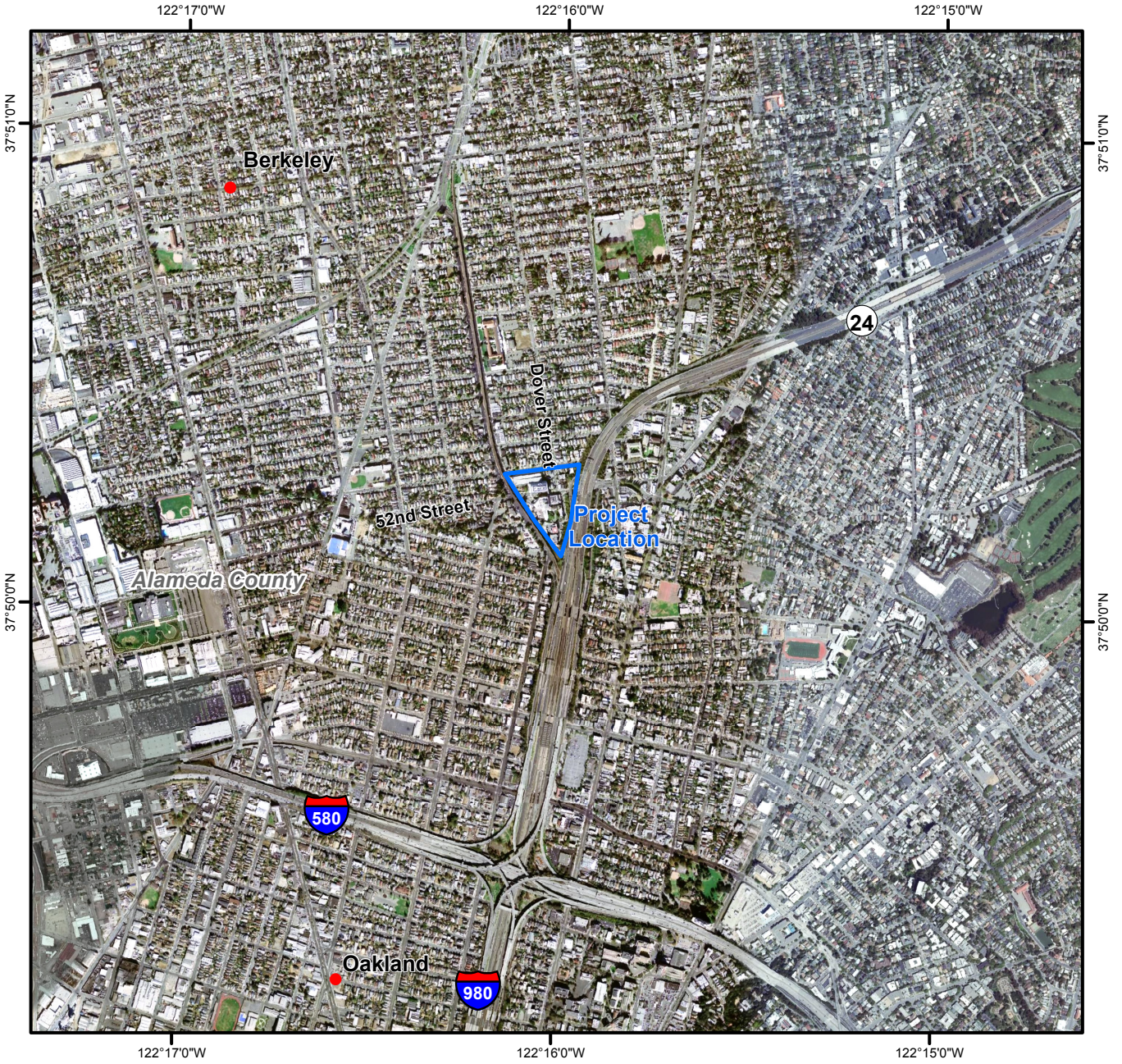
PLATES



Source: Aerial photograph obtained from Google Earth

Site Location:
Latitude: 37.8380°
Longitude: -122.2665°

N

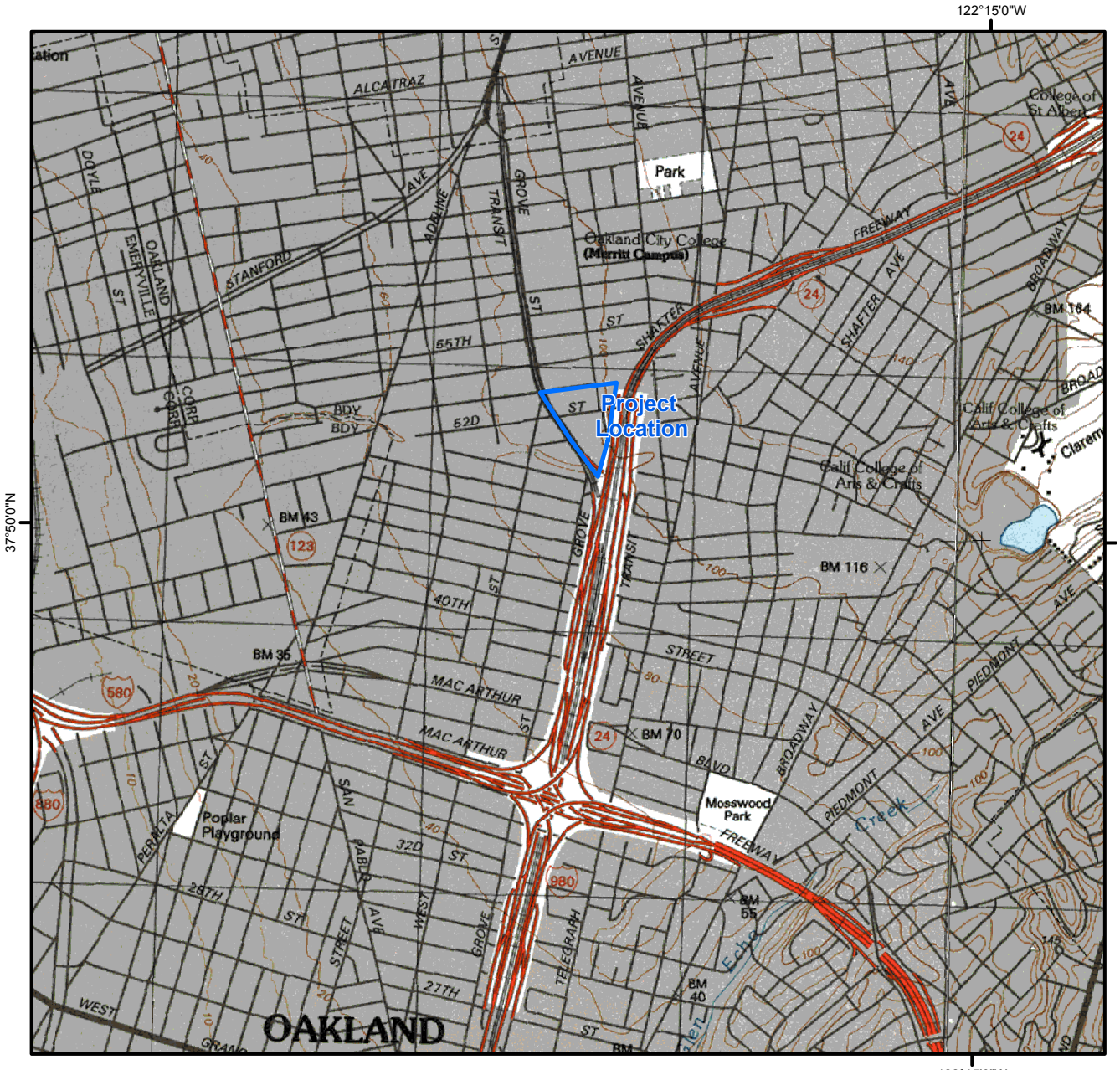
1:24,000

0 1,000 2,000 4,000 Feet

VICINITY MAP
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 1

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate1-Vicinity_Map.mxd, 04/23/08, ksheil



Source: USGS, Oakland West Quadrangle, California, 7.5-Minute Series (Topographic), 1993, Scale 1:24,000

Site Location:
Latitude: 37.8380°
Longitude: -122.2665°

N

1:24,000

0 1,000 2,000 4,000 Feet

TOPOGRAPHIC SITE MAP
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 2

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate2-Topography.mxd, 04/21/08, ksheil

Legend

- B-101 — 2008 Fugro Borings (approximate)
- PB-1 (87) — Historic Borings (approximate), with Number and Report Date
- Hospital Facilities
- Contours, elevations in feet
- A — A' Cross Section
- Approximate Location of Proposed New Structures
- Approximate Location of Proposed New Driveways

Notes:

(74) Harza Kaldveer "Patient Services Pavilion – Phase I"
Project No. K938-8-1421

(78) Kaldveer "Addition to Children's Hospital"
Project No. K480-J

(86a) Kaldveer "Proposed Medical Office Building and Parking Structure"
Project No. K928-1

(86b) Kaldveer "Family Center Investigation"
Project No. K876-3

(87) Kaldveer "Nursing Tower Investigation"
Project No. K339-7

(90) Kaldveer "Proposed Medical Office Building and Parking Structure"
Project No. K339-9

(92a) Kaldveer "Ambulatory Services Center"
Project No. K938-5-1124

(92b) Kaldveer Harza "Proposed Cath Laboratory"
Project No. K938-7-1313

(92c) Kaldveer "Chori Laboratory Addition"
Project No. K938-6-1125

(94) Harza "Geologic Hazards and Seismic Response Study"
Project No. K938-8-1421

(97) Harza "Helistop Facility"
Project No. L998-G

(02) Fugro "Western Expansion"
Project No. 21110-GI

HORIZONTAL DATUM
GRID: California State Plane, NAD 83
ZONE: 3, CA-0403
UNITS: U.S. Survey, Feet

VERTICAL DATUM
North American Vertical Datum, 1983 (NAVD 88)
UNITS: Feet

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GEODETIC INFORMATION

SPHEROID:	
SEMI-MAJOR AXIS:	GRS 1980
SEMI-MINOR AXIS:	20850147.590
INVERSE FLATTENING:	298.25722101
ECCENTRICITY*:	0.00869380
PROJECTION:	LAMBERT CONFORMAL CONIC
LATITUDE OF ORIGIN:	36.5°
FALSE EASTING:	6,561,666
FALSE NORTHING:	1,640,416

N

FUGRO WEST, INC.
1000 Broadway, Suite 440
Oakland, Ca 94607
Tel: (510) 268-0461
Fax: (510) 268-0137
www.fugrowest.com

SITE MAP

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

SCALE=1:600

0 25 50 100 Feet

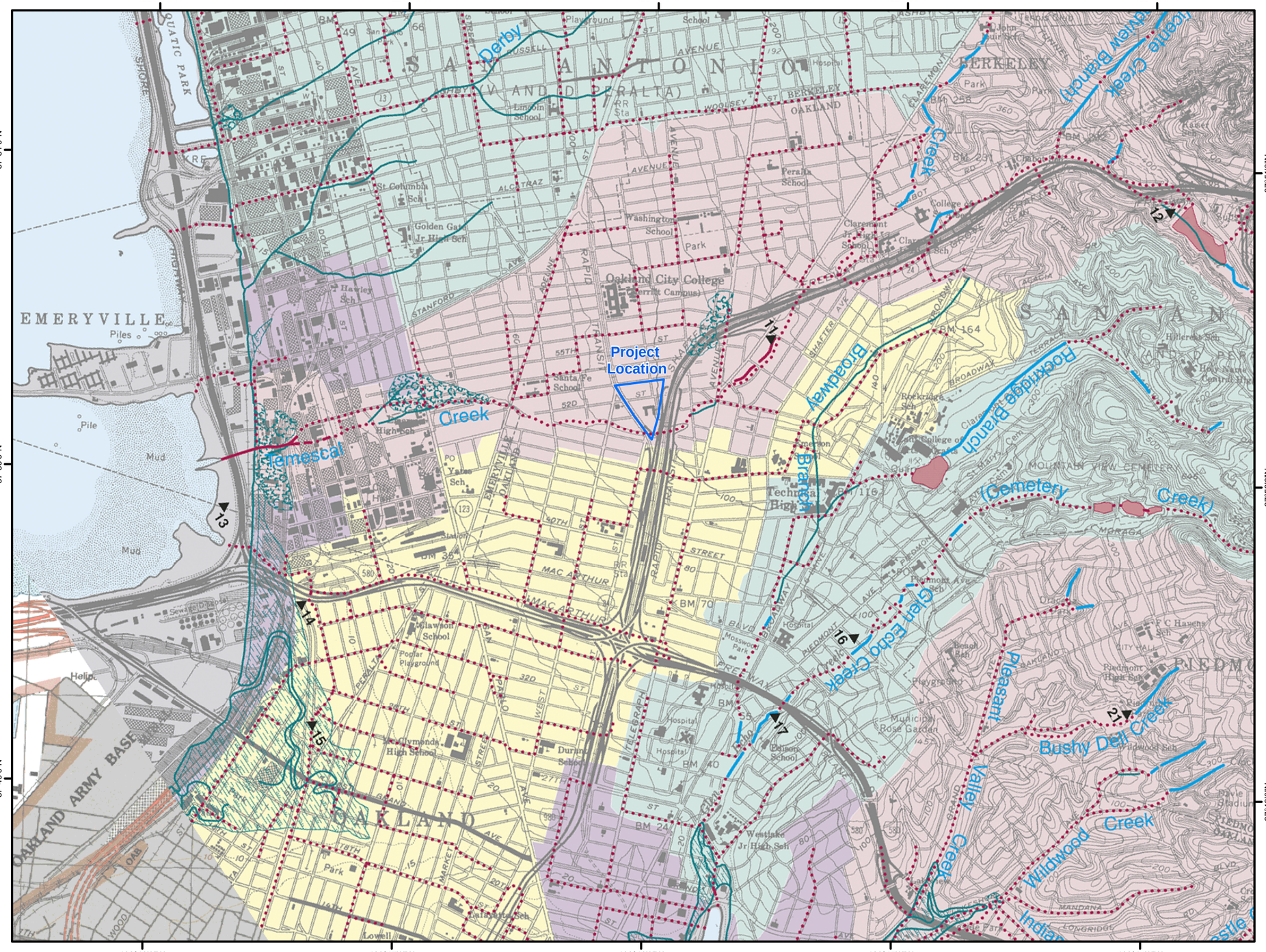
NO:	DATE:	DESCRIPTION:	DRAWN:	CHKD:	APPR:
1	March 2008	Site Map	KES	NH	MP

JOB NUMBER: 1595.002

PLATE: 3

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Output\March_2008\mxd\Plate3-Site_Plan.mxd, 04/2/2008, kaheit

Children's Hospital and Research Center Oakland
Project No. 1595.002



Legend

- Creeks
- Former creeks, buried or drained - dashed where channel was ephemeral
- Underground culverts & storm drains
- Engineered channels
- Flood control channels > 100 feet wide
- Water spreads over the ground
- Water seeps out of the ground
- Present watersheds
- Artificial bodies of water
- Bay or slough
- Tidal marsh, circa 1850
  - Now water
  - Now filled or drained land
- Tidal marsh formed after 1850
- Fresh water marsh, modern
  red=artificial, blue=natural
- Willow groves, circa 1850

N

1:24,000

0 1,000 2,000 4,000 Feet

CREEK LOCATION MAP
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 4

Source: Creek and Watershed Map of Oakland and Berkeley; Sowers; 2000.

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate4-Creek_Map.mxd, 04/23/08, ksheil

FUGRO

122°18'0"W 122°17'0"W 122°16'0"W 122°15'0"W 122°14'0"W

37°51'0"N 37°50'0"N 37°49'0"N

Boundary between Oakland West and Oakland East Maps

Project Location

Source: USGS Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California; Graymer; 2000.

Legend

Contact

Faults (dashed where approx. located, small dashes where inferred, dotted where concealed, queried where location is uncertain)

Fault

Reverse or Thrust Fault

Normal Fault

70 Strike and Dip of Bedding

49 Strike and Dip of Bedding, top indicator observed

Geology

water

Surficial Deposits

af	Artificial Fill (Historic)	Qhl	Natural Levee Deposits (Holocene)
Qhasc	Artificial Stream Channels (Historic)	Qhsc	Stream Channel Deposits (Holocene)
Qhaf1	Younger Alluvial Fan Deposits (Holocene)	Qms	Merrit Sand (Holocene and Pleistocene)
Qhaf	Alluvial Fan and Fluvial Deposits (Holocene)	Qmt	Marine Terrace Deposits (Pleistocene)
Qhb	Basin Deposits (Holocene)	Qpaf	Alluvial Fan and Fluvial Deposits (Pleistocene)
Qhbr	Beach Ridge Deposits (Holocene)		

Great Valley Complex

Ku	Undivided Great Valley Complex Rocks (Cretaceous)
Ksh	Siltstone and Shale
KJk	Knoxville Formation (Early Cretaceous and Late Jurassic)
Jsv	Keratophyre and Quartz Keratophyre (Late Jurassic)

Coast Range Ophiolite

Jgb	Gabbro

Franciscan Complex

Kfn	Sandstone of the Novato Quarry Terrane of Blake and Others (1984) (Late Cretaceous)	fc	Serpentinite Blocks
Kfgm	Fine-grained Quartz Diorite (Late Cretaceous (?))	sp	Chert Blocks
KJfs	Franciscan Complex Sandstone, Undivided (Late Cretaceous to Late Jurassic)	fg	Greenstone Blocks
KJfm	Franciscan Complex, Melange (Cretaceous Late Jussaric)	fm	Schist Blocks
fs	Graywacke and Meta-Graywacke Blocks		

N

1:24,000

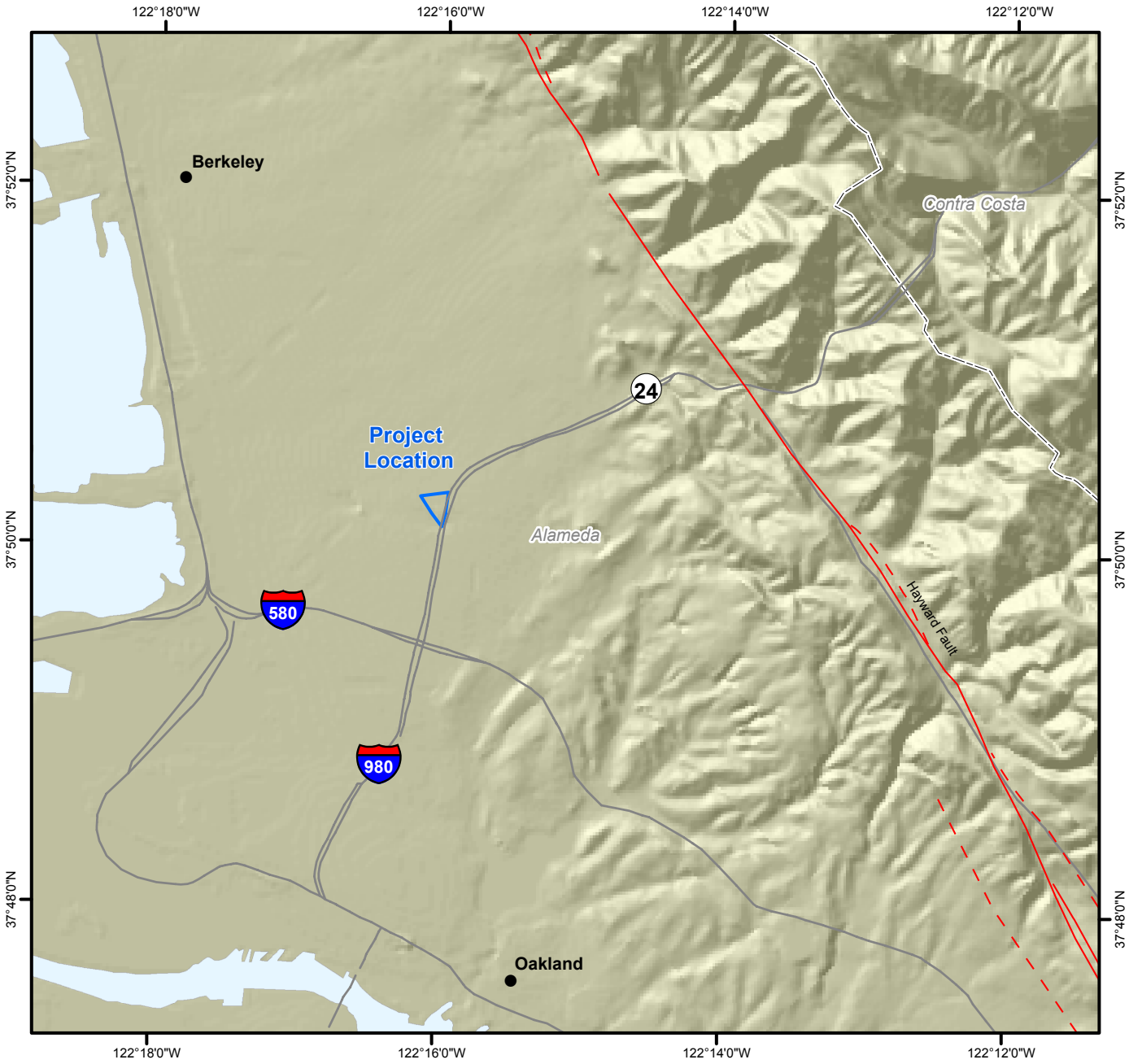
0 1,000 2,000 4,000 Feet

GEOLOGIC MAP

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 5

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate5-Geology.mxd, 04/23/08, ksheil



Legend

- Major Roads
- County Limits

Faults

- Active Fault, dashed where inferred
- Potentially Active Fault, dashed where inferred

Source: Bryant, 2005; Jennings, 1994; Fisher et al., 2005

N

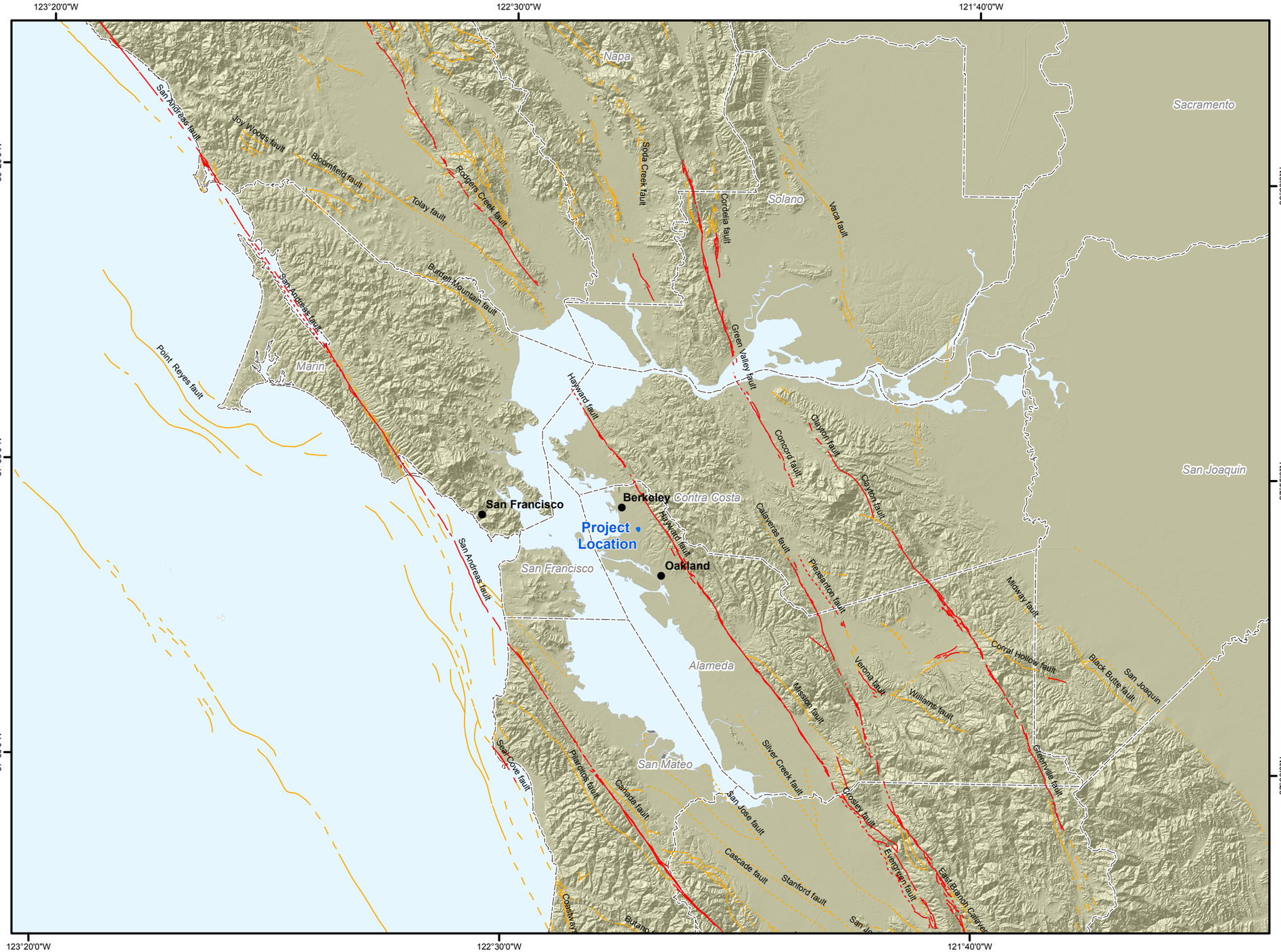
1:63,360

0 0.5 1 2 Miles

VICINITY FAULT MAP
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 6b

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate5b-Vacinity_Faults.mxd, 04/02/08, ksheil



Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Sources:

a) Bryant, 2005

b) Jennings, 1994

N

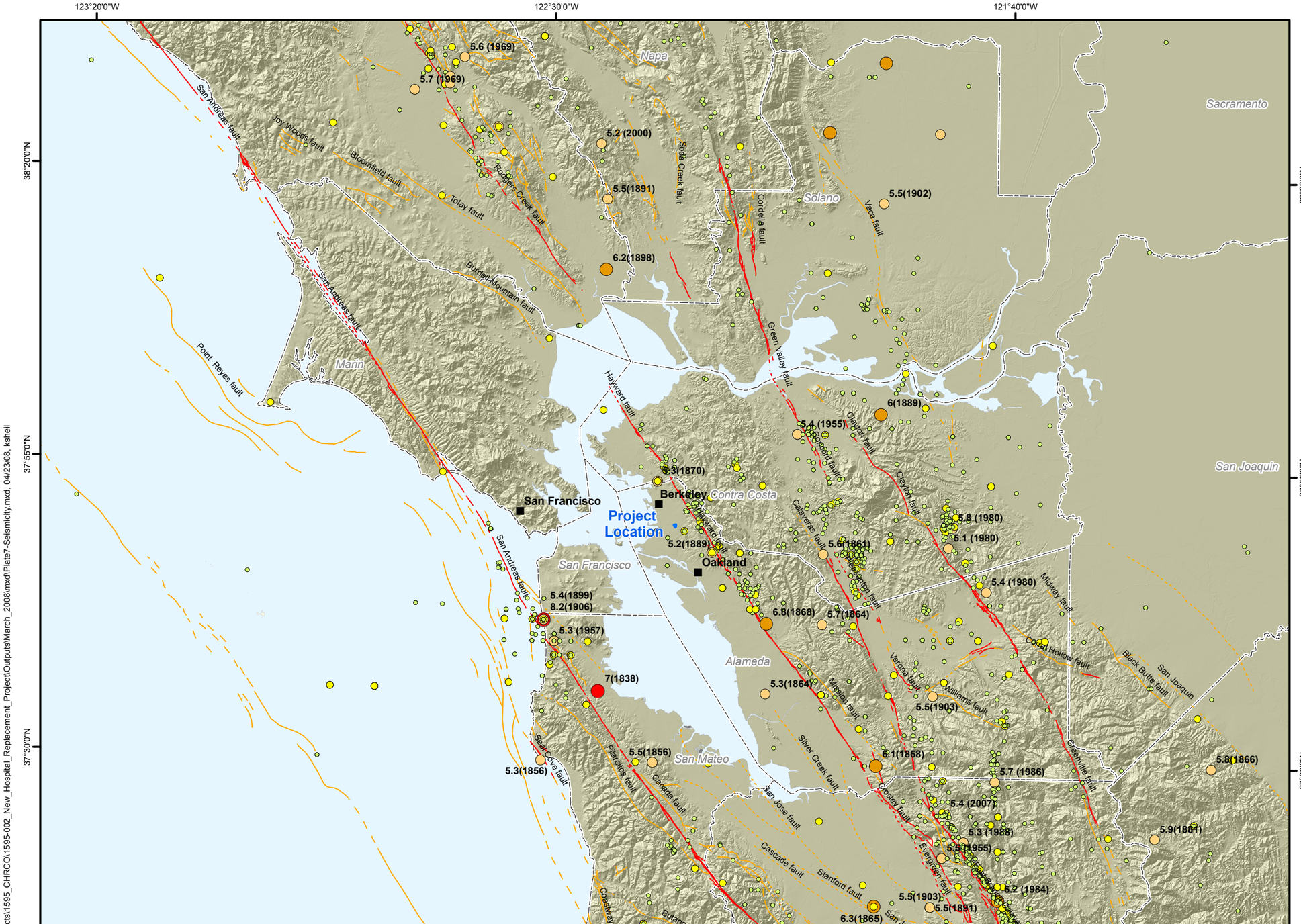
1:633,600

0 5 10 20 Miles

REGIONAL FAULT MAP
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 6a

Children's Hospital and Research Center Oakland
Project No. 1595.002



Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Historical Earthquake Magnitude (1836 - March 2008)
Magnitudes equal to and greater than 5 are labeled.

- 3.0 - 3.9
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7+

Sources:

1) Earthquake Data:
   a) Earthquake epicenters from ANSS Composite Catalog Search, 1898 to 2008, <www.ncedc.org/anss/> (downloaded March 2008)

   b) USGS Earthquake Hazards Program Search, 1735 to 1974, <http://neic.usgs.gov/neis/epic/epic_circ.html> (downloaded March 2008)

2) Faults:
   a) Bryant, 2005

   b) Jennings, 1994

N

1:633,600

0 5 10 20 Miles

FAULTING AND HISTORICAL SEISMICITY
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 7

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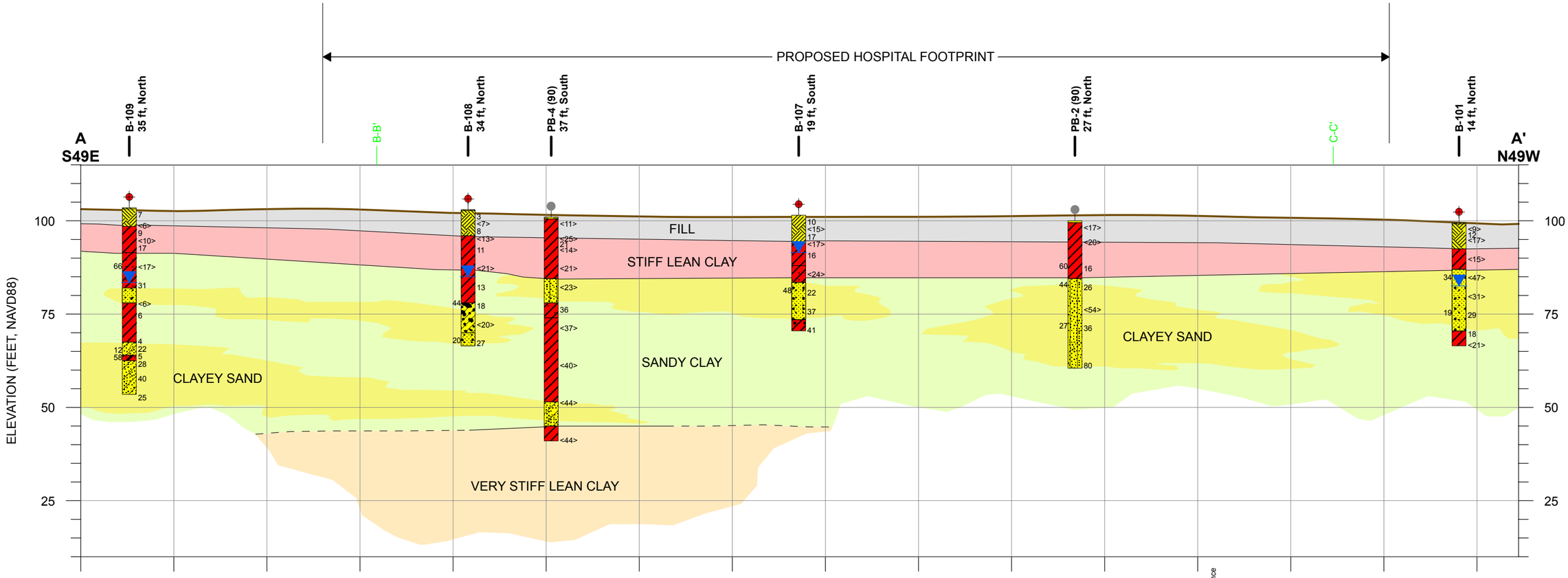
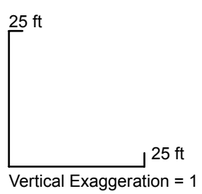


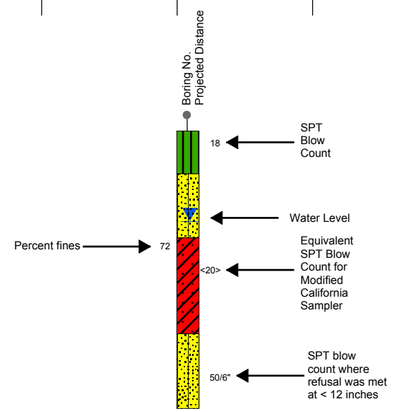
Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.



SOIL TYPES

- Fill
- GRAVEL (GP or GW)
- SAND (SP or SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL-S)
- SILT (ML)



SUBSURFACE CROSS SECTION A-A'
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 8a

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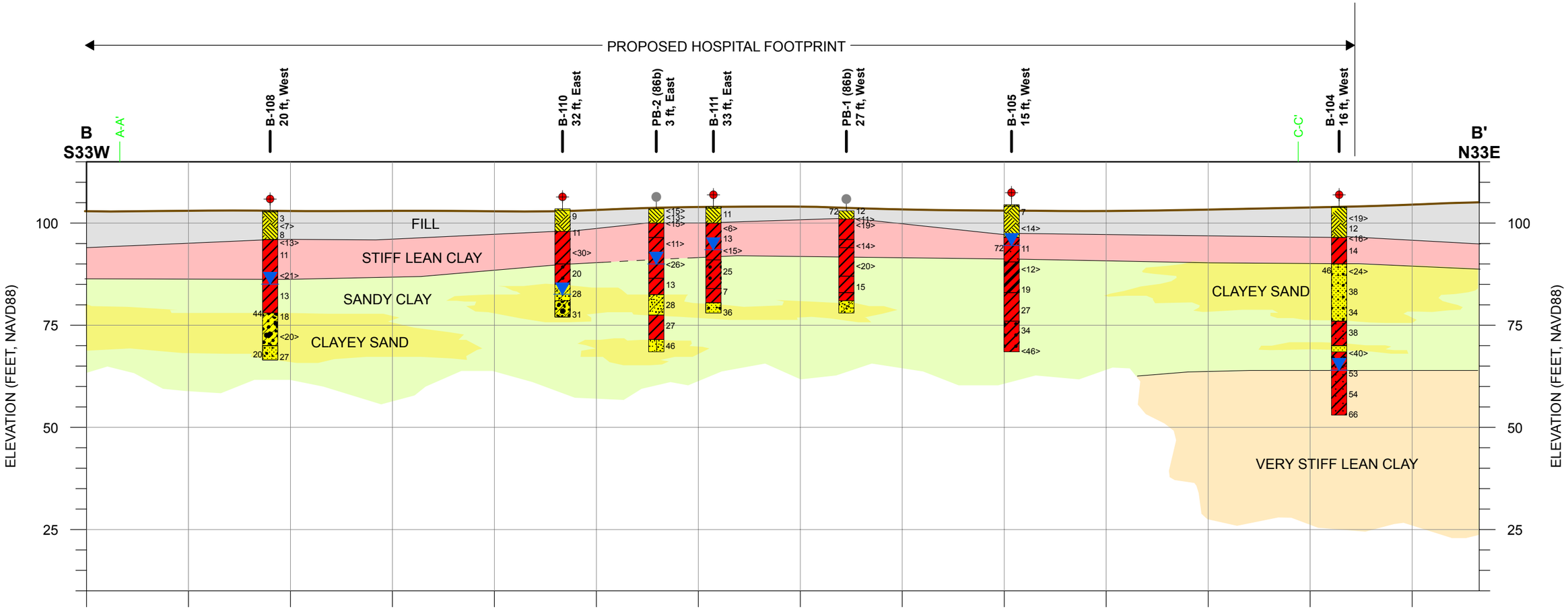

Notes:

1. Refer to Plate 3 for locations of cross section.
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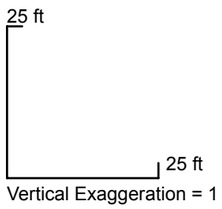
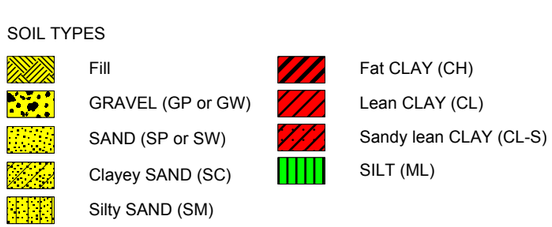
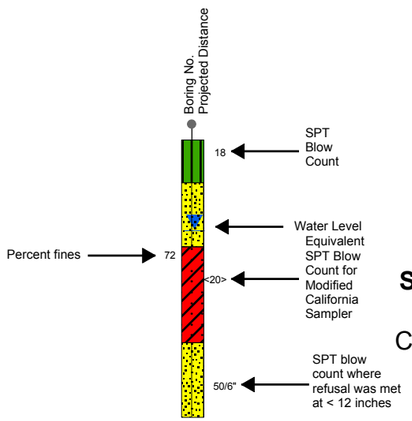
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.



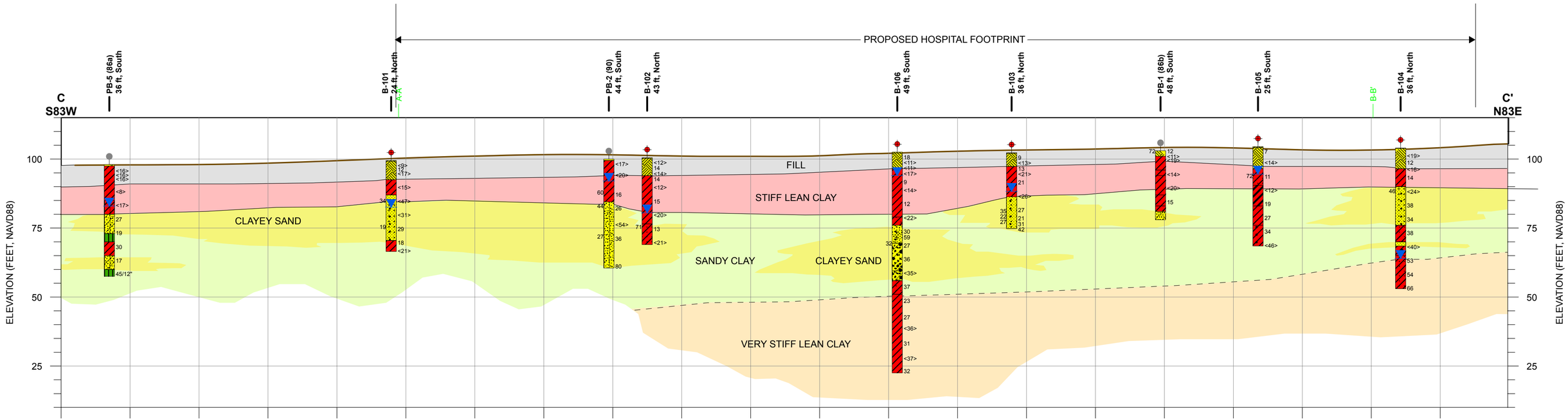




SUBSURFACE CROSS SECTION B-B'
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

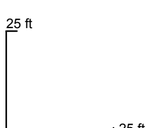
PLATE 8b

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate8b-Xsect_B.mxd, 04/24/08, ksheil



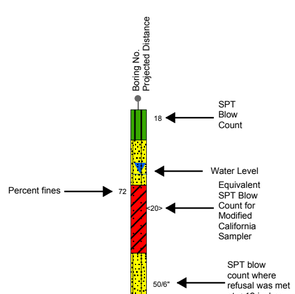
Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.



SOIL TYPES

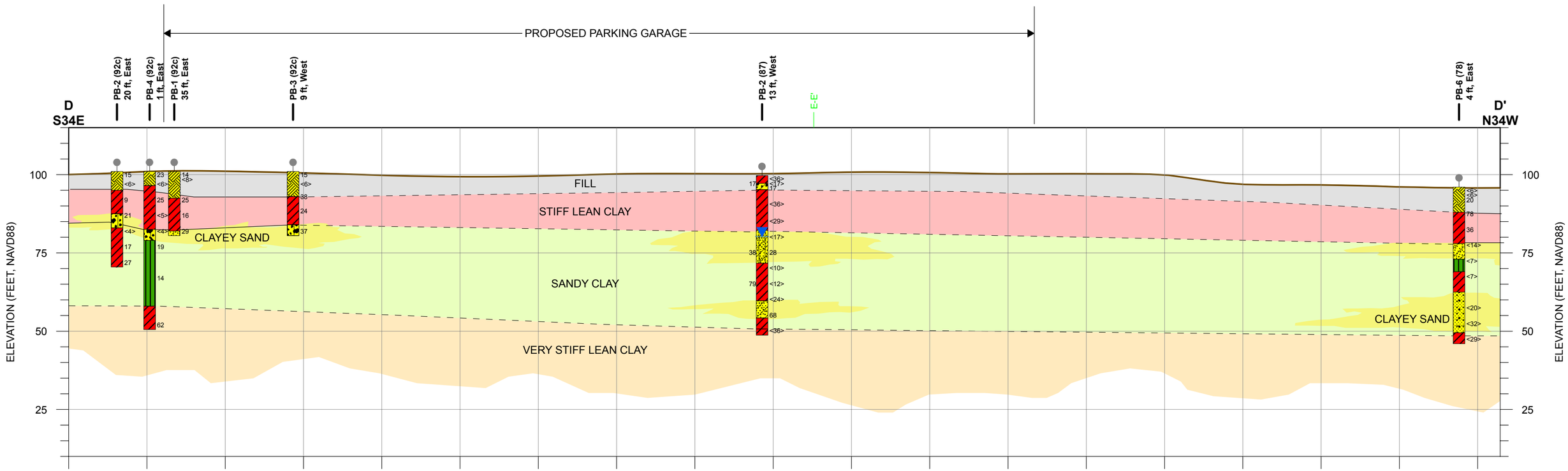
- Fill
- GRAVEL (GP or GW)
- SAND (SP or SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL-S)
- SILT (ML)



SUBSURFACE CROSS SECTION C-C'
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

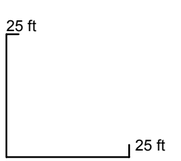
PLATE 8c

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate8c-Xsect_C.mxd, 04/24/08, kshell



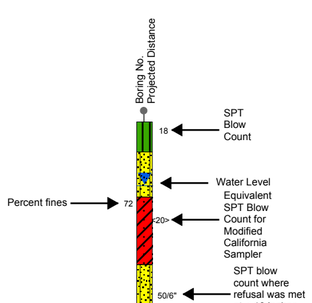
Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.



SOIL TYPES

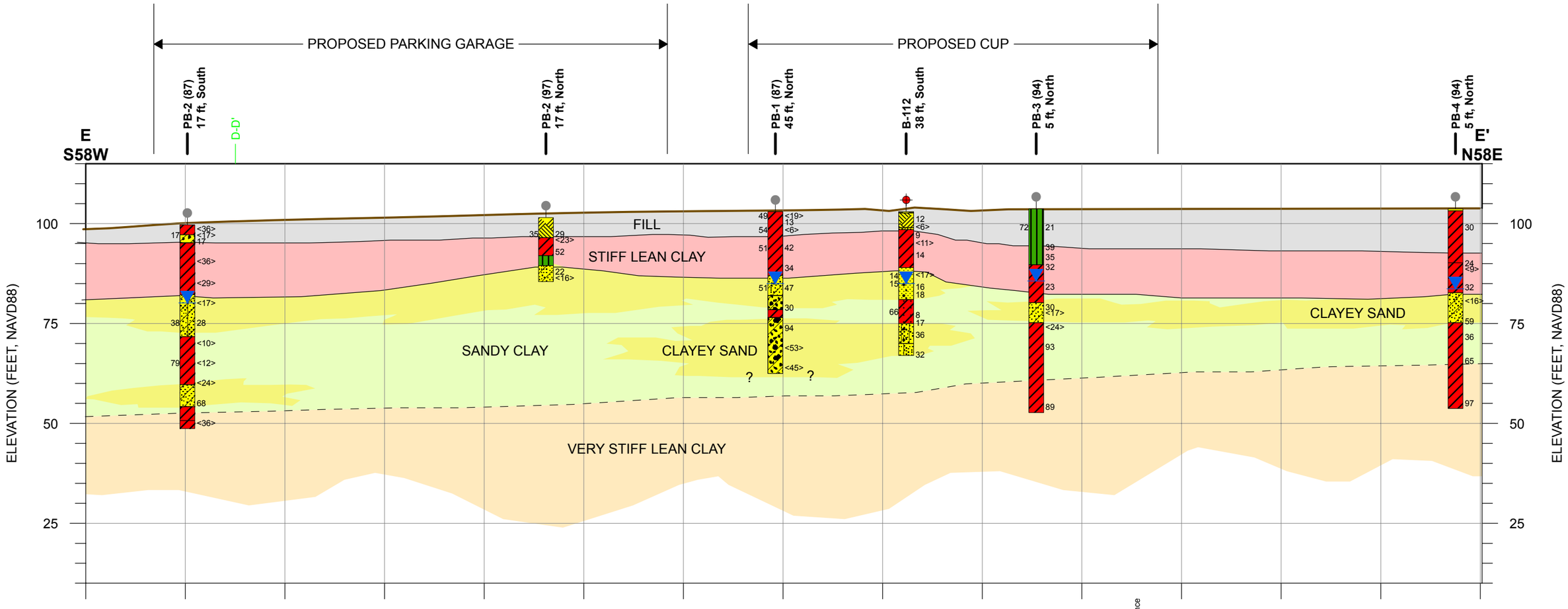
- Fill
- GRAVEL (GP or GW)
- SAND (SP or SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL-S)
- SILT (ML)



SUBSURFACE CROSS SECTION D-D'
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

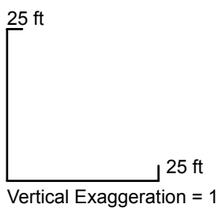
PLATE 8d

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate8d-Xsect_D.mxd, 04/24/08, ksheil



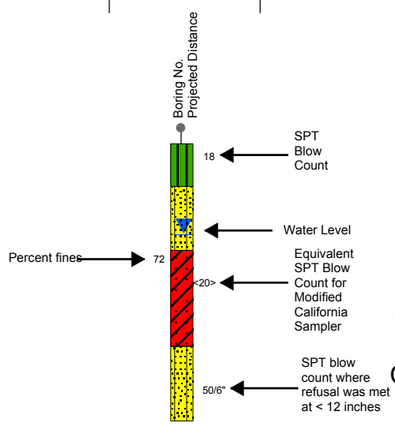
Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.



SOIL TYPES

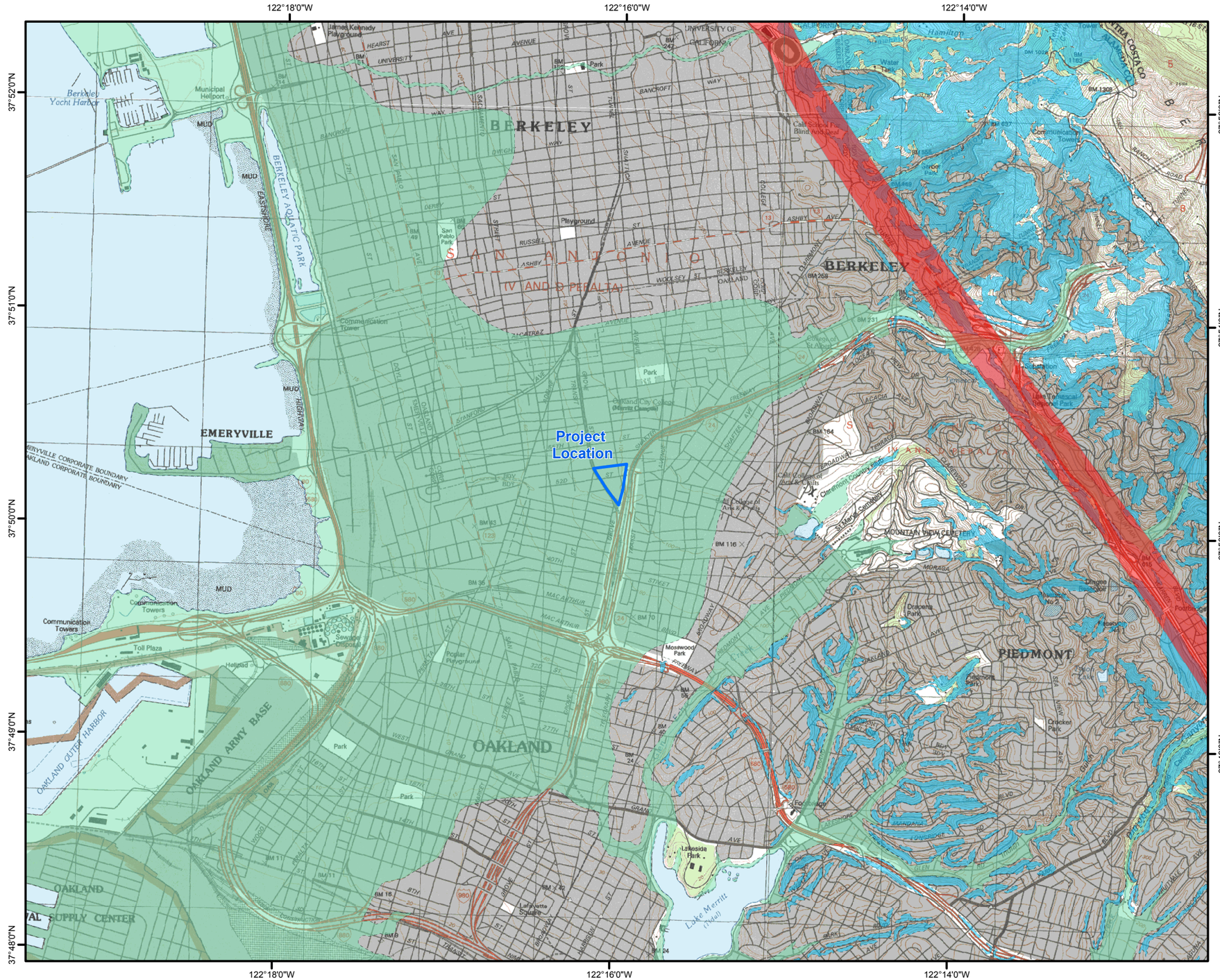
- Fill
- GRAVEL (GP or GW)
- SAND (SP or SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL-S)
- SILT (ML)



SUBSURFACE CROSS SECTION E-E'
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 8e

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate8e-Xsect_E.mxd, 04/24/08, ksheil



Legend

Liquefaction Hazard Zone
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacements.

Earthquake-Induced Landslide Hazard Zone
Areas where previous occurrence of landslide movement, or local topographic, and geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements.

Alquist-Priolo Earthquake Fault Zone

Sources:
1) Alquist-Priolo (AP) Special Studies Zone Boundaries: California Geologic Survey (2002)
2) Liquefaction and Landslide Special Studies Zones: California Geologic Survey <http://www.conservation.ca.gov/cgs/shzp/>

N

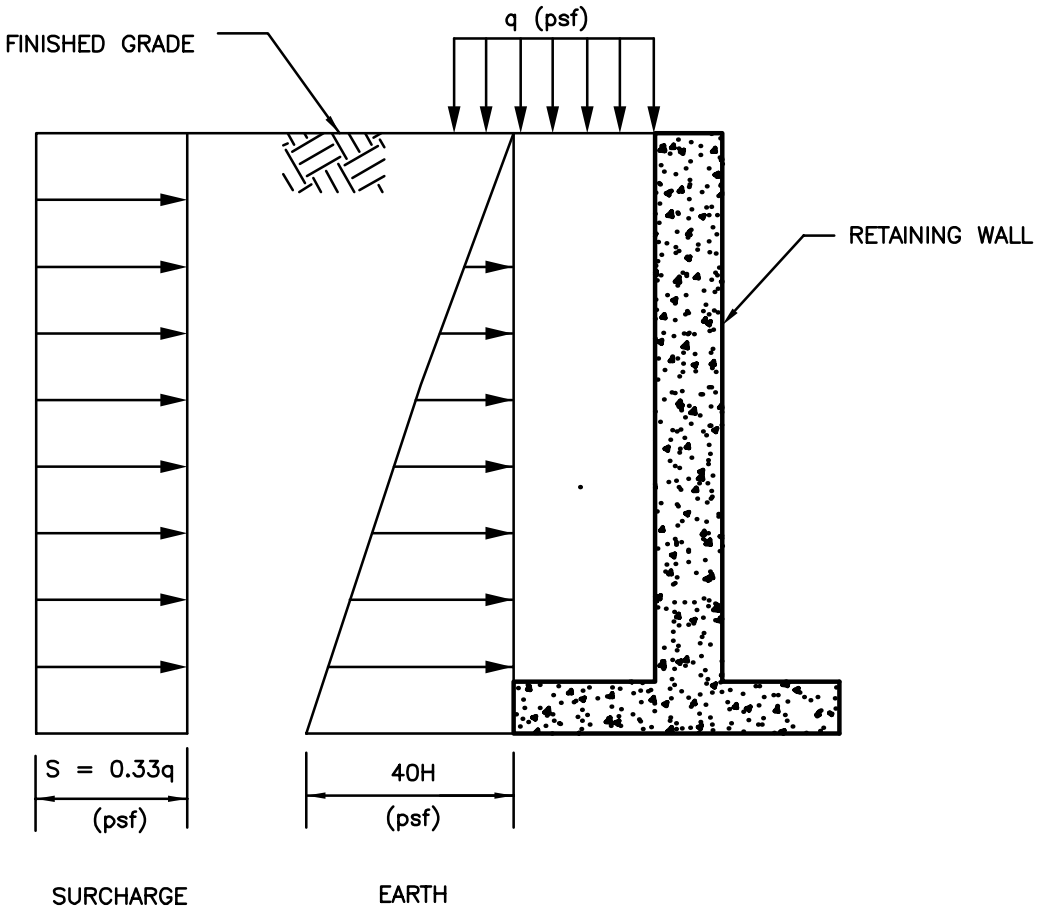
1:36,000

0 1,500 3,000 6,000 Feet

CGS SEISMIC HAZARD ZONE MAP
WITH ALQUIST-PRIOLO EARTHQUAKE ZONES
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, CA

PLATE 9

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate9-Seismic_Hazard.mxd, 04/22/08, ksheil



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.33q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

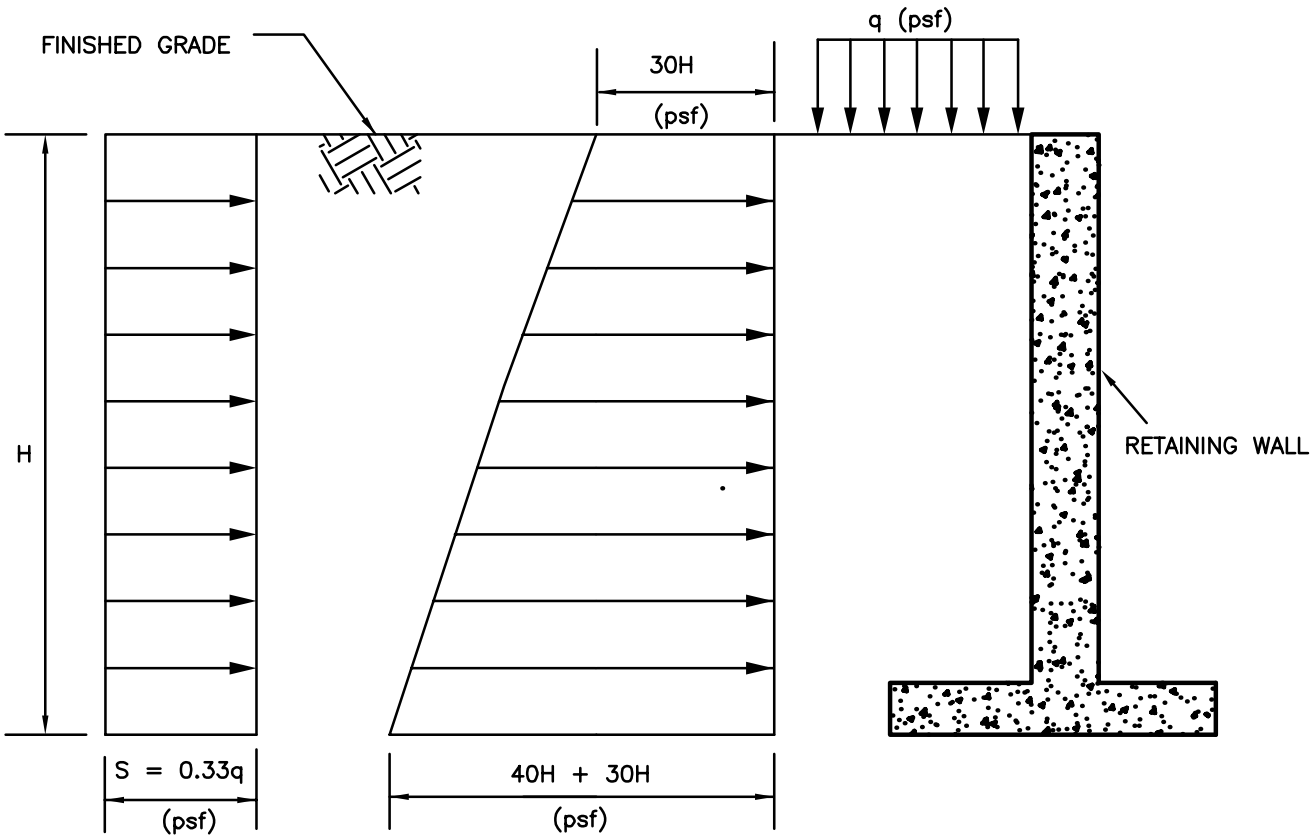
1) UNRESTRAINED WALLS ARE FREE TO MOVE/ROTATE.
2) WALLS ASSUMED TO BE FULLY BACK DRAINED
3) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL
4) ASSUMES LEVEL BACKFILL.

STATIC LATERAL PRESSURES ON UNRESTRAINED WALLS

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 10a

G: \jobdocs\1595\1595.002 CHRCO\Drawings\A1595_002_05.dwg 4-25-08 03:15:53 PM vtong



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.33q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

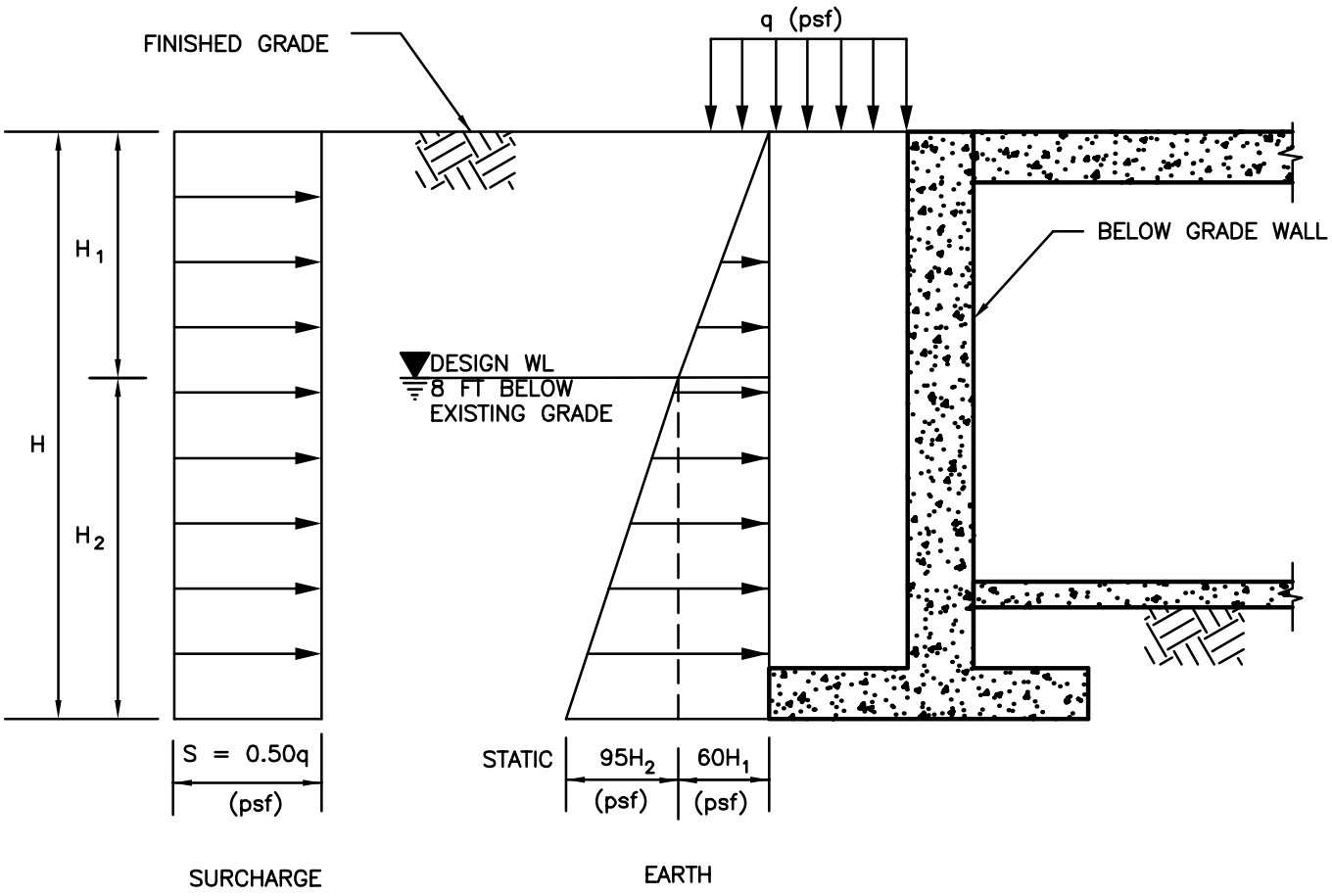
1) UNRESTRAINED WALLS ARE FREE TO MOVE/ROTATE.
2) WALLS ASSUMED TO BE FULLY BACK DRAINED
3) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL
4) ASSUMES LEVEL BACKFILL.

SEISMIC LATERAL PRESSURES ON UNRESTRAINED WALLS

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 10b

G: \jobdocs\1595\1595.002 CHRCO\Drawings\A1595_002_02.dwg 4-25-08 03:16:38 PM vtong



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.50q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

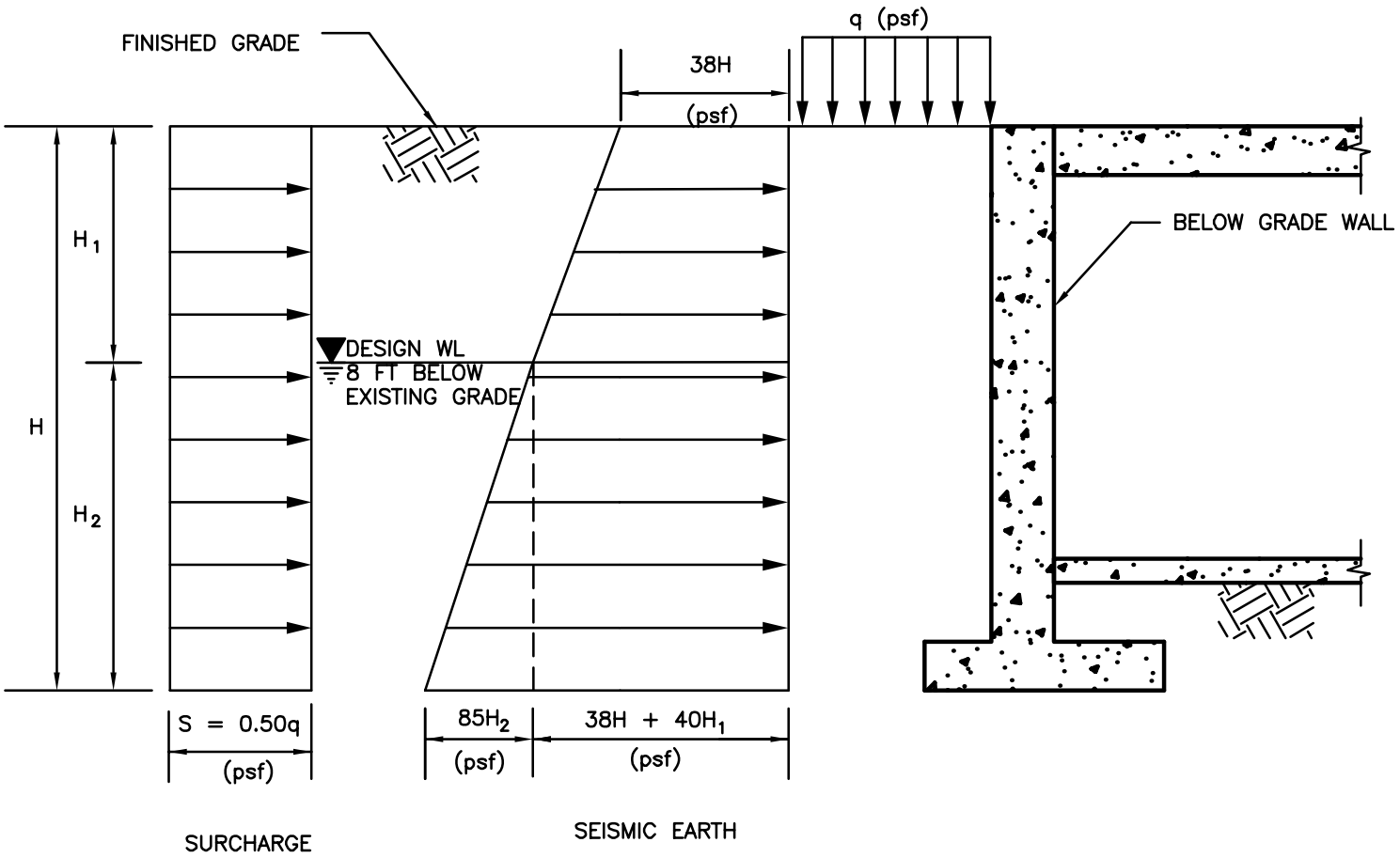
1) RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE.
2) DESIGN GROUND WATER LEVEL IS 8 FEET BELOW EXISITNG GRADE.
3) USE $60H_1$ ABOVE AND $95H_2$ BELOW THE DESIGN WATER LEVEL FOR BOTH STATIC AND SEISMIC LOADING CONDITIONS.
4) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL.
5) ASSUMES LEVEL BACKFILL.

STATIC LATERAL PRESSURES ON RESTRAINED WALLS

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 11a

G:\jobdocs\1595\1595.002 CHRCO\Drawings\A1595_002_04.dwg 4-25-08 03:56:23 PM vtong



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.50q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

1) RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE.
2) DESIGN GROUND WATER LEVEL IS 8 FEET BELOW EXISITNG GRADE.
3) USE $40H_1$ ABOVE AND $85H_2$ BELOW THE DESIGN WATER LEVEL FOR BOTH STATIC AND SEISMIC LOADING CONDITIONS.
4) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL.
5) ASSUMES LEVEL BACKFILL.

SEISMIC LATERAL PRESSURES ON RESTRAINED WALLS

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 11b

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MAJOR DIVISIONS			GROUP NAMES	
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW	Well-Graded Gravel
			GP	Poorly Graded Gravel
		Gravels with more than 12% fines	GM	Silty Gravel
			GC	Clayey Gravel
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	Clean sand less than 5% fines	SW	Well-Graded Sand
			SP	Poorly Graded Sand
		Sands with more than 12% fines	SM	Silty Sand
			SC	Clayey Sand
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML	Silt
			CL	Lean Clay
			OL	Organic Silt
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH	Elastic Silt
			CH	Fat Clay
			OH	Organic Clay
HIGHLY ORGANIC SOILS			PT	Peat or Highly Organic Soils
			FILL	Debris or Mixed Fill
			AC	Asphalt Concrete Pavement with Aggregate Base

GENERAL NOTES

Classification of Soils per ASTM D2487 or D2488

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

Sloped line in break column indicates transitional boundary

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

SAMPLER DRIVING RESISTANCE

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

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
50/7"	50 blows drove sampler 7" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

STRENGTH TEST METHOD

U = Unconfined Compression
Q = Unconsolidated Undrained Triaxial
T = Torvane
P = Pocket Penetrometer
M = Miniature Vane
F = Field Vane

OTHER TESTS

k = Permeability
Consol = Consolidation
Gs = Specific Gravity
MA = Particle Size Analysis
EI = Expansion Index
OVM = Organic Vapor Measurement

WATER LEVEL SYMBOLS

- Initial or perched water level
- Final ground water level
- Seepages encountered

SAMPLER TYPE

1 SPT, 2 MC, 3 CA, 4 SH, 5 BB, 6 HA, 7 LS, 8 PS, 9 VS, 10 NR, 11 RC

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

1. SPT Sampler, driven 1 3/8" ID, 2" OD
2. MOD CA Liner Sampler 2 3/8" ID, 3" OD
3. CA Liner Sampler 1 7/8" ID, 2.5" OD
4. Thin-walled Tube, pushed 2 7/8" ID, 3" OD
5. Bulk Bag Sample (from cuttings)
6. Hand Auger Sample
7. Lexan Sample
8. Pitcher Sample
9. Vibracore Sample
10. No Sample Recovered
11. Rock Core

SOIL STRUCTURE

Fissured: Containing shrinkage or relief cracks, often filled with fine sand or silt, usually more or less vertical.

Pocket: Inclusion of material of different texture that is smaller than the diameter of the sample.

Parting: Inclusion less than 1/8 inch thick extending through the sample.

Seam: Inclusion 1/8 inch to 3 inches thick extending through the sample.

Layer: Inclusion greater than 3 inches thick extending through the sample.

Laminated: Soil sample composed of alternating partings or seams of different soil types.

Interlayered: Soil sample composed of alternating layers of different soil type.

Intermixed: Soil sample composed of pockets of different soil type, and layered or laminated structure is not evident.

CONSISTENCY

Clays	Blows/Foot SPT	Undrained Shear Strength (ksf)
Very Soft	0 - 2	0 - 0.25
Soft	2 - 4	0.25 - 0.5
Firm	4 - 8	0.5 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

RELATIVE DENSITY

Sands and Gravels	Blows/Foot SPT
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

INCREASING VISUAL MOISTURE CONTENT

Dry → Moist → Wet

Information on each boring log is a compilation of subsurface conditions and soil or rock classifications obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the time and places indicated, and can vary with time, geologic condition, or construction activity.

TERMS AND SYMBOLS USED ON BORING LOGS PLATE A-1

LOCATION: SEE SITE PLAN, PLATE 3
N 2132471 E 6051342
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 99.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
					Topsoil 4 inches							
		1			**ARTIFICIAL FILL (af)**							
		2		(15)	Lean CLAY (CL): stiff, black, moist, trace rootlets, faint organic odor						1.5 P	
		3		12								
95 / 5		4		(28)		103	23				2.0 P	
		5			- dark brown, below 5'						2.5 P	
					ALLUVIUM (Qal)							
					Lean CLAY (CL): stiff to very stiff, brown, mottled orange, moist, medium plasticity, trace angular gravel, trace silt	~~102~~	~~23~~				~~1.5 P~~	
90 / 10		6		(24)								
					Clayey SAND with gravel (SC): very dense, light brown, moist, low plasticity, subrounded to subangular, trace medium-to coarse-grained sand							
85 / 15		7		(76)		107	18	34			3.5 M+	
		8			▽ - grayish brown, angular to subrounded, with fine-to medium-grained sand, below 17'							
80 / 20		9		(49)		111	18				3.5 M+	
		10										
75 / 25		11		29	- medium dense, less fines, below 24'			19				
70 / 30		12		18	Lean CLAY (CL): very stiff, brown, mottled dark brown, wet, medium plasticity						2.3 P	
		13		(33)							2.0 P	
65 / 35					End of Boring at 33'							
60												

BORING DEPTH: 32.0 ft
BACKFILL: Grout
DEPTH TO WATER: 17.0 ft
COMPLETION DATE: February 11, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-101
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132502 E 6051432
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 100.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 0				Topsoil 3 inches							
	2		(20)	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, black, moist, trace rootlets, faint organic odor						1.5 P	R-value = 4
	3		14								
95 / 5	4 5		(23)							2.5 P 3.5 P	
	6		14	**ALLUVIUM (Qal)** Lean CLAY with sand (CL): stiff, brown, mottled orange, moist, trace roots, fine grained sand							
90 / 10	7 8		(20)	- greenish gray, trace angular to subangular gravel, at10'						2.5 P	
85 / 15	9		15	- gray, trace angular fine gravel, below 15'						3.0 P	
80 / 20	10 11		(32)	- increased gravel content, below 20'		20		35	21		
75 / 25	12		13	Sandy Lean CLAY (CL): stiff, brown, wet, low to medium plasticity, trace angular fine gravel		20	71	23	11		
70 / 30	13 14		(33)	- increased gravel content, at 31'							
65 / 35				End of Boring at 31.5'							

BORING DEPTH: 31.5 ft
BACKFILL: Grout
DEPTH TO WATER: 20.0 ft
COMPLETION DATE: February 11, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-102
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

Sheet 1 of 1

LOCATION: SEE SITE PLAN, PLATE 3
N 2132512 E 6051564
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 102.3 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 0				Topsoil 3 inches							
	1		9	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, black, moist, trace rootlets, medium to high plasticity						2.0 P	
	2 3		(21)								
95 / 5	4		13	**ALLUVIUM (Qal)** Lean CLAY (CL): stiff to very stiff, grayish brown, moist, medium to high plasticity						2.5 P	
	5 6		(34)		100	23					
90 / 10	7		21	- very stiff, mottled orange, below 10'							
85 / 15	8 9		(41)	Clayey SAND with gravel (SC): medium dense, brown, wet, fine-to coarse-grained, low to medium plasticity, subangular to subrounded fine gravel							
80 / 20	10		27	- 4 inch thick clay layer from 20' to 20'4". - Fine SAND with clay, low to medium plasticity, from 21' to 22'			35				
	11						22				
75 / 25	12		21								
	13		31	- dense, increased gravel, below 25' - pockets of green silt, at 26'			27				
70 / 30	14		42								
65 / 35				End of Boring at 27.5'							

BORING DEPTH: 27.5 ft
BACKFILL: Grout
DEPTH TO WATER: 14.0 ft
COMPLETION DATE: February 15, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-103
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132530 E 6051704
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 104.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
				Asphalt Concrete Pavement - 2 inches							
100 / 5	1		(30)	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff to very stiff, black, moist, medium plasticity	107	17		43	26	1.0 P	Corrosion
	2		12								
	3		(26)	**ALLUVIUM (Qal)** Lean CLAY (CL): stiff, brownish gray, moist, low to medium plasticity, traces of red iron stains						2.0 P	
95 / 10	4										
	5		14							2.0 P	
90 / 15	6		(38)	Clayey SAND with gravel (SC): dense, brown, moist, very fine-grained, low plasticity, subangular to subrounded gravel			46			3.0 P	
	7										
85 / 20	8		38								
80 / 25	9		34	- less gravel, mottled black with red iron stains and black seams, below 25'							
75 / 30	10		38	Lean CLAY with sand (CL): hard, brown, mottled black, with red iron stains, moist, low to medium plasticity, fine-to coarse-grained sand, trace subangular to subrounded fine gravel							
70 / 35	11		(64)	Poorly-graded SAND (SP): dense, brown, moist, medium grained							
	12			Gravelly Lean CLAY (CL): hard, brown, moist, subangular to angular, fine to coarse gravel							
65				Sandy Lean CLAY (CL): hard, brown, moist, low							

Continued

BORING DEPTH: 51.2 ft
BACKFILL: Grout
DEPTH TO WATER: 40.0 ft
COMPLETION DATE: February 15, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-104
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

LOCATION: SEE SITE PLAN, PLATE 3
N 2132530 E 6051704
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 104.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
60 / 45	13		53	plasticity, fine-grained sand, trace subangular fine gravel							
55 / 50	14		54	- increased gravel content, from 45' to 46'							
50 / 55	15		66	- increased plasticity, with red iron stains, at 50'							
				End of Boring at 51'							
45 / 60											
40 / 65											
35 / 70											
30 / 75											
25											

BORING DEPTH: 51.2 ft
BACKFILL: Grout
DEPTH TO WATER: 40.0 ft
COMPLETION DATE: February 15, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-104
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

BORING LOG OAK G:\JOBDOCS\1595\1595.002 CHRCO\GINT\1595.002.GPJ LIBRARY_092906OAK.GLB 4/22/08 12:54 p

LOCATION: SEE SITE PLAN, PLATE 3
N 2132463 E 6051660
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 104.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
	1		7	Topsoil 4 inches **ARTIFICIAL FILL (af)** Lean CLAY (CL): firm to stiff, brown, moist, trace rootlets		26		41	24	1.0 P	
100 5	2 3		(22)	- grades to blackish gray, below 5'	100	23		41	28	2.0 P	
				ALLUVIUM (Qal)							
95 10	4 5		11	Sandy Lean CLAY (CL): stiff, brown, wet, low to medium plasticity, very fine-grained sand			72				
90 15	6 7		(20)	Fat CLAY (CH): stiff, gray, wet, high plasticity, trace fine-grained sand		27				2.5 P	Corrosion
85 20	8		19	- very stiff, below 20' - pockets of subangular gravel up to 3/4-inch in size at 20'						3.5 P	
				Lean CLAY with sand (CL): very stiff, brown, wet, fine-to medium-grained sand, trace subangular to angular gravel							
80 25	9		27							4.5 P	
75 30	10		34	Gravelly Lean CLAY with sand (CL): hard, brown, moist, angular to subangular gravel, fine-to medium-grained sand							
70 35			(74)	- increased gravel content, at 35'							
65				End of Boring at 36'							

BORING DEPTH: 36.3 ft
BACKFILL: Grout
DEPTH TO WATER: 10.0 ft
COMPLETION DATE: February 15, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-105
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132422 E 6051534
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 102.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
	2		18	Topsoil **ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff to very stiff, brownish black, moist, trace rootlets, organic odor, low to medium plasticity						3.0 P	R-Value = 9
100 5	3 4 5		(17)							2.0 P	
			(17)	**ALLUVIUM (Qal)** Lean CLAY (CL): very stiff, brown, mottled orange, moist, low plasticity, trace fine-grained sand							
95	6 7		(28)							3.0 P	
10	8		9	- medium to high plasticity, trace silt, trace angular to subrounded, fine gravel, below 10'						1.5 P	
90	9 10 11		(23)	- sand seams, at 13.5'	105	21				3.3 M 0.9 P 3.5 P	
15	12		12	- mottled orange and black, with red iron stains, below 18' - without gravel, below 19'						3.0 P	
85 20											
80	13 14		(35)	Sandy Lean CLAY with gravel (CL): very stiff, brown, with red iron stains, moist, medium plasticity, fine-grained sand, angular to subrounded coarse gravel up to 1/2-inch diameter	108	20				3.5 M+ 4.0 P	
25	15		30	Well-graded SAND with gravel (SW): dense, brown, moist, fine-to coarse-grained sand, subangular to subrounded fine gravel, Intermixed with clay						4.0 P	
75 30	16		59								
70	17		27	Clayey GRAVEL (GC): medium dense to dense, brown, with red iron stains, moist, angular to subrounded, fine to coarse gravel, with fine-grained sand, medium plasticity			33				
35 65	18		36								

Continued

BORING DEPTH: 80.0 ft
BACKFILL: Grout
DEPTH TO WATER: 8.5 ft
COMPLETION DATE: February 12, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger to 11.5', Rotary Wash from 11.5' to 80'
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-106
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132422 E 6051534
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 102.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
60 / 45	19 20		(56)	Clayey GRAVEL (GC) (continued)							
				Clayey GRAVEL with sand (GC): very dense, brownish gray, moist, angular to subrounded fine to coarse gravel up to 1-inch in size, medium to high plasticity, fine-to medium-grained sand							
55 / 50	21		37	Lean CLAY with sand (CL): hard, brown, mottled black, with red iron stains, moist, fine-grained sand, trace fine gravel						4.0 P	
				Lean CLAY with sand (CL) (continued)							
50 / 55	22		23	Lean CLAY (CL): very stiff, brown, moist, trace silt, trace fine-grained sand						4.5 P	
45 / 60	23		27								
				- hard drilling, with gravel, at 62'							
40 / 65	24 25		(57)	- sand pockets, increased plasticity at 64'						3.5 M+ 4.0 P	
35 / 70				- with fine gravel, below 67'							
	26		31	- increasing sand content, below 69'						4.0 P	
30 / 75	27 28		(59)	Lean CLAY (CL) (continued) - less gravel, below 75'						3.5 M+ 4.5 P	
25	29		32	End of Boring at 80'							

BORING DEPTH: 80.0 ft
BACKFILL: Grout
DEPTH TO WATER: 8.5 ft
COMPLETION DATE: February 12, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger to 11.5', Rotary Wash from 11.5' to 80'
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-106
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132330 E 6051455
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 101.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 0	1		10	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, black, moist, trace rootlets						2.0 P	
	2 3		(24)							2.5 P	
5 / 95	4		17	- very stiff, at 5'						3.5 P	
	5 6		(28)	**ALLUVIUM (Qal)**						3.5 P	
10 / 90	7		16	Lean CLAY (CL): stiff, brown, moist, trace fine-grained sand, trace subrounded to subangular gravel		18		29	16		
				- sand content increases							
				- pockets of fat clay							
15 / 85	8 9		(38)	Sandy Lean CLAY with gravel (CL): very stiff, brown, wet, medium plasticity, fine-to coarse-grained sand, subrounded to subangular fine to medium gravel							
20 / 80	10		22	Clayey SAND with gravel (SC): dense, brown, wet, fine-to coarse-grained, medium plasticity, subrounded to angular gravel			48				
25 / 75	11		37								
				Gravelly Lean CLAY with sand (CL): hard, brown, moist, medium to high plasticity, angular to subrounded fine to medium gravel, fine-to coarse-grained sand							
30 / 70	12		41	End of Boring at 31'							
35 / 65											

BORING DEPTH: 31.0 ft
BACKFILL: Grout
DEPTH TO WATER: 10.0 ft
COMPLETION DATE: February 14, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-107
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132313 E 6051557
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 103.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 0	1		3	Topsoil **ARTIFICIAL FILL (af)** Lean CLAY (CL): soft to firm, black, moist, trace rootlets		20		38	22	1.0 P	
5	2 3		(12)							1.0 P	
	4		8								
95 / 10	5 6		(21)	**ALLUVIUM (Qal)** Lean CLAY (CL): stiff, brown, moist, medium to high plasticity, trace subangular to subrounded fine gravels, trace fine-to coarse-grained sand						3.5 P	
	7		11			21		41	29	4.0 P	
90 / 15	8 9		(33)	- very stiff, increased gravel content, below 15'							
85				▽							
20	10		13	- wet, with fine-grained sand, trace subangular to subrounded fine gravel		25		42	27		
80 / 25	11		18	Clayey GRAVEL (GC): medium dense to dense, gray, mottled orange, wet, medium plasticity, subrounded to subangular, trace fine-to medium-grained sand			44				
75 / 30	12 13		(32)	- decreasing gravel content, below 30'						4.5 P	
70 / 35	14		27	Clayey SAND with gravel (SC): medium dense, brown, wet, fine-to coarse-grained, low plasticity, subangular to subrounded gravel			20				
65				End of Boring at 36.5'							

BORING DEPTH: 36.5 ft
BACKFILL: Grout
DEPTH TO WATER: 18.0 ft
COMPLETION DATE: February 14, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-108
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132255 E 6051627
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 103.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 0	1		7	Topsoil **ARTIFICIAL FILL (af)** Lean CLAY (CL): firm, black, moist, low to moderate plasticity, trace rounded gravel							
5	2 3		(9)		107	17				1.5 P	
			9	**ALLUVIUM (Qal)** Lean CLAY (CL): stiff, brown, mottled black, moist, low to medium plasticity, trace angular gravel up to about 1/2-inch diameter							
95 / 10	4 5		(16)		110	18				3.0 P	
	6		17								
90 / 15	7 8		(28)	Sandy Lean CLAY with gravel (CL): very stiff, brown, moist, low plasticity, fine-grained sand, angular to rounded fine gravel	104	19 19	66			3.5 M+	
85				▽							
20	9		31				21			4.3 P	
80 / 25	10 11			Clayey SAND with gravel (SC): dense, light gray, moist, medium-to coarse-grained, low to medium plasticity, angular to subrounded fine gravel							
	12 13		(10)	Lean CLAY (CL): firm, brown, moist to wet, medium to high plasticity fines; trace fine gravel and sand.						1.0 P	
75 / 30	14		6	- low plasticity, mottled orange, below 28'						0.6 P	
70 / 35	15		4	- increased sand content, at 35'						0.4 P	
	16 17			Well-graded SAND with clay (SW-SC): medium dense, dark gray, moist to wet, fine to medium grained, pockets of rounded to subrounded fine gravel at 37'		21	12	30	16		
65	18 19		22								
	20		5								

Continued

BORING DEPTH: 50.0 ft
BACKFILL: Grout
DEPTH TO WATER: 20.0 ft
COMPLETION DATE: February 13, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger to 35', Rotary Wash from 35' to 50'
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-109
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132255 E 6051627
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 103.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
60 / 45	21		28	- increasing gravel content, subangualar to subrounded, below 38' - 1-foot thick layer of well graded gravel, at 39.5'			58				
				Sandy Fat CLAY (CH): firm, gray, wet, trace fine-grained sand							
55 / 50	22		40	Clayey SAND with gravel (SC): medium dense to dense, brown, moist, fine-to coarse-grained sand, medium to high plasticity, angular to subrounded gravel - becomes dense, granularity increases, at 45' - increased gravel, at 50'							
50 / 55	23		25	End of Boring at 50'							
45 / 60											
40 / 65											
35 / 70											
30 / 75											
25											

BORING DEPTH: 50.0 ft
BACKFILL: Grout
DEPTH TO WATER: 20.0 ft
COMPLETION DATE: February 13, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger to 35', Rotary Wash from 35' to 50'
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-109

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 2132344 E 6051640
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 103.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
				Topsoil							
100 / 5	1		9	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, black, moist, with rootlets						2.5 P	
	2		11	**ALLUVIUM (Qal)** Lean CLAY (CL): stiff, brown, mottled black and orange, moist, low plasticity, trace subangular fine gravel						2.5 P	
95 / 10	3 4		(48)	- very stiff, below 10'						4.5 P	
90 / 15	5		20	Gravelly Lean CLAY with sand (CL): very stiff, brown, moist, low plasticity, subangular to subrounded fine to coarse gravel							
85 / 20	6		28	Clayey SAND (SC): medium dense, brown, moist, medium-to coarse-grained sand, low plasticity, subangular to subrounded gravel							
80 / 25	7		31	Clayey GRAVEL with sand (GC): dense, brown, wet, subangular to subrounded fine to medium gravel, medium plasticity, fine-to medium-grained sand							
75 / 30				End of Boring at 26.5'							
70 / 35											
65											

BORING DEPTH: 26.5 ft
BACKFILL: Grout
DEPTH TO WATER: 21.0 ft
COMPLETION DATE: February 13, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-110

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

Sheet 1 of 1

LOCATION: SEE SITE PLAN, PLATE 3
N 2132375 E 6051661
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 104.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 5	1		11	Topsoil **ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, black, moist, trace rootlets, with red iron stains						2.0 P	
	2 3		(9)	**ALLUVIUM (Qal)** Lean CLAY (CL): firm to stiff, brown, moist, low plasticity, trace fine gravel	93	27				1.0 P	
95 / 10	4		13	- increased plasticity, below 8'						2.0 P	
	5 6		(24)	▽ - wet, stiff, at 10'	107	21				2.5 P	
90 / 15	7		25	Gravelly Lean CLAY with sand (CL): very stiff, brown, wet, low to medium plasticity, angular to subrounded fine to coarse gravel, fine-to medium-grained sand						4.5 P	
85 / 20	8		7	Lean CLAY (CL): firm, brown, wet, medium plasticity, trace fine-grained sand, fine to coarse gravel						1.5 P	
80 / 25	9 10		36	Clayey SAND with gravel (SC): dense, brown, wet, medium-to coarse-grained sand, medium plasticity, angular to subrounded fine to coarse gravel							
75 / 30				End of Boring at 26'							
70 / 35											
65											

BORING DEPTH: 26.0 ft
BACKFILL: Grout
DEPTH TO WATER: 10.5 ft
COMPLETION DATE: February 13, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-111
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

Sheet 1 of 1

LOCATION: SEE SITE PLAN, PLATE 3
N 2131764 E 6051547
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 103.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 5	1		12	Asphalt Concrete Pavement **ARTIFICIAL FILL (af)** Lean CLAY (CL): firm to stiff, black, moist, trace fine gravel							
	2 3		(9)		92	22				1.8 M 0.5 R	Corrosion
	4		9	- 6" clayey sand layer, trace gravel, from 4 to 4.5'							
95 / 10	5 6		(18)	**ALLUVIUM (Qal)** Lean CLAY with sand (CL): stiff to very stiff, brown, moist, fine-grained sand, low plasticity fines, trace subangular to subrounded fine gravel	102	23				2.0 P 3.0 M 1.0 R 2.5 P	
	7		14	- low to medium plasticity, trace fine-grained sand						3.5 P	
90 / 15	8 9		(28)	Clayey SAND with gravel (SC): medium dense, brown, moist to wet, fine- to coarse-grained sand, well graded, subangular to subrounded fine to coarse gravel up to 1.25-inch in size; medium plasticity			14				
85 / 20	10		16	▽ - with red iron staining, at 18'		17	15	31	17		
	11		18	- increased sand content, at 20'							
80 / 25	12		8	Sandy Lean CLAY (CL): firm, gray, wet, very fine-grained sand, low plasticity			66				
75 / 30	13		17								
	14		36	Silty SAND with gravel (SM): dense, brown, moist, fine-to coarse-grained sand, medium plasticity, subangular to subrounded fine gravel							
70 / 35	15		32	Poorly-graded SAND with silt (SP-SM): dense, brown, moist, medium-to coarse-grained, low plasticity							
65				End of Boring at 36'							

BORING DEPTH: 36.0 ft
BACKFILL: Grout
DEPTH TO WATER: 18.0 ft
COMPLETION DATE: February 14, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-112
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	ERT
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	9/7/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 2" AC/4" AB AGGREGATE BASE, 4 inches	Firm								
CLAY (CL), brown, silty, sandy (fine- to medium-grained), damp (with silt, some sand (fine- to medium-grained below 2½ feet)	Stiff				13	23	96		LL=37, PI=21, Passing #200 Sieve = 77%
					9				
			5		26	21	99		
(trace gravel (subangular, fine) below 8½ feet)			10		14	19	109	2.3	
SAND (SC), motled reddish-brown to borwn, fine- to coares-grained sand, with gravel (fine and coarse, subangular to angular), some clay, some interbedded gravel, damp	Medium Dense		15		40	14	120	1.0	
(some clay, moist to wet below 18½ feet)			20		31	17	115		Triaxial Test, Deviator Stress=0.79 ksf
			25		17				
CLAY (CL), mottled light rust brown and grey, silty trace sand (fine-grained), damp	Very Stiff		30		20				
SAND (SC), light orange brown, fine- to coarse-grained, gravelly (fine and coarse, subangular to angular), with clay, moist to wet	Medium Dense		35		31				
					48				

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL WESTERN EXPANSION
Oakland, California

PROJECT NO.	DATE	BORING NO.
17979-CA	September 2000	EB-1

File Name: G:\ENGINEER\GINTW\PROJECTS\17979.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	ERT
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	9/7/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SAND (SC), Continued,	Medium Dense								
	Dense		45		53				

Bottom of Boring = 45 Feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of penetration resistance values, see Appendix A.
3. Ground water level was measured at 22½ feet at the time of drilling.
4. The boring was backfilled with neat cement in accordance with Alameda County Public Works Department requirements.
5. LL = Liquid Limit, PI = Plasticity Index

425 Roland Way
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EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL WESTERN EXPANSION
Oakland, California

PROJECT NO.	DATE	BORING NO.
17979-CA	September 2000	EB-1

File Name: G:\ENGINEER\GINTW\PROJECTS\17979.GPJ Report Template: FUGRO Output Date: 3/14/02

2008 REPORT ID: PB-1 (02)

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/12" AB									
FILL: CLAY, (CL/CH), dark brown, some gravel (subangular), trace sand, damp	Very Stiff				47				
					12				
FILL: SAND (SP), light brown, damp, (trench backfill)	Medium Dense		5		21				
CLAY (CL), brown, silty, some gravel (subangular to subrounded), trace sand (fine- grained), damp	Stiff				17				Passing #200 Sieve = 58%
(grading mottled brown and gray, some sand (fine- to coarse- grained), no gravel at 14 feet)			10		26	26	98	1.9	
			15		22	25	102		Passing #200 Sieve = 68%, LL=33, PI=19
SAND (SC), brown, fine- to coarse-grained, with gravel (subrounded to subangular), some clay, wet	Medium Dense		20		35	17			Passing #4 Sieve = 66%, #200 Sieve = 18%
CLAY (CL), reddish-brown mottled gray, silty, moist	Very Stiff		25						
(interbed of gravel between 27 and 28 feet					45	23	106	14.8	Passing #200 Sieve = 89% LL=43, PI=27

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EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-2

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), moist	Very Stiff								
(grading trace gravel (coarse, subrounded) at 34½ feet)			35		37				
SAND (SC), brown, fine- to coarse-grained, with gravel (coarse, subangular to subrounded), some clay and silt, wet	Dense								Passing #4 Sieve = 62%, #200 Sieve = 18%
CLAY (CL), brown mottled gray, sandy (fine- to coarse- grained), some gravel (coarse, subangular to subrounded), trace silt, damp	Hard		40		54	16			Passing #200 Sieve = 57%, LL=39, PI=23
SAND (SC), brown, fine- to coarse-grained, some clay and silt, trace gravel (fine to coarse, subangular to subrounded), moist	Dense		45		52	21			Passing #200 Sieve = 26%
CLAY (CL), brown, silty, some sand (fine- to coarse- grained), damp	Hard		50		61	25	102	18.3	

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid Limit, PI=Plasticity Index

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Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-2

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

2008 REPORT ID: PB-2 (02)

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/12" AB underlain by geosynthetic									
CLAY (CL), brown, with sand (fine-grained), some gravel (fine, weak subangular sandstone fragments), trace silt, damp	Very Stiff				47				
					24				
			5		54	22	102		
(grading gravelly at 8½ feet)	Stiff				19				
			10						
SAND (SC), brown, fine- to coarse-grained, with gravel (coarse, subangular to subrounded), some clay and silt, damp	Medium Dense				43				Passing #4 Sieve = 63%, #200 Sieve = 20%
			15						
(grading gray-brown, fine- grained, clayey, some silt, moist to wet at 18 feet)					26	24	101	1.5	Passing #200 Sieve = 48%, LL=28, PI=7
			20						
(grading reddish-brown, fine- to coarse-grained, some gravel (fine to coarse, subrounded to subangular), some clay, wet					45	20			Passing #4 Sieve = 71%, #200 Sieve = 15%
			25						Passing #200 Sieve = 75%, LL=38, PI=24
CLAY (CL), brown mottled gray, silty, some sand (fine- to coarse- grained), damp	Very Stiff				48	21	111	9.7	

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EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-3

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

2008 REPORT ID: PB-3 (02)

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), brown mottled gray, silty, some sand (fine- to coarse- grained), damp	Very Stiff								
GRAVEL (GC), brown, fine to coarse, subrounded to subangular, some sand (fine- to coarse- grained), some clay, wet	Dense		35		58	13			Passing #4 Sieve = 45%, #200 Sieve = 16%
SAND (SC), brown, fine- to coarse-grained, silty, trace gravel (fine to coarse, subrounded to subangular), moist	Dense		40		52	22			Passing #4 Sieve = 97%, #200 Sieve = 41%, LL=46, PI=28
CLAY (CL), rust brown, sandy (fine- to coarse- grained), some silt, wet	Very Stiff		45		43	21	112	11.6	Passing #200 Sieve = 54%, LL=44, PI=28
(grading silty, with sand (fine- grained), damp to moist at 48½ feet)	Hard		50		51	22	108		Passing #200 Sieve = 63%, LL=36, PI=19

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid Limit, PI=Plasticity Index

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EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-3

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

2008 REPORT ID: PB-3 (02)

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
PAVEMENT SECTION: 4" AC/18" AB									
CLAY (CL/CH), black, with silt, trace gravel (subrounded), damp	Very Stiff				30				
					23				
GRAVEL (GC), brown, coarse, subrounded to subangular, clayey, with sand (fine- grained), damp	Dense		5		31				
CLAY (CL), brown, silty, some gravel (fine to coarse, subrounded to subangular), trace sand (fine- grained), damp	Stiff				25	18	109		
			10						
GRAVEL (GW), brown, fine to coarse, subrounded to subangular, sandy (fine- to coarse- grained), damp	Medium Dense				26				
			15						
SAND (SM), brown, fine- to coarse-grained, with gravel (fine to coarse, subrounded to subangular), some silt, moist	Dense				43	▼			Passing #4 Sieve = 61%, #200 Sieve = 17%
			20						
(grading clayey sand at 23 feet)	Medium Dense					▽ 23			Passing #4 Sieve = 66%, #200 Sieve = 15% at 23½ ft
(lens of sandy clay at 24½ feet)					35	30			Passing #200 Sieve = 68%, LL=38, PI=22 at 24½ ft
(grading sandy gravel (fine- grained), moist at 25 feet)			25						
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), damp	Very Stiff				38	23	106	10.2	Passing #200 Sieve = 94%, LL=43, PI=28

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EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-4

File Name: G:\ENGINEER\GINTWPROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), damp	Very Stiff								
	Hard				40				
SAND (SC), brown, fine- to coarse-grained, some gravel (fine, subangular), trace silt, wet	Dense		35						
(grading some silt and clay at 39 feet)			40		54	16			Passing #4 Sieve = 71%, #200 Sieve = 27%, LL=40, PI=23
					59				Passing #4 Sieve = 75%, #200 Sieve = 22%
CLAY (CL), brown, with sand (coarse-grained), damp	Hard		45						
			50		31				

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lbdown-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid limt, PI=Plasticity Index

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EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-4

File Name: G:\ENGINEER\GINTWPROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

2008 REPORT ID: PB-4 (02)

DRILL RIG	CME 75 HSA	SURFACE ELEVATION	96 feet	LOGGED BY	MHL
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	11/24/97

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
PAVEMENT: 2-1/2-inched A.C. over 9-1/2-inches A.B.					50/5"				
FILL: SAND (SW), yellowish-brown, fine-to coarse-grained, some gravel (fine to coarse, angular), trace to some clay, dry	Dense								
	Dense				44	21	94		LL=37%, PI=21%, Passing #200 Sieve=72%
FILL: SAND (SC), yellowish-brown, fine-to-coarse-grained, some clay, trace gravel (fine to coarse, angular), dry-damp	Very Dense		5		51				
FILL: CLAY (CL), dark brown, silty, some sand (fine grained), trace gravel (fine, angular), damp									
CLAY (CL), brown to dark brown, silty, some sand (fine grained), damp	Hard				50/5"	21	107		
(grey with yellowish-red iron staining, with sand (fine-to medium-grained), trace gravel (fine to medium, rounded) below 12 feet)			10		50/6"	18	111		
			15						
SILT (ML), light grey with yellowish-red iron staining, some sand (fine grained), trace pockets of wet clay, damp	Hard				75	20	108		
					37				
			20						

Bottom of Boring = 20 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. The boring was backfilled with native soil cuttings immediately upon completion.

HARZA
Consulting Engineers & Scientists

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL HELISTOP
Oakland, California

PROJECT NO.	DATE	BORING NO.
L998G	December 1997	EB-1

DRILL RIG	gCME 75 HSA	SURFACE ELEVATION	96 feet	LOGGED BY	MHL
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	11/24/97

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
PAVEMENT: 2-inches A.C. over 8-inches A.B.									
FILL: SAND (SC), brown-dark brown, fine-grained, silty, some clay, trace gravel (fine to medium, subangular), trace wood and brick debris, damp	Med. Dense								LL=31%, PI=19%, Passing #200 Sieve=35%
CLAY (CL), dark brown with yellowish-red iron staining, silty, trace to some sand (fine grained), damp	Hard		5		29				
					37				
	Very Stiff				52	21	106		
SILT (ML), grey with yellowish-red iron staining, some sand (fine grained), some clay, damp	Very Stiff		10						
SAND (SC), yellowish-brown, fine-to coarse-grained, some clay, some gravel (fine to medium, subrouned), damp-moist (yellowish-brown with red iron and black staining below 13-1/2 feet)	Med. Dense				22	17	108		
			15		25				

Bottom of Boring = 16-1/2 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. The boring was backfilled with native soil cuttings immediately upon completion.

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EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL HELISTOP
Oakland, California

PROJECT NO.	DATE	BORING NO.
L998G	December 1997	EB-2

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	18 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), (continued) (sandy at 34-1/2 to 36 feet)	Hard				93/11"				
(sand appears cemented below 40 feet)			40						
			45						
			50		89				
Bottom of Boring = 51 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 24 feet at time of drilling. After completion of drilling, ground water was measured at 18 feet. 4. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot. 5. PI = Plasticity Index, LL = Liquid Limit.									

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-3

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	20 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
6 inches asphaltic concrete, 7 inches aggregate baserock									
CLAY (CL), dark brown to black, silty, damp	Very Stiff		5		30				PP=9.0 ksf, TV=4.3 ksf
(brown, plasticity decreases below 9-1/2 feet)			10		36				PP=9.0 ksf, TV=4.5+ ksf
CLAY (CL), olive gray with rust mottling, silty, moist	Very Stiff		15		24 15	23	102		PP=8.0 ksf, TV=4.3 ksf PP=8.0 ksf, TV=2.5 ksf
(trace sand (fine- to coarse-grained) and gravel (fine, subrounded) at 19-1/2 to 20 feet)			20		32	▼			
SAND (SC), brown, fine- to coarse-grained, with gravel (fine, subangular to subrounded), trace clay, wet	Medium Dense				26				
(some clay at 28 to 28-1/2 feet)			25						
CLAY (CL), rust brown, with gray mottling, some sand (fine- to coarse-grained), moist	Hard		30		59				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-4

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	20 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL) (continued)	Hard								
(cemented at 38 to 30-1/2 feet)			40		65				
			45						
(sand not apparent at 48-1/2 to 50 feet)			50		97/11"				
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 22 feet at time of drilling. After completion of drilling, ground water was measured at 20 feet. 4. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot.									

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-4

2008 REPORT ID: PB-4 (94)

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	18 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
4 inches of asphaltic concrete, 5 inches of aggregate baserock									
SILT (ML), dark brown, clayey, some sand (fine-grained), trace rootlets, dry	Stiff		5		21	16	99		PI=19%, LL=36%, Passing #200 Sieve=72%
(coarse-grained sand apparent, trace gravel (fine, subangular) at 9-1/2 to 11 feet)	Very Stiff		10		39 35				PP=9.0+ ksf, TV=4.5+ ksf
CLAY (CL), olive gray with rust mottling, silty, moist	Very Stiff		15		32	22	103		PP=6.5 ksf, TV=2.8 ksf
	Stiff		20		23				PP=3.0 ksf, TV=2.3 ksf
SAND (SC), brown, fine- to coarse-grained, with clay, and gravel (fine, subangular), wet	Medium Dense		25		30 28				
CLAY (CL), rust brown to brown, with sand (fine- to coarse-grained), trace gravel (fine, angular to subrounded), moist	Hard		30		38				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-3

2008 REPORT ID: PB-3 (94)

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	101 Feet	LOGGED BY	CDT/MJ
DEPTH TO GROUND WATER	22 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/3/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), dark brown, silty, trace rootlets	Stiff								
SAND (SM), brown, fine- to coarse-grained, with gravel (fine, subangular to subrounded), trace silt, damp	Loose		5		15				
	Medium Dense		10		29				
CLAY (CL), brown, with sand (fine- to coarse-grained), trace silt, moist	Firm		15		5				
	Stiff				14	19	107	1.1	TV=2.5 ksf
(tan-gray with rust mottling, damp at 18-1/2 to 20 feet)	Stiff to Very Stiff		20		26				PP=4.0 ksf, TV=3.0 ksf
(moist to wet at 23-1/2 to 25 feet)	Stiff		25		17				
SAND (SM), tan-gray, fine- to medium-grained, some silt, wet	Medium Dense		30		13				Passing #200 Sieve=25%
(trace gravel (fine, subangular) at 33-1/2 to 35 feet)	Dense				45				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-1

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	101 Feet	LOGGED BY	CDT/MJ
DEPTH TO GROUND WATER	22 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/3/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
SAND (SM) (continued)	Dense								
CLAY (CL), gray, silty, damp to moist	Stiff		40		14				
(with sand (fine- to medium-grained at 48-1/2 to 50 feet)	Hard		45		58				PP=9.0 ksf, TV=5.0 ksf
			50						

Bottom of Boring = 50 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see first page Appendix A.
3. Ground water level was measured at 27 feet at time of drilling.
4. An observation well was installed in this boring to periodically measure the ground water depth.
5. Ground water was measured at 22 feet on 9/22/92.
6. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot.

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-1

2008 REPORT ID: PB-1 (94)OW

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	97 Feet	LOGGED BY	MJ
DEPTH TO GROUND WATER	16 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/4/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
8 inches asphaltic concrete, 4 inches aggregate baserock									
CLAY (CL), dark brown, silty, trace gravel (fine, subangular), damp	Very Stiff								
(red-brown with gray mottling, trace sand (fine-grained) at 4-1/2 to 6 feet)			5		24				
			10		19				
SAND (SC), red-brown, fine- to coarse-grained, with gravel (fine, subangular), moist	Medium Dense								
(gravelly, wet at 13 to 14-1/2 feet)	Dense		15		35				
(fine-grained sand layer at 17-1/2 feet)	Medium Dense				20				Passing #200 Sieve = 36%
			20		24				
			25						
CLAY (CL), tan-brown with gray mottling, trace sand (fine-grained), moist	Very Stiff				29				
			30						
(with sand at 28 to 29-1/2 feet)	Hard				55				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-6

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	97 Feet	LOGGED BY	MJ
DEPTH TO GROUND WATER	16 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/4/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL) (continued)	Hard								
SAND (SC), red-brown, fine- to coarse-grained, with clay, trace gravel (fine, angular), damp	Very Dense		40		84				Passing #200 Sieve = 21%
			45						
	Dense		50		74				
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 17-1/2 feet at time of drilling. 4. An observation well was installed in this boring to periodically measure the ground water depth. 5. Ground water was measured at 16 feet on 9/22/92.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-6

2008 REPORT ID: PB-6 (94)OW

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Enc.	BORING DIAMETER	5-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
3" Asphaltic Concrete									
FILL: CLAY (CL), brown mottled rust and blue, silty, with sand (fine- to coarse-grained), some gravel (fine- to coarse-grained), trace organics	Firm				14*				PP = 1.5 tsf
	Stiff				13				
(less gravel and organics)			5						
(light brown, no organics)			10		25*	18	108	2.9	PP = 3.5 tsf
CLAY (CL), light brown mottled rust, with sand (fine- to coarse-grained), some silt, some gravel (fine-grained)	Stiff		15		16*	18	109	1.5	PP = 1.7 tsf
SAND (SM), light brown mottled gray, fine- to coarse-grained, gravelly (fine-grained), some silt and clay	Medium dense		20		29*				
Bottom of Boring = 20-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not encountered at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	1

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2" Asphaltic Concrete over 2" Gravel									
FILL: CLAY (CL), light brown mottled blue-green and black, silty, with sand (fine- to coarse-grained), trace gravel (fine-grained), trace organics	Firm				15*	18	102		PP = 1.5 tsf
	Stiff		5		10				
(dark brown mottled rust and gray, some gravel (fine-grained))									
(brown mottled rust and gray, with sand (fine- to coarse-grained), some gravel (fine- to coarse-grained))	Firm		10		9*	16	107	0.9	PP = 1.0 tsf
GRAVEL (GM), gray-brown, fine-grained, some sand (fine- to coarse-grained), trace silt and clay	Medium dense		15		21*				
CLAY (CL), tan and rust, some silt, trace sand (fine-grained)	Firm		20		7				
(some sand (fine- to coarse-grained), trace gravel (fine-grained))			25		17*	23	102	2.5	PP = 1.2 tsf
(silty, some gravel (fine-grained))	Stiff		30		27*				PP = 1.7 tsf
Bottom of Boring = 30-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not established at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	2

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Enc.	BORING DIAMETER	5-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2-1/2" Asphaltic Concrete over 4-1/2" Gravel									
FILL: CLAY (CL), brown, silty, trace sand (fine- to coarse-grained), trace gravel (fine-grained)	Firm				15*	20	90		PP = 2.0 tsf
(some sand (fine- to coarse-grained), more gravel)	Stiff		5		10				
(brown mottled rust and gray, some gravel (fine-grained))			10		38*	17	107	3.4	PP = 4.0 tsf
CLAY (CL), brown, silty, trace sand (fine- to medium-grained)	Stiff				248				
GRAVEL (GM), brown mottled rust and gray, fine- to coarse-grained, some sand (fine- to coarse-grained), trace silt and clay	Medium dense		15						
			20		37*				
Bottom of Boring = 20-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not encountered at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	3

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2" Asphaltic Concrete									
FILL: CLAY (CL), black and blue-green, silty, some sand (fine- to coarse-grained), trace gravel (fine-grained)	Stiff				23*	19	102		PP = 3.2 tsf
(brown mottled rust and gray, some gravel (fine- to coarse-grained), trace organics)			5		9				
(with sand (fine- to coarse-grained), trace gravel (fine- to coarse- grained), no organics)			10		25*	13	110	2.0	PP > 4.5 tsf
(sandy (fine- to coarse-grained))			15		8				
GRAVEL (GM), gray, fine-grained, trace sand (fine- to coarse-grained), trace silt and clay	Loose		20		6				
SILT (ML), tan mottled rust, sandy (fine-grained), trace clay, trace sand (medium- to coarse-grained)	Stiff		25		19*	28	95	2.8	PP = 1.5 tsf
(less sand)			30						

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	4

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SILT (ML), tan mottled rust, with sand (fine-grained), trace clay, trace sand (medium- to coarse-grained)	Stiff								
	Firm		35		14*	24	100		PP = 1.2 tsf
(same as above)			40						
(same as above)	Stiff								
CLAY (CL), orange-brown mottled black and rust, some sand (fine-grained), trace sand (medium- to coarse-grained)	Hard		45						
			50		62*				PP > 4.5 tsf
Bottom of Boring = 50-1/2 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A. 3. The groundwater level was not established at the time of drilling. 4. PP = Pocket Penetrometer 5. The boring was backfilled with cuttings and capped with asphaltic concrete.									

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	4

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION		LOGGED BY	MD/PW
DEPTH TO GROUNDWATER	16.0 Feet	BORING DIAMETER	6-inch	DATE DRILLED	6/6/90

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
Asphalt Concrete (2") **Baserock (6")**					29*	25	95		
CLAY (CL), black, some silt, trace sand (fine- to coarse-grained)	very stiff		5						
(grading orange-tan) (lens, sandy, fine-grained)			10		20				
(mottled grey and orange)			15		19	27			
SAND (SC), orange, fine- to coarse-grained, some silt and clay, trace gravel (fine-grained)	medium dense		20		44*				-#200 = 26%
CLAY (CL), mottled brown, black, and orange, some silt and sand (fine- to coarse-grained)	very stiff		25		43*	22	105		Su = 3.6ksf
(grading light grey)			30						
SAND (SC), fine- to coarse-grained, mottled grey, brown, red and orange, some silt and clay	dense		35		38				-#200 = 24%
CLAY (CL), sandy (fine- to coarse-grained), some silt	hard		40						
			45		45				

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EXPLORATORY BORING LOG

OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	BORING NO
K339-9	November 1990	1

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION		LOGGED BY	MD/PW
DEPTH TO GROUNDWATER	16.0 Feet	BORING DIAMETER	6-inch	DATE DRILLED	6/6/90

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), sandy (fine- to coarse-grained), some silt	hard		55						
					80*	18	114	17.6	
			60						
			65						
					50/6*	15	120	19.2	
			70						
			75						
					44				
Bottom of Boring = 79 1/2 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*), see first page Appendix A. 3. Ground water level was encountered at 16 feet at time of drilling.									

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EXPLORATORY BORING LOG

OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	BORING NO
K339-9	November 1990	1

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION		LOGGED BY	MD/PW
DEPTH TO GROUNDWATER	8.0 Feet	BORING DIAMETER	6-inch	DATE DRILLED	6/6/90

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
Gravelly sand (6")									
CLAY (CL), black, some silt and trace sand (fine- to coarse-grained)	stiff-very stiff				28*				
(tan) (grading silty, some sand, fine-grained)			5		32*	22	106		
			10						
(grading sandy, fine- to coarse-grained, trace silt, trace gravel, fine-grained)					16				-#200 = 60%
			15						
SAND (SM), fine-grained, mottled orange, red, black and brown, silty	medium dense				26				-#200 = 44%
	dense		20		87*				
(grading some silt and clay)			25		36				-#200 = 27%
			30						
	very dense		35		80				
Bottom of Boring = 39 1/2 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*), see first page Appendix A. 3. Ground water level was encountered at									

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EXPLORATORY BORING LOG

OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	BORING NO
K339-9	November 1990	2

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION		LOGGED BY	MD/PW
DEPTH TO GROUNDWATER	8.0 Feet	BORING DIAMETER	6-inch	DATE DRILLED	6/6/90

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
8 feet at time of drilling.									

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EXPLORATORY BORING LOG

OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	BORING NO
K339-9	November 1990	2

DRILL RIG	HSA	SURFACE ELEVATION	—	LOGGED BY	KF
DEPTH TO GROUNDWATER	37 ft.	BORING DIAMETER	8 -inch	DATE DRILLED	6/11/90

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
7" A.C.									
CLAY (CL), dark brown, silty, trace sand (f-gr)	stiff				17*	21	89		
(grading brown-grey)	hard		5		40*	16	112	11.8	
(grading some sand (f-crs. gr) and fine-gr. gravel)	very stiff				21				
	stiff		10		23*	19	107	3.6	
(grading no gravel)									
	hard		15		33*	19	109	13.4	
SAND (SM), brown-rust, fine-coarse-grained, gravelly (f-gr), some silt and clay	medium dense		20		37*				-#200 - 18
CLAY (CL-SC) brown-rust, sandy (f-crs-gr), trace gravel (f-gr), silty	hard		25		36				
CLAY (CL), gray-brown, silty, trace sand (f-gr)	very stiff-hard		30		60*	19	112		S_u = 3.8 ksf
(grading some fine-to medium-grained sand)			35						
	hard				64*	21	111		S_u = 6.2 ksf

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EXPLORATORY BORING LOG

Proposed Medical Office Building and Parking Structure
Children's Hospital
Oakland, CA

PROJECT NO.	DATE	BORING NO
K339-9		4

DRILL RIG	HSA	SURFACE ELEVATION	—	LOGGED BY	KF
DEPTH TO GROUNDWATER	37 ft.	BORING DIAMETER	8 -inch	DATE DRILLED	6/11/90

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (continued)	hard		45		64*				
SAND (SC), rust-brown, fine to coarse-grained, gravelly (f-g), some silt and clay	dense-very dense		50		70*	14	123		
CLAY (CL), rust-brown, sandy (f-crs-gr), silty	hard		55						
			60		70*	19	113		S_u = 6.1 ksf
BOB = 60 ft.									
NOTES: 1) — 2) * 3) ATOD, 37 ft.			65						
			30						
			35						

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EXPLORATORY BORING LOG

~~Regional Children's Hospital Expansion~~
Oakland, CA

PROJECT NO.	DATE	BORING NO
K339-1		4

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 97 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	18' (See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 4" Baserock				1					
SAND, (fine-grained), clayey, with silt, traces of gravel (fine-grained)	dark grey	medium dense	SC-CL	2		30	17		
Liquid Limit = 30%									
Plasticity Index = 12%		loose		3		13*			
Passing #200 Sieve = 49%									
SILT, sandy, (fine-grained), trace of gravel, (fine grained)	tan	stiff	ML	4					
Passing #200 Sieve = 54%				5		9	24		
CLAY, silty, some sand, (fine to coarse-grained)	grey	stiff	CL	6					
				7					
				8					
(gravel fine grained more sand)				9					
Liquid Limit = 33%	mottled	very	CL	10		42*	14	114	10.5
Platicity Index = 33%	with	stiff							
Passing #200 Sieve = 51%	tan								
				11					
				12					
				13					
(grading more gravel, fine-grianed)	light grey			14		34*	24	101	8.6
				15					
SAND, (fine-to coarse-grained) silty, trace of gravel, (fine grained), some clay	light grey mottled with tan	medium dense	SM	16					
				17					
				18			▽		
				19					
Passing #200 Sieve = 51%			SM-ML	20		47*			

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EXPLORATORY BORING LOG

NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	1

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION ± 97 Feet | LOGGED BY LG

DEPTH TO GROUNDWATER 18' (See Note | BORING DIAMETER 8 Inches | DATE DRILLED 12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse-grained), some clay and gravel (fine-grained)	light grey	medium	SC	21		47*			
GRAVEL, (fine - grained), sandy, (fine-to coarse-grained), some clay and silt	brown	medium dense	GC	22 23 24					
CLAY, silty, sandy (fine-to coarse-grained), with gravel (fine grained)	grey mottled with tan	stiff	CL	25		30*	20		
GRAVEL (fine-grained), sandy, (fine-to coarse-grained), some silt and clay	tan	very dense	GC	26 27 28					
Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see frist page Appendix A. 3. Groundwater level was measured at a depth of 20 feet at time of drilling. 3½ hours later the groundwater level was measured at 18 feet.				29 30		94*/11	16	119	12.0
				31 32 33 34					
(sandy clay lense)				35 36 37 38 39		85			
				40		72	18		

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	1

2008 REPORT ID: PB-1 (87)

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION ± 94 Feet | LOGGED BY LG

DEPTH TO GROUNDWATER 19½' (See Note 3) | BORING DIAMETER 8 Inches | DATE DRILLED 12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 4" Baserock				1					
CLAY, silty gravelly (fine-grained), some sand (fine-to coarse-grained)	dark grey	very stiff	CL	2		57*	19	114	
GRAVEL (fine-grained) sandy, (fine-to coarse-grained), with clay Passing #200 Sieve = 17%	grey	medium dense	GC	3 4		28*			
CLAY, silty some sand (fine-grained) traces of gravel (occasional gravel lenses)	dark grey	very stiff	CL	5 6 7 8		17			
(grading gravelly, fine-grained)	mottled with tan			9 10 11 12 13 14		58*	17		
(grading with traces of gravel fine grained)				15 16 17		47*	21		
SAND, (fine-to coarse-grained), some silt, some clay	brown	medium dense	SC-SM	18 19					
(occasional clay lense)				20		28*	22	101	

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, Calfiornia

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	2

2008 REPORT ID: PB-2 (87)

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION ± 94 Feet | LOGGED BY LG

DEPTH TO GROUNDWATER 19½'(See Note 3) | BORING DIAMETER 8 Inches | DATE DRILLED 12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse grained), gravelly, (fine-grained), with silt, some clay (occasional clay lense) Passing #200 Sieve = 38%	brown	medium dense	SC-SM	21–25		28			
CLAY, sandy, (fine to coarse-grained), with gravel (fine grained), silty (occasional sand or gravel lense)	light grey	firm	CL-SC	25–30		16*	22	104	0.5
(grading without gravel, less sand) Liquid Limit = 30% Plasticity Index = 16% Passing #200 Sieve = 79%		stiff	CL	30–35		20*	27	96	0.6
SAND, (fine to coarse-grained) some clay and silt, trace of gravel (fine-grained)	brown	dense	SC	35–40		39*	29		

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO. K339-7 | DATE December 1987 | BORING NO. 2

2008 REPORT ID: PB-2 (87)

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION ± 94 Feet | LOGGED BY LG

DEPTH TO GROUNDWATER 19½'(See Note 3) | BORING DIAMETER 8 Inches | DATE DRILLED 12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse grained) some silt, trace of gravel (fine-grained) (occasional gravel lense)	brown	medium dense	SC	41–45		39*			
(grading gravelly)						68			
CLAY, silty, with sand (fine to coarse-grained) (trace of gravel)	light grey	hard	CL	46–49					
CLAY, silty, sandy (medium-grained)	rust	hard	CL	50		58*/6"	21		
Bottom of Boring = 51 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3, Groundwater level was measured at a depth of 19½ feet at time of drilling.				51–60					

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO. K339-7 | DATE December 1987 | BORING NO. 2

2008 REPORT ID: PB-2 (87)

DRILL RIG Continuous Flight Auger | SURFACE ELEVATION -- | LOGGED BY JN/KS

DEPTH TO GROUNDWATER 15' (see note 3) | BORING DIAMETER 6 Inches | DATE DRILLED 6/4/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT, %	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
6" Gravel Surface (Parking Area)									
CLAY, silty, trace of organics	black	stiff	CL	1		26*			
						26*	22	102	6.1
(grading with silt, without organics)	brown			5		26*	21	102	4.1
(grading some sand fine-coarse grained and gravel) Liquid Limit = 31% Plasticity Index = 11% Passing #200 Sieve = 85%		firm		10		13*	20	100	
CLAY, very silty	mottled orange & grey	stiff	CL-ML	15		27*	25 ▽	98	10.6
(grading sandy fine grained)									
SAND (fine-coarse grained) with silt and clay, trace of gravel	orange mottled with grey & black	medium dense	SM-SC	20		27	24		
(grading gravelly fine grained)									
Notes: See Boring 4 for Notes 1 and 2. 3. Groundwater level was measured at 15 feet at time of drilling.									
SILT, with sand (fine grained), trace of clay	orange mottled with grey	very stiff	ML	25		19	22		
CLAY, sandy (fine-coarse grained) gravelly	orange	very stiff	CL	30		30	25		
SAND, clayey, silty Passing #200 Sieve = 56%	orange mottled with grey	medium dense	SM-SC	35		17	19		
SILT, sandy (fine-coarse grained) some gravel	yellow	hard	ML	40		45/12"	24		
Bottom of Boring = 40½ Feet									

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EXPLORATORY BORING LOG

PROPOSED MEDICAL OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	BORING NO.
K928-1	June 1986	5

2008 REPORT ID: PB-5 (86a)

DRILL RIG Continuous Flight Auger | SURFACE ELEVATION -- | LOGGED BY JN/KS

DEPTH TO GROUNDWATER 15' (see note 3) | BORING DIAMETER 6 Inches | DATE DRILLED 6/4/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT, %	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, trace of sand (coarse grained) and organics	black brown	stiff	CL	1		23*	14		
(sand grading fine)						30*	18	96	3.4
Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3. Groundwater level was measured at 15 feet at time of drilling.		very stiff		5		21	17		
GRAVEL, sandy, clayey, some silt Passing #200 Sieve = 30%	brown orange	medium dense	GC-GM	10		21*	18		
SILT, clayey	orange mottled with grey	very stiff	ML-CL	15		23	▽ 24		
SAND (fine-coarse grained), gravelly with clay, some silt	orange yellow	medium dense	SM-SC	20		18	22		
(grading fine sand, silty) (less clay)	mottled with grey		SM						
SAND (coarse grained), some silt and clay, trace of gravel	orange & dark grey	medium dense	SM-SC	25		19	20		
CLAY, very silty, trace of sand (fine grained)	orange mottled with grey	hard	CL	30		36	21		
GRAVEL, sandy (fine-coarse grained), some clay and silt Passing #200 Sieve = 20%	orange	very dense	GM-GC	35		52	16		
Bottom of Boring = 40½ Feet				40		46	20		

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EXPLORATORY BORING LOG

PROPOSED MEDICAL OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	BORING NO.
K928-1	June 1986	6

2008 REPORT ID: PB-6 (86a)

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION --	LOGGED BY JN/KS
DEPTH TO GROUNDWATER 12' (see note 3)	BORING DIAMETER 6 Inches	DATE DRILLED 6/4/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
6" Gravel Surface (Parking Area)									
CLAY, silty, trace of organics (grading without organics, trace of sand coarse grained)	black brown	very stiff	CL	1		26*	24		
						27*	20	102	7.9
(sand grading fine)		hard		5		29*	19	108	9.0
CLAY, silty, trace of sand	brown	stiff	CL						
(interbedded gravel 2' thick) (grading some sand fine-coarse grained)	light brown light						20	106	3.1
CLAY, very silty, trace of sand (fine-coarse grained)	orange mottled with grey	stiff	CL	10			▽		
SAND (fine-coarse grained), with clay and silt, some gravel	orange yellow	dense	SC	15		53*			
(sand grading fine-medium grained, without gravel) (interbedded gravel layer) (interbedded sand without gravel) Passing #200 Sieve = 27%				20		74*	18		
(interbedded gravel)									
(grading gravelly, some clay and silt)				25		30	20		
(grading slightly more silt)		very dense		30		54	18		
(grading with clay and silt) (interbedded gravel)	orange mottled with grey yellow	dense	SM-SC	35		44	16		
CLAY, very silty, some sand (fine grained), trace of gravel	orange mottled with grey	hard	CL ML	40		62	22		

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EXPLORATORY BORING LOG

PROPOSED MEDICAL OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	Previous Boring No.
K339-9	November 1990	7

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION --	LOGGED BY JN/KS
DEPTH TO GROUNDWATER 12' (see note 3)	BORING DIAMETER 6 Inches	DATE DRILLED 6/4/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, very silty, some sand (fine coarse grained), trace of gravel (continued)	orange mottled with grey	hard	CL-ML	41					
SAND, gravelly, with silt and clay	orange mottled	very dense	SM-SC	45					
(interbedded layers of clayey silt, trace of sand fine grained)				50		78			
				55					
				60					
Bottom of Boring = 60½ Feet									
Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3. Groundwater level was measured at 12 feet at time of drilling.				65					
				70					
				75					
				90					

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EXPLORATORY BORING LOG

PROPOSED MEDICAL OFFICE BUILDING AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	Previous Boring No.
K339-9	November 1990	7

2008 REPORT ID: PB-7 (86a)

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	--	LOGGED BY	DYR
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
6" sandy GRAVEL									
CLAY, silty, trace of sand and organics Liquid Limit = 37% Plasticity Index = 14% Passing #200 Sieve = 72% (FILL)	brown black	stiff	CL	1-3		12 18*	24 23	 102	 4.3
CLAY, silty, trace of sand and gravels	dark brown mottled with grey	stiff	CL	4-5		30*	23	103	6.3
CLAY, silty, trace of sand	light brown grey	stiff	CL	6-8					
CLAY, very silty, with sand	light brown	stiff	CL	9-10		22*	19		
CLAY, silty (interbedded lenses of sandy clay with gravels)	mottled tan, grey	stiff	CL	11-15		32*	22	106	12.7
CLAY, silty, trace of gravels with sand (fine grained)	olive grey and orange	stiff	CL-CH	16-19		15	23		
CLAY, silty, some sand	light brown	stiff	CL	20					

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EXPLORATORY BORING LOG
FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND
Oakland, California

PROJECT NO.	DATE	BORING NO.
K876-3	April 1986	1

2008 REPORT ID: PB-1 (86b)

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	--	LOGGED BY	DYR
DEPTH TO GROUNDWATER	Not Established	BORING DIAMETER	6 Inches	DATE DRILLED	3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, some sand (continued)	light brown	stiff	CL	21-22					
SAND (fine - coarse grained), with clay and silt, some gravel	orange tan	dense	SC	23-24					
Bottom of Boring = 25 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A.				25-40					

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EXPLORATORY BORING LOG
FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND
Oakland, California

PROJECT NO.	DATE	BORING NO.
K876-3	April 1986	1

2008 REPORT ID: PB-1 (86b)

DRILL RIG Continuous Flight Auger | SURFACE ELEVATION -- | LOGGED BY DYR

DEPTH TO GROUNDWATER 13½' (see note 3) | BORING DIAMETER 6 Inches | DATE DRILLED 3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
7½" sandy GRAVEL									
CLAY, silty with sand, trace of organics, wood	black	stiff	CL	1					
				2		24*	20	108	14.6
(interbedded with gravelly lenses)				3		21*	22	103	5.1
(FILL)				4					
CLAY, silty, trace of sand	brown mottled with grey	stiff	CL	5		24*	23	102	9.0
				6					
				7					
CLAY, silty with sand and some pea gravel	light brown mottled with orange grey	stiff	CL	8					
				9		17*			
(grading with less sand)	light brown			10					
				11					
				12					
		very stiff		13			▽		
				14		42*	11	111	14.4
				15					
				16					
CLAY, silty, trace of gravels and sand	olive grey	stiff	CL	17					
				18					
				19		13	27		
				20					

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EXPLORATORY BORING LOG
FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND
Oakland, California

PROJECT NO.	DATE	BORING NO.
K876-3	April 1986	2

DRILL RIG Continuous Flight Auger | SURFACE ELEVATION -- | LOGGED BY DYR

DEPTH TO GROUNDWATER 13½' (see note 3) | BORING DIAMETER 6 Iches | DATE DRILLED 3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty (continued)	olive grey	stiff	CL	21					
SAND (fine - coarse grained), with clay, silt and gravels	mottled orange tan, grey	medium dense	SC	22					
				23					
				24		28			
				25					
CLAY, sandy, some gravels	orange tan mottled with grey	very stiff	CL	26					
				27					
				28					
				29		27	20		
				30					
				31					
				32					
SAND (fine - coarse grained), with silt and clay, some gravel	orange tan	dense	SM	33					
				34		46			
Bottom of Boring = 35 Feet				35					
				36					
				37					
				38					
				39					
				40					

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A.
3. Groundwater level was measured at time of drilling.

PETER KALDVEER AND ASSOCIATES, INC.
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EXPLORATORY BORING LOG
FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND
Oakland, California

PROJECT NO.	DATE	BORING NO.
K876-3	April 1986	2

2008 REPORT ID: PB-2 (86b)

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	--	LOGGED BY	DYR
DEPTH TO GROUNDWATER	11' (see note 3)	BORING DIAMETER	6 Inches	DATE DRILLED	3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
7½" sandy GRAVEL									
CLAY, silty, with sand, trace of gravels, wood (FILL)	black	stiff	CL	1					
				2		36*	22	100	9.2
CLAY, silty, some sand	dark brown mottled with grey	stiff	CL	3		11	24		
				4					
				5		28*			
				6					
				7					
CLAY, silty, trace of sand (thin lense of well graded SAND with gravel)	light brown mottled with grey	stiff	CL	8					
				9		31*	21	108	9.1
CLAY, very silty, some sand	light brown	stiff	CL	10					
				11			▽		
CLAY, silty, some sand, trace of gravels (grading with more sand)	yellow tan	stiff	CL	12					
				13					
				14		32*			
				15					
				16					
				17					
CLAY, silty, some sand, trace of gravels	olive grey	stiff	CL-CH	18					
				19		16	23		
				20					

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EXPLORATORY BORING LOG
FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND
Oakland, California

PROJECT NO.	DATE	BORING NO.
K876-3	April 1986	3

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	--	LOGGED BY	DYR
DEPTH TO GROUNDWATER	11' (see note 3)	BORING DIAMETER	6 Inches	DATE DRILLED	3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, some sand, trace of gravels (continued)	olive grey	stiff	CL-CH	21					
SAND (fine - coarse grained), with silt and clayey, some gravels	orange tan	dense	SC	22					
				23					
				24		20/6"			
Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3. Groundwater level was measured at time of drilling.				25					
				26					
				27					
				28					
				29		46			
				30					
				31					
				32					
				33					
				34					
CLAY, silty, some sand	orange tan	stiff	CL	35					
Bottom of Boring = 35 Feet				36					
				37					
				38					
				39					
				40					

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Geotechnical Consultants

EXPLORATORY BORING LOG
FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND
Oakland, California

PROJECT NO.	DATE	BORING NO.
K876-3	April 1986	3

2008 REPORT ID: PB-3 (86b)

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	--	LOGGED BY	DYR
DEPTH TO GROUNDWATER	6' (see note 3)	BORING DIAMETER	6 Inches	DATE DRILLED	3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
6½" sandy GRAVEL									
CLAY, silty, some sand, trace of organics Liquid Limit = 39% Plasticity Index = 16% Passing #200 Sieve = 68%	brown black	very stiff stiff	CL	1 2 3		20 27*	24		
CLAY, silty, some sand, trace of organics and gravels	brown grey	stiff	CL	4 5 6		28*	21	106	6.2
CLAY, silty, some sand and trace of pea gravel (thin lense of well graded sand with some gravels)	light brown mottled with orange black	stiff	CL	7 8 9 10		18*	22	106	2.8
(grading with more pea gravels and sand)		very stiff		11 12 13 14 15		57*			
				16 17 18 19 20		29			

PETER KALDVEER AND ASSOCIATES, INC.	EXPLORATORY BORING LOG		
Geotechnical Consultants	FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND Oakland, California		
	PROJECT NO.	DATE	BORING NO.
	K876-3	April 1986	4

2008 REPORT ID: PB-4 (86b)

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	--	LOGGED BY	DYR
DEPTH TO GROUNDWATER	6' (see note 3)	BORING DIAMETER	6 Inches	DATE DRILLED	3/21/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, some sand, trace of pea gravel (continued)	light brown mottled orange	very stiff	CL	21 22					
SAND, clayey, some pea gravel	mottled orange tan and grey	dense	SC	23 24 25		36			
Bottom of Boring = 25 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3. Groundwater level was measured at 10 feet at the time of drilling. Three hours later the groundwater level was measured at 6 feet.				26 27 28 29 30 31 32 33 34 35 36 37 38 39 40					

PETER KALDVEER AND ASSOCIATES, INC.	EXPLORATORY BORING LOG		
Geotechnical Consultants	FAMILY CENTER-CHILDRENS HOSPITAL OF OAKLAND Oakland, California		
	PROJECT NO.	DATE	BORING NO.
	K876-3	April 1986	4

2008 REPORT ID: PB-4 (86b)

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	--	LOGGED BY	F.M.
DEPTH TO GROUNDWATER	23' (SeeNote3)	BORING DIAMETER	8 Inches	DATE DRILLED	6/22/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 7" BASEROCK				1					
CLAY, silty with some sand (fine- to coarse grained)	mottled brown-black	stiff	CL	2		10	12		
CLAY, silty, sandy (fine-grained) with chunks of masonry pipe	mottled browns-blacks	stiff	CL-ML	3		10	9		
Dry Density = 104 pcf				4					
				5		20*			>5.0
(Possible Fill)				6					
				7					
CLAY, very sandy (fine to coarse grained) with gravel to ½"	mottled browns-black	hard	CL	8					
				9		78*		1.8	3.5
Liquid Limit = 35%				10					
Plasticity Index = 21%				11					
Passing #200 Sieve = 55%				12					
Dry Density = 115 pcf				13					
				14		36*			
				15					
				16					
				17					
				18					
SAND (fine to coarse grained) with clay and gravel to ½"	mottled brown-grey-black	medium dense	SC	19		23	23		
Passing #200 Sieve = 16%				20					

Peter Kaldveer and Associates

Geotechnical Consultants

EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

2008 REPORT ID: PB-6 (78)

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	--	LOGGED BY	F.M.
DEPTH TO GROUNDWATER	23' (SeeNote3)	BORING DIAMETER	8 Inches	DATE DRILLED	6/21/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (fine to coarse grained) with clay and gravel to ½" (continued)	mottled brown-grey-black	medium dense	SC	21					
				22					
				23			▽		
SILT, clayey with sand (fine-grained)	mottled tan-rust	stiff	ML	24		12	25		
				25					
				26					
CLAY, silty, sandy (fine to medium grained)	blue-green	stiff	CL	27					
				28					
				29		12	19		
				30					
				31					
				32					
				33					
SAND (fine to coarse grained) gravelly to 1", with some clay	mottled brown-grey-black	dense	SC-GC	34		push 500-1000 psf	13		
				35					
				36					
				37					
(increasing clay content)				38					
Passing #200 Sieve = 19%				39		32	13		
				40					

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EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

2008 REPORT ID: PB-6 (78)

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION -- | LOGGED BY F.M.

DEPTH TO GROUNDWATER 23' (SeeNote3) | BORING DIAMETER 8 Inches | DATE DRILLED 6/22/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (fine to medium grained) gravelly to 1" with some clay (continued)	mottled browns-black-red	dense	GC	41–46		52	16		
CLAY, silty with some sand (fine to medium grained)	mottled red-brown-black	hard	CL	47–49		46	20		
Bottom of Boring = 50 Feet				50					

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see page A-1.
3. Groundwater level measured at time of drilling.

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Geotechnical Consultants

EXPLORATORY BORING LOG
ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

2008 REPORT ID: PB-6 (78)

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, sandy Liquid Limit = 22% Plasticity Index = 19% Passing #200 Sieve = 62%		dark brown	firm	CL	1		5	26		
(grading sandier with depth)					2–4					
Liquid Limit = 36% Plasticity Index = 17%			stiff		5–6		12	19		
CLAY, sandy, gravelly Dry Density = 105 pcf		tan	stiff	CL	7–11			22		1.3
(grading less gravelly with depth)					12–16		11	21		
CLAY, silty		tan	stiff	CL	17–20					

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Consulting Engineers

EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

2008 REPORT ID: PB-1 (74)

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty Dry Density = 102 pcf		tan	stiff	CL	21–22			24	0.6	1.2
SAND (well graded), gravelly, clayey Passing #4 Sieve = 70% Passing #50 Sieve = 29% Passing #200 Sieve = 15%		tan	medium dense	SC	23–26		19			
CLAY, silty Dry Density = 104 pcf		mottled tan & brown	very stiff	CL	27–32			29		6.1
SAND (well graded), gravelly		tan	medium dense	SW-SP	33–35		19	19		
CLAY, silty		tan	very stiff	CL	36–39					
SAND (well graded), clayey, gravelly		tan	dense	SC	40					

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EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (well graded), clayey, gravelly Passing #4 Sieve = 78% Passing #50 Sieve = 41% Passing #200 Sieve = 26%		tan	dense	SC	41–44		34			
CLAY, sandy, gravelly Passing #200 Sieve = 57% Dry Density = 111 pcf		tan	hard	CL	45–47			20		9.1
GRAVEL (well graded), sandy		brown	dense	GW	48–49					
CLAY, silty, sandy Liquid Limit = 35% Plasticity Index = 18%		tan	very stiff	CL	50–53		31	17		
SAND (well graded), clayey, gravelly Passing #4 Sieve = 78% Passing #50 Sieve = 42% Passing #200 Sieve = 29%		brown	very dense	SC	54–56		70			
CLAY, silty, sandy		tan	hard	CL	57–60					

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EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

2008 REPORT ID: PB-1 (74)

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/10/73

DESCRIPTION AND CLASSIFICATION					DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE BLOWS/FT.	WATER CONTENT %	SHEAR STRENGTH BY TORVANE TSF	UNCONFINED COMPRESSIVE STRENGTH TSF
DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE						
CLAY, silty, sandy Dry Density = 124 pcf		tan	hard	CL	61					
SAND (well graded), clayey, gravelly		tan	dense	SC-SW	62		15			
					63					
CLAY, silty, sandy		brown	hard	CL	64					
Dry Density = 104 pcf					65					
					66			24		4.7
					67					
					68					
					69					
					70					
Dry Density = 113 pcf					71			19		7.7
					72					
					73					
					74					
					75					
					76					
(grading gravelly with depth)					77					
					78					
					79					
					80					

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/10/73

DESCRIPTION AND CLASSIFICATION					DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE BLOWS/FT.	WATER CONTENT %	SHEAR STRENGTH BY TORVANE TSF	UNCONFINED COMPRESSIVE STRENGTH TSF
DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE						
CLAY, silty, sandy, gravelly		brown	hard	CL	81		73			
					82					
(grading less gravelly with depth)					83					
					84					
					85					
Dry Density = 105 pcf					86			23		5.7
					87					
					88					
					89					
					90					
(grading stiffer with depth)					91					
					92					
					93					
					94					
					95					
Dry Density = 107 pcf					96			21		6.9
					97					
					98					
					99					
					100					

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

2008 REPORT ID: PB-1 (74)

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, sandy		brown	hard	CL	101					
					102					
SAND (well graded)		brown	very dense	SC	103					
					104					
Passing #200 Sieve = 40%					105		85/11"			
					106					
					107					
					108					
CLAY, silty, sandy		brown	hard	CL	109					
					110					
					111					
					112					
					113					
					114					
(grading sandier with depth)					115					
Dry Density = 109 pcf					116			20		7.3
					117					
SAND (well graded), clayey		brown	very dense	SC	118					
					119					
					120					

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EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (well graded), clayey		brown	very dense	SC	121					
CLAY, silty		tan	hard	CL	122					
					123					
					124					
					125					
Dry Density = 112 pcf					126			19		14.0
					127					
					128					
(sandy between depths of 129 and 130 feet)					129					
					130					
					131					
					132					
					133					
					134					
					135					
Dry Density = 104 pcf					136			23		6.8
					137					
					138					
					139					
					140					

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EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

2008 REPORT ID: PB-1 (74)

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYM-BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH KSF
CLAY, silty		tan	hard	CL	141					
					142					
					143					
(grading sandier with depth)					144					
					145					
CLAY, silty, sandy		dark brown	hard	CL	146					
					147					
					148					
Dry Density = 114 pcf					149			19		12.8
					150					
Bottom of Boring = 150 Feet										

Notes:

(1) The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

(2) The groundwater level was not established because of the use of bentonite during drilling.

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Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

2008 REPORT ID: PB-1 (74)

APPENDIX A
FIELD EXPLORATIONS

APPENDIX A
FIELD EXPLORATIONS

The field exploration consisted of a surface reconnaissance and a subsurface exploration program. Our subsurface investigation included drilling 12 conventional geotechnical borings. The following sections contain a discussion regarding the equipment and procedures to perform the exploration. The logs of the borings, as well as a key for the classification of the soil (Plate A-1), are included as part of this appendix.

Taber Consultants, of West Sacramento, California, drilled the 12 borings for this project between February 11 and 15, 2008. The borings were drilled using a truck-mounted drill rig equipped with 4-inch diameter solid-flight augers and mud rotary drilling capabilities. The borings were advanced to a maximum depth of about 80 feet below grade. The approximate locations of the borings are shown on Plate 3 – Site Plan.

Soil samples were obtained using either a standard split spoon sampler (2-inch outer diameter, 1 3/8-inch inner diameter) or Modified California Liner sampler (3-inch outer diameter, 2 3/8-inch inner diameter). The sampler type is indicated on the boring logs as designated on the Terms and Symbols, Plate A-1. Soil samples were taken semi continuously to a depth of about 10 feet and at 5-foot intervals thereafter. The Standard Penetration Test (SPT) was performed as part of the sampling procedure (in accordance with ASTM D-1586). The Standard Penetration Test is a measure of soil density and consistency. The SPT-N value (blow count) is defined as the number of blows required to drive a 2-inch diameter split-barrel sampler 12-inches, after an initial penetration of 6-inches, using a 140-pound hammer falling freely for 30-inches. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the sampler the last 12 inches. We note that samples obtained using the Modified California Liner sampler are not considered standard N-values because of the larger diameter of the samplers.

A Fugro engineer observed the drilling operation and logged the soil and rock encountered. Our inspecting engineer recorded SPT N-values. Recovered soil samples were visually examined and classified in accordance with the Unified Soil Classification System (USCS) and ASTM D-2488 (visual method). Our engineer estimated soil undrained shear strength in the field with a Pocket Penetrometer. Soil samples were transported to our office and laboratory for further examination, confirmation of classification and laboratory testing.

The surface elevations on the boring logs were estimated by interpreting the topographic contours on the project topographic map prepared by Chaudhary & Associates, Inc.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated. We do not warrant they are representative of subsurface conditions at other locations and times.

APPENDIX B
LABORATORY TESTING PROGRAM

APPENDIX B
LABORATORY TESTING PROGRAM

The laboratory-testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. A summary of the laboratory tests performed is included as Plate B-1.

The natural water content was measured on 27 samples recovered from the borings in accordance with ASTM Test Designation D-2216. Water contents are recorded on the boring logs at the appropriate sample depths.

Dry density was measured on 16 samples of the subsurface soils in accordance with ASTM Test Designation D-2937. The results of these tests are shown on the boring logs at the appropriate sample depths.

Atterberg limits were performed on 11 samples of the subsurface soils to estimate the range of water content over which these materials exhibit plasticity. The Atterberg limits were performed in accordance with ASTM Test Designations D-428 and D-424. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of these tests are presented on Plate B-2a, and on the logs of the borings at the appropriate sample depths. Results of Atterberg limit test from pertinent historic borings are included on Plate B-2b.

The percent passing the No. 200 sieve was measured on 14 samples of the subsurface soils to aid in the classification of these soils. These tests were performed in accordance with ASTM Test Designation D-1140. The results of these tests are shown on the boring logs at the appropriate sample depths.

Sieve analyses were performed on four samples of the subsurface soils to aid in the classification of these soils. This test was performed in accordance with ASTM Test Designation C136. The results of these tests are presented on Plate B-3.

Miniature vane tests were performed on nine samples of the subsurface soils to estimate the undrained shear strength and residual strength of these layers to provide data for bearing capacity and foundation recommendations. These tests were performed in accordance with ASTM Test Designation D4648. The results of these tests are presented on the boring logs at the appropriate sample depths.

Two Resistance (R-value) tests were performed on representative samples of the surface soils onsite to provide data for pavement design. The tests were performed in accordance with California Test Method 301-F and indicated a typical R-value of 4 and 9 at an exudation pressure of 300 pounds per square inch. The results of the tests are presented below:

R-value Test Results – Bulk Sample 1 (B-102)

Description of Material	Dry Density (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
Dark Brown fat CLAY with sand (CH)	92.1	27.7	223	0	2
	93.1	26.5	286	0	3
	94.9	25.4	342	0	8
R-Value = 4 at Exudation pressure of 300 psi					

R-value Test Results – Bulk Sample 2 (B-106)

Description of Material	Dry Density (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
Dark Brown lean to fat CLAY with sand (CL-CH)	100.6	21.8	231	0	5
	101.5	20.7	278	0	7
	105	19.5	334	0	14
R-Value = 9 at Exudation pressure of 300 psi					

BORING	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC%	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	TEST LISTING
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4				
B-101	4.5	4	Lean CLAY with sand (CL)	126	103	23																	T
B-101	10.0	6	Lean CLAY (CL)	126	102	23																	T
B-101	14.5	7	Clayey SAND with gravel ()				34																S
B-101	15.0	8	Clayey SAND with gravel (SC)	126	107	18									3.45+								T, m
B-101	19.5	9	Clayey SAND with gravel (SC)	131	111	18									3.45+								T, m
B-101	24.0	11	Clayey SAND with gravel (SC)				19																S
B-102	2.0	Bulk	Lean CLAY (CL)																	4			R
B-102	20.0	11	Lean CLAY with sand (CL)			20		35	21														M, A
B-102	25.0	12	Sandy Lean CLAY (CL)			20	71	23	11														M, A, FC
B-103	7.5	5	Lean CLAY (CL)	123	100	23																	T
B-103	21.0	11	Clayey SAND with gravel (SC)				35																FC
B-103	23.0	12	Clayey SAND with gravel (SC)				22																FC
B-103	25.0	13	Clayey SAND with gravel (SC)				27																FC
B-104	3.0	2	Lean CLAY (CL)	125	107	17		43	26							1300	6.80	52	110				T, A, Co
B-104	15.5	6	Clayey SAND with gravel (SC)				46																FC
B-105	1.0	1	Lean CLAY with sand (CL)			26		41	24														M, A
B-105	5.5	2	Lean CLAY with sand (CL)	123	100	23		41	28														T, A
B-105	10.5	4	Sandy Lean CLAY (CL)				72																FC
B-105	15.5	6	Fat CLAY (CH)			27										1700	7.50	ND	ND				M, Co
B-106	2.0	Bulk	Lean CLAY (CL)																	9			R
B-106	13.5	10	Lean CLAY (CL)	128	105	21									3.31								T, m
B-106	24.0	14	Sandy Lean CLAY with gravel (CL)	129	108	20									3.45+								T, m
B-106	33.0	17	Clayey GRAVEL (GC)				33																FC
B-106	63.5	24	Lean CLAY (CL)												3.45+								m
B-106	74.5	27	Lean CLAY (CL)												3.45+								m
B-107	10.0	7	Lean CLAY with sand (CL)			18		29	16														M, A
B-107	20.0	10	Clayey SAND with gravel (SC)				48																FC
B-108	1.0	1	Lean CLAY with sand (CL)			20		38	22														M, A
B-108	10.0	7	Lean CLAY with sand (CL)			21		41	29														M, A
B-108	20.0	10	Lean CLAY with sand (CL)			25		42	27														M, A

Classification Tests
UWW = Unit Wet Weight
UDW = Unit Dry Weight
MC = Moisture Content
Fines = % Passing #200 Sieve
LL = Liquid Limit
PI = Plasticity Index
NP = Non-plastic

Direct Shear Test
C = Assigned Cohesion, ksf
PHI = Assigned Friction Angle, degrees

Compaction Test
MAX DD = Maximum Dry Density
OPT MC = Optimum Moisture Content

Compressive Strength Tests
Qu = Unconfined Compression
Su = Undrained Shear Strength
u = Unconsolidated Undrained
p = Pocket Penetrometer
t = Torvane
m = Miniature Vane

Corrosivity Tests
R = Resistivity, ohm-cm, satur.
pH = pH
Cl = Chloride, ppm
SO_4 = Sulfate, ppm

Test Listing Abbreviations
M = Moisture Content
T = Total & Dry Unit Weight
S = Sieve Analysis
FC = % Passing #200 Sieve
H = Hydrometer Analysis
A = Atterberg Limits
P = Compaction Test
D = Direct Shear Test
C = Consolidation Test
Co = Corrosivity Tests
CU = CU Triaxial
U = UU Triaxial
R = R-Value
SE = Sand Equivelant

SUMMARY OF LABORATORY TEST RESULTS

New Hospital Replacement Project
Children's Hospital and Research Center

Oakland, California

PLATE B--1a

SUM-1 LAB_SUMMRY (G:\JOBDOCS\1595\1595.002 CHRCO\GINT\1595.002.GPJ) - VTA- 5/9/08 08:47 AM

BORING	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC%	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	TEST LISTING
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4				
B-108	25.0	11	Clayey GRAVEL (GC)				44																FC
B-108	35.0	14	Clayey SAND with gravel (SC)				20																FC
B-109	5.0	3	Lean CLAY (CL)	125	107	17																	T
B-109	9.0	5	Lean CLAY (CL)	130	110	18																	T
B-109	15.5	7	Sandy Lean CLAY (CL)			19	66																M, FC
B-109	16.0	8	Sandy Lean CLAY (CL)	123	104	19									3.45+								T, m
B-109	21.0	10	Clayey SAND with gravel (SC)				21																S
B-109	37.0	17	Well-graded SAND with clay (SW-SC)			21		30	16														M, A
B-109	38.0	18	Well-graded SAND with clay (SW-SC)				12																FC
B-109	40.0	20	Sandy Fat CLAY (CH)				58																FC
B-111	5.0	2	Lean CLAY (CL)	118	93	27																	T
B-111	11.0	6	Sandy Lean CLAY (CL)	129	107	21																	T
B-112	4.0	3	Lean CLAY with sand (CL)	113	92	22									1.84	1800	5.60	ND	33				T, Co, m
B-112	7.5	5	Lean CLAY with sand (CL)	125	102	23									3.03								T, m
B-112	16.0	9	Clayey SAND with gravel (SC)				14																FC
B-112	18.0	10	Clayey SAND with gravel (SC)			17	15	31	17														M, A, S
B-112	25.0	12	Sandy Lean CLAY (CL)				66																FC

Classification Tests
UWW = Unit Wet Weight
UDW = Unit Dry Weight
MC = Moisture Content
Fines = % Passing #200 Sieve
LL = Liquid Limit
PI = Plasticity Index
NP = Non-plastic

Direct Shear Test
C = Assigned Cohesion, ksf
PHI = Assigned Friction Angle, degrees

Compaction Test
MAX DD = Maximum Dry Density
OPT MC = Optimum Moisture Content

Compressive Strength Tests
Qu = Unconfined Compression
Su = Undrained Shear Strength
u = Unconsolidated Undrained
p = Pocket Penetrometer
t = Torvane
m = Miniature Vane

Corrosivity Tests
R = Resistivity, ohm-cm, satur.
pH = pH
Cl = Chloride, ppm
SO_4 = Sulfate, ppm

Test Listing Abbreviations
M = Moisture Content
T = Total & Dry Unit Weight
S = Sieve Analysis
FC = % Passing #200 Sieve
H = Hydrometer Analysis
A = Atterberg Limits
P = Compaction Test
D = Direct Shear Test
C = Consolidation Test
Co = Corrosivity Tests
CU = CU Triaxial
U = UU Triaxial
R = R-Value
SE = Sand Equivelant

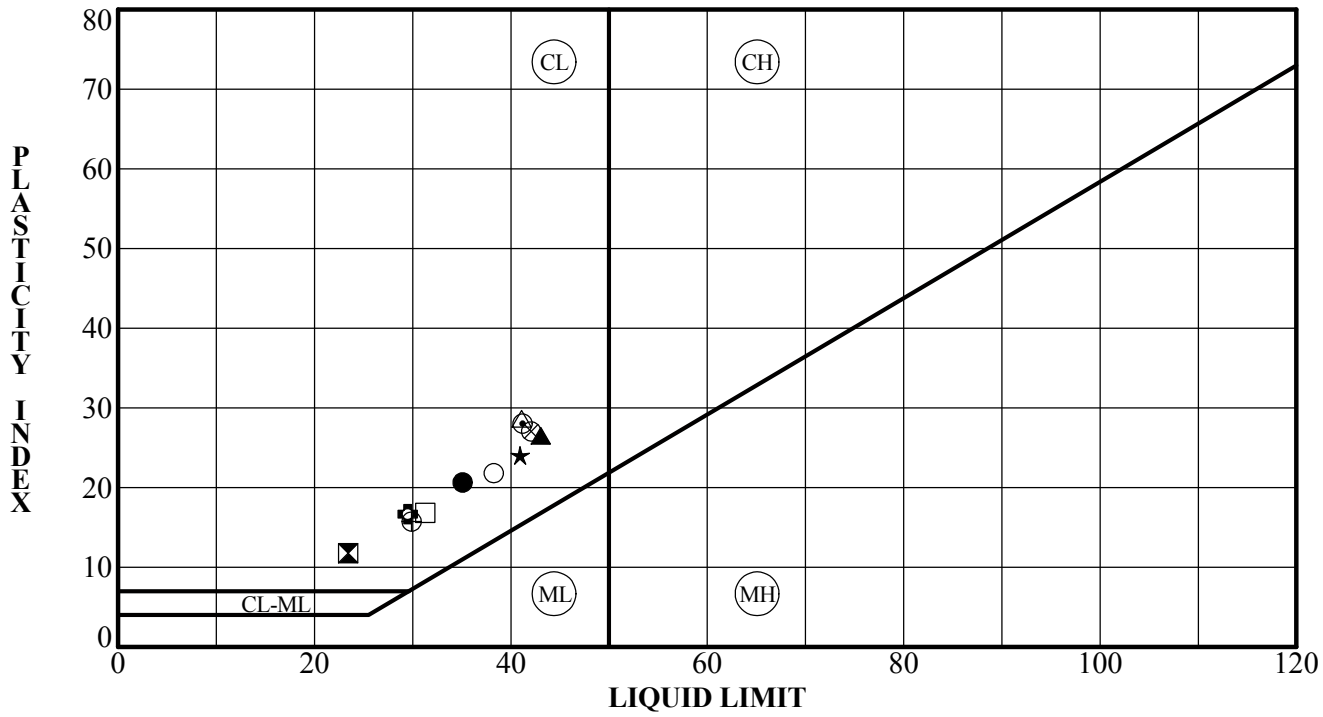
SUMMARY OF LABORATORY TEST RESULTS

New Hospital Replacement Project
Children's Hospital and Research Center

Oakland, California

PLATE B--1b

SUM-1 LAB_SUMMRY (G:\JOBDOCS\1595\1595.002 CHRCO\GINT\1595.002.GPJ) - VTA- 5/9/08 08:47 AM



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	B-102	20.0	35	21	0.260	20		CL
⊠	B-102	25.0	23	12	0.675	20	71	CL
▲	B-104	3.0	43	26	0.032	17		CL
★	B-105	1.0	41	24	0.374	26		CL
⊙	B-105	5.5	41	28	0.344	23		CL
✿	B-107	10.0	29	17	0.312	18		CL
○	B-108	1.0	38	22	0.140	20		CL
△	B-108	10.0	41	29	0.305	21		CL
⊗	B-108	20.0	42	27	0.373	25		CL
⊕	B-109	37.0	30	16	0.454	21	12	SC
□	B-112	18.0	31	17	0.157	17		SC

ATTERBERG 1595.002.GPJ VERSION-012505.GDT 3/26/08

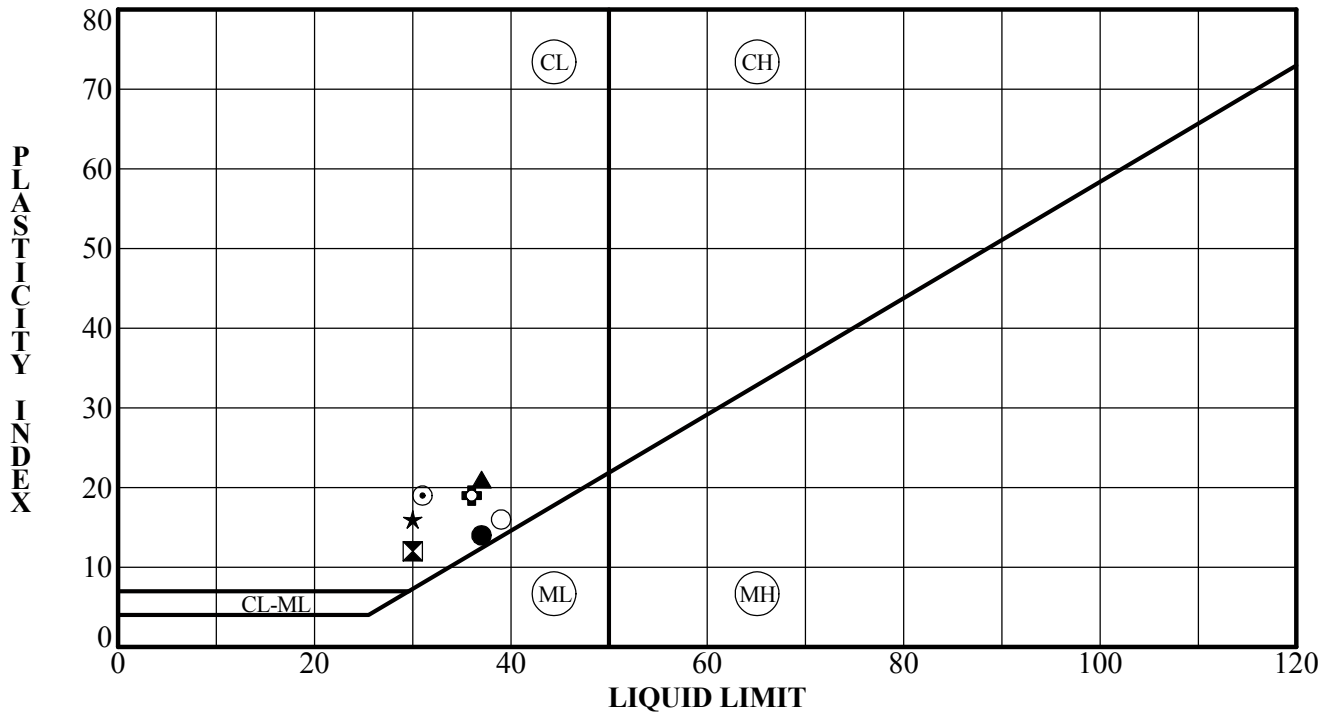
FUGRO

PREP'D BY: KG
APP'D BY: M. Paquette
DATE: 3/26/08
DWG FILE:

PLASTICITY CHART AND DATA

NEW HOSPITAL REPLACEMENT PROJECT
Oakland, California

PLATE
B-2a
PROJECT No.
1595.002



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	PB-1 (86b)	0.0	37	14	0.071	24	72	CL
⊠	PB-1 (87)	1.0	30	12	---	---	49	SC
▲	PB-1 (97)	3.0	37	21	0.238	21	72	CL
★	PB-2 (87)	34.5	30	16	0.813	27	79	CL
⊙	PB-2 (97)	4.0	31	19	---	---	35	SC
✿	PB-3 (94)	4.5	36	19	---	---	72	CL
○	PB-4 (86b)	1.0	39	16	0.063	24	68	CL

FUGRO

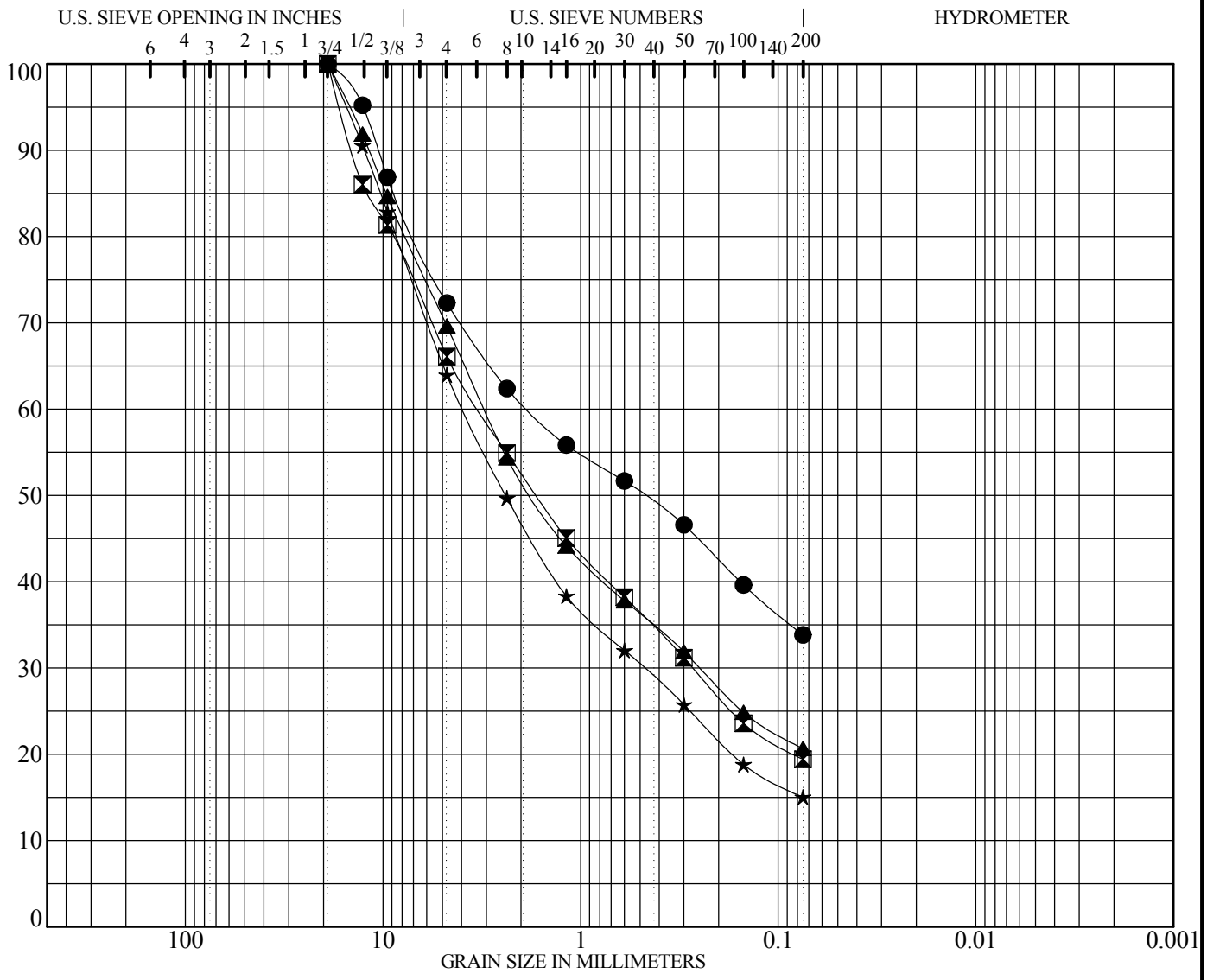
PREP'D BY:
APP'D BY:
DATE: 3/26/08
DWG FILE:

PLASTICITY CHART AND DATA

NEW HOSPITAL REPLACEMENT PROJECT
Oakland, California

PLATE
B-2b
PROJECT No.
1595.002

ATTERBERG 1595.002.GPJ VERSION-012505.GDT 3/26/08



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	B-101	14.5	34	72	Yellow brown clayey SAND with gravel	SC
⊠	B-101	24.0	19	66	Light brown clayey SAND with gravel	SC
▲	B-109	21.0	21	70	Orange brown clayey SAND with gravel	SC
★	B-112	18.0	15	64	Brown clayey SAND with gravel	SC

FUGRO

PREP'D BY: KG
APP'D BY: M. Paquette
DATE: 3/26/08
DWG FILE:

GRADATION TEST DATA

NEW HOSPITAL REPLACEMENT PROJECT
Oakland, California

PLATE
B-3
PROJECT No.
1595.002

GRADATION_B 1595.002.GPJ STD.GDT 3/26/08

California State Certified Laboratory No.2153

C E R C O
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

19 March 2008

Job No.0803031
Cust. No.11608

Mr. Kap Gupta
Fugro West, Inc.
1000 Broadway, Suite 440
Oakland, CA 94607

Subject: Project No.: 1595.002
Project Name: Oakland – Childrens Hospital
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. Gupta:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on March 05, 2008. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, all samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from none detected to 52 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration ranges from none detected to 110 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 5.6 to 7.5 which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potentials range from 410 to 450-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No.2153

CERCO analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

Client: Fugro West
Client's Project No.: 1595.002
Client's Project Name: Oakland - Childrens Hospital
Date Sampled: Not indicated
Date Received: 5-Mar-08
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 19-Mar-2008

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0803031-001	B-104/1 @ 2.5'	420	6.8	-	1,300	-	52	110
0803031-002	B-105/6 @ 15.5'	450	7.5	-	1,700	-	N.D.	N.D.
0803031-003	B-112/3 @ 4'	410	5.6	-	1,800	-	N.D.	33

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	11-Mar-2008	11-Mar-2008	-	13-Mar-2008	-	11-Mar-2008	11-Mar-2008

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel:925.462.2771
Fax:925.462.2775

CERCO analytical, inc.

Chain Of Custody

Page of

Job No. 080203 | Client No. 11608 | Client Project I.D. 1595.002

Sample Schedule: WK MO QT AN Resample New: | Date Sampled | Date Due

Contact Name: Kap Gupta

Company Name: Fugro West Oakland

Sample Source: Oakland - Childrens Hospital

Lab No.	Sample I.D.	Date	Time	Matrix	Container	Preserv.	Qty.	ASTM Test Methods* with Brief Evaluation	CalTrans Test Methods** with Brief Evaluation
001	B-104/1 @2.5	N					1	/	
002	B-105/6 @15.5						1	/	
003	B-112/3 @4.0						1	/	

MATRIX: DW - Drinking Water; GW - Ground Water; SW - Surface Water; WW - Waste Water; SL - Sludge; S - Soil

ABBREVIATIONS: HB - Hosebib; PV - Petcock Valve; PT - Pressure Tank; PH - Pump House; RR - Restroom; GL - Glass; PL - Plastic; ST - Sterile

SAMPLE RECEIPT: Total No. of Containers: 3; Rec'd Good Cond./Cold; Conforms to Record; Sampler

Relinquished By	Date	Time
	3/5/08	rd: 10
Received By	Date	Time
	3/5/08	10:08
Relinquished By	Date	Time
Received By	Date	Time
Relinquished By	Date	Time
Received By	Date	Time

Sample Location/Instructions

* Includes Redox, pH, Resistivity (100% Saturation), Chloride and Sulfate

** Includes pH, Minimum Resistivity, Chloride & Sulfate

There may be an additional charge for submitting soil in brass tubes.

APPENDIX C
SEISMIC HAZARD ANALYSIS AND GROUND MOTIONS

APPENDIX C
SEISMIC HAZARD ANALYSIS AND SITE-SPECIFIC GROUND MOTIONS

1.0 INTRODUCTION

This appendix documents the seismic hazard evaluation study consisting of probabilistic seismic hazard analysis (PSHA) and deterministic analysis conducted to update the design ground motion criteria as per the 2007 CBC code for New Hospital Replacement Project for Children's Hospital & Research Center at Oakland (CHRCO). The new hospital site is located on Dover Street, between 52nd Street and 53rd Street, in Oakland, Alameda County, California.

The geotechnical study conducted by Harza Kaldveer (Now Fugro) in 1994 included a seismic hazard evaluation. Since January 2008, California has adopted the 2007 California Building Code (CBC, 2007); therefore our seismic hazard evaluation study as summarized in this appendix has been revised to meet the requirements of the 2007 CBC for the hospital to be designed according to 2007 CBC.

Site-specific probabilistic seismic hazard analysis (PSHA) and deterministic analysis have been conducted in accordance with the 2007 CBC requirements to develop design ground motion criteria for a Maximum Considered Earthquake (MCE) and a Design Basis Earthquake (DBE).

The scope of work for this aspect of the project consisted of:

- Characterizing the subsurface conditions, based on the available subsurface information;
- Identifying potential seismic sources and fault parameters;
- Performing probabilistic seismic hazard analyses (PSHA) to generate acceleration response spectra at the ground surface for horizontal and vertical components; and
- Comparing the PSHA results with deterministic analysis in order to develop the site-specific design spectra for the project according to the 2007 CBC.

2.0 SUBSURFACE CONDITIONS

The soils encountered across the site in our borings typically consisted of a layer of lean clay fill, over interlayered clayey sands to sandy clays with varying gravel content. The clay fill was observed to a typical depth of about 5 feet. Stiff lean clay was generally encountered between depths of 5 and 20 feet. Below this stiff clay, interlayered clayey sands and sandy clays, as well as some gravel layers, were observed to depths ranging from 24 to 46 feet. The previous deep boring, PB-1 (74), shows a lean clay layer below 50 feet to a depth of 150 feet with two clayey sand layers encountered from 102 to108.5 feet and 118 to121 feet. The subsurface conditions at the site of each proposed structure are presented in Section 4.3 of this report. The boring logs are presented in Appendix A. In general, the subsurface conditions across the site are fairly similar and consist of fill underlain by alluvial deposits.

Idealized Soil Profile

Idealized soil profile, design unit weight, and shear strength parameters were developed based on the results of the field investigations and laboratory testing conducted by Fugro. The profile is based on the field and laboratory test data from twelve exploratory borings.

Due to the variability of the soil with the alluvial deposits, data from all of the 12 borings conducted by Fugro were utilized to develop idealized shear wave velocity profiles. Generally the soil profile consisted of a layer of stiff clay fill, underlain by sand and clay layers, underlain by stiff clay. Figure C-1 shows shear wave velocity profiles for all 12 borings performed by Fugro in 2008. Based on these, two idealized profiles were developed, representative of the lower and upper bounds of the shear wave velocities. Profiles "Design Low" and "Design High" have average shear wave velocities of approximately 800 and 1000 feet per second, respectively. Since the two profiles are similar, a single idealized average shear wave velocity of approximately 900 feet per second was used for the seismic hazard analyses.

Based on the idealized shear wave velocities, the entire project site is considered to be Site Class "D" per CBC 2007.

3.0 METHODOLOGY FOR DEVELOPMENT OF DESIGN RESPONSE SPECTRA

The procedure for development of a design response spectrum for this site consisted of the following three steps:

1. Conducting probabilistic seismic hazard analysis (PSHA) using the Next Generation Attenuation (NGA) relationships and in accordance with the 2007 CBC requirements.
2. Conducting deterministic seismic hazard analysis (DSHA) to determine the largest median acceleration response spectrum from characteristic earthquakes on all known active faults in the region.
3. Probabilistic and deterministic results were used to develop design ground motion criteria for a Maximum Considered Earthquake (MCE) and a Design Basis Earthquake (DBE) according to CBC (2007) that is applicable at the foundation level. CBC (2007) defines MCE to be an event with 2 percent probability of occurrence in 50 years (2475-year return period event). The DBE is obtained as 2/3 of the MCE along with some deterministic checks. Results provided include the horizontal MCE and DBE spectra, and deaggregation of results by magnitude and distance as well as by contributing faults, as needed for CGS review.

4.0 PROBABILISTIC SEISMIC HAZARD ANALYSES

Introduction

Probabilistic seismic hazard analyses were performed to evaluate the probability of occurrence of ground motion corresponding to the 2475-year return period event in order to develop the Maximum Considered Earthquake (MCE) per CBC 2007. The spectral analyses

consider site-specific subsurface conditions and the regional seismic setting. The spectra were developed at the ground surface.

Tectonic/Seismic Setting

The Oakland Children's Hospital is located in the Coast Ranges geomorphic province of California, which is characterized by northwest-southeast trending valleys and ridges. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates and subsequent strike-slip faulting along the San Andreas fault zone. The potential for surface fault rupture at the site is low. The site lies approximately 2 miles from the closest active fault zone (Hayward fault).

Identification and Characterization of Potential Seismogenic Sources

A potential seismogenic source is one that exhibits historic displacement, historic earthquake activity, or geologic evidence of activity during the latest Holocene (approximately the past 11,000 years). All significant seismogenic sources within 100 km of the site were identified and characterized with respect to geometric and geologic conditions that affect the development of seismic source parameters for use in the development of the earthquake ground motions. The key geometric, geologic and seismologic characteristics that were compiled or estimated from available data for each potential seismogenic source include:

- Style of Faulting (e.g. strike-slip, reverse, normal);
- Closest source-to-site distance;
- Total fault length and rupture length;
- Rupture width;
- Slip rate; and
- Maximum magnitude (M_{max}).

Table C-1 lists the key fault parameters for the sources found within a 25 km radius of the site. These parameters were subsequently used in the seismic hazard analyses and were adapted from the Working Group on California Earthquake Probabilities (WGCEP, 2003).

Table C-1: Geologic Characteristics of Significant Seismogenic Sources within 25 km of the Site

Potential Seismogenic Source [1]	Type [2], [3]	Closest Distance in km (miles)	Slip Rate (mm/yr)[9]	M_{max}[9] [4]
Hayward Rogers Creek (HS)	RL-SS	3.3 (2.1)	9.0	6.67
Hayward Rogers Creek (HN)	RL-SS	3.3 (2.1)	9.0	6.49
Hayward Rogers Creek (HS+HN)	RL-SS	3.3 (2.1)	9.0	6.91
Hayward Rogers Creek (HN+RC)	RL-SS	3.3 (2.1)	9.0	7.11
Hayward Rogers Creek (HS+HN+RC)	RL-SS	3.3 (2.1)	9.0	7.26
Hayward Rogers Creek (FLOATING)	RL-SS	3.3 (2.1)	9.0	6.90
Background	-	-	Activity Rate (M>5.0) 0.45 events/yr	7.25
Calaveras (CN)	RL-SS	20.6 (12.8)	6.0	6.78
Calaveras (CC+CN)	RL-SS	20.6 (12.8)	6.0	6.90
Calaveras (CS+CC+CN)	RL-SS	20.7 (12.9)	6.0	6.93
Calaveras (Floating)	RL-SS	20.7 (12.9)	6.0	6.20
Mt Diablo (MTD)	RL-SS	25.2 (15.7)	2.0	6.65
Concord (CON)	RL-SS	25.2 (15.7)	4.0	6.25
Concord (CON+GVS)	RL-SS	25.2 (15.7)	5.0	6.58
Concord (CON+GVS+GVN)	RL-SS	25.2 (15.7)	5.0	6.71
Concord (FLOATING)	RL-SS	25.2 (15.7)	5.0	6.20
San Andreas (SAP)	RL-SS	25.7 (16.0)	17.0	7.15
San Andreas (SAS+SAP)	RL-SS	25.7 (16.0)	17.0	7.42
San Andreas (SAN+SAP+SAS)	RL-SS	25.7 (16.0)	24.0	7.76
San Andreas (SAP+SAN+SAO)	RL-SS	25.7 (16.0)	24.0	7.83
San Andreas (SAS+SAP+SAN+SAO)	RL-SS	25.7 (16.0)	24.0	7.90
San Andreas (FLOATING)	RL-SS	25.7 (16.0)	24.0	6.90

Notes:1. Various fault segments and rupture scenarios were considered as described in WGCEP (2003)
2. Fault parameters were adapted from the CGS fault database (2003) and WGCEP (2003).
3. SS = Strike Slip Fault, RL = Right Lateral, R = Reverse (Dip in degrees), O = Oblique
4. M_{max}= Maximum moment magnitude with highest weightage in the seismotectonic model (per WGCEP 2003)
5. SAS =Santa Cruz Mountain, SAP = Peninsula, SAN = North Coast North, SAO = North Coast South
6. SGN = San Gregorio North, SGS = San Gregorio South
7. HN = Hayward North, HS = Hayward South, RC = Rodgers Creek
8. CON = Concord, GVS = Green Valley South, GVN = Green Valley North
9. Only Mean slip rate and M_{max} values are listed here. WGCEP (2003) provides upper- and lower-bound values that were also used with appropriate weighting factors. Those are not listed here for brevity.

Seismic Hazard Analysis Methodology

Probabilistic seismic hazard analyses (PSHA) were carried out using the computer program HAZ38 (Abrahamson, 2007). The program employs the analytical procedure to compute seismic hazard originally developed by Cornell (1968).

The activity of the seismic sources was modeled using the Youngs and Coppersmith (1985) composite model to simulate the relative distribution of magnitudes for each fault. In implementing the model in the hazard code, it was considered that for each fault the mean characteristic magnitude (M_{char}) is the maximum magnitude (M_{max}) tabulated in Table C-1. The composite model includes a characteristic earthquake distribution for the large magnitude earthquakes ($M > M_{char} - 0.25$) and an exponential distribution for smaller magnitude earthquakes ($M \le M_{char} - 0.25$). The slope of the exponential part (i.e. b-value) was assigned a value of 0.8 based on regional seismicity data.

The probability density function of the Youngs and Coppersmith model is given by:

$$f_m^{YC}(M) = \begin{cases} \frac{1}{1+c_2} \cdot \frac{\beta \cdot \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 1.25)\right)}{1 - \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)} & for \quad \overline{M}_{char} - 0.25 \le M \le \overline{M}_{char} + 0.25 \\ \frac{1}{1+c_2} \cdot \frac{\beta \cdot \exp\left(-\beta \cdot (M - M_{\min})\right)}{1 - \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)} & for \quad M_{\min} \le M \le \overline{M}_{char} - 0.25 \end{cases}$$

where

$$c_2 = \frac{0.5 \cdot \beta \cdot \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 1.25)\right)}{1 - \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)},$$

$\beta = b \cdot \ln(10)$, and

$\overline{M}_{char}$ is the mean characteristic magnitude of the fault here taken equal to M_{max}.

The seismic hazard analyses were conducted using the recently developed Next Generation of Attenuation (NGAs) Relationships developed for shallow crustal earthquakes, as shown on Figure C-2 and considered the influence of near-source and rupture directivity effects. These relationships used were developed as part of the PEER NGA program (http://peer.berkeley.edu/). These are:

- Abrahamson and Silva (2008);

Boore and Atkinson (2007); and

Campbell and Bozorgnia (2007).

A fourth relationship has been developed by Chiou and Youngs (2006), but the relationship has currently not been finalized. This relationship was accordingly not included as part of our analyses.

Near-Source Effects

Because of the close proximity of the Hayward Rogers Creek fault to the project site (approximately 3.3 km), the near-source effects were judged to be important and were taken into account. For this purpose, additional analyses using HAZ38 were conducted to obtain the Fault Normal and Fault Parallel components as well as for average directivity.

Extending the Design Spectra to Long Periods

The directivity effects are defined only to periods of 5 seconds. This section summarizes the procedures used to develop design data for longer structural periods than those for which empirical attenuation relationships are available. A well-established methodology to extend a design spectrum to longer periods is not available in the literature. However, there appears to be consensus among the scientific community (e.g., Silva and Abrahamson, 1992; NEHRP, 2003) that in the absence of data at long periods design response spectra can be extended based on a constant spectral velocity assumption for the intermediate periods and a constant spectral displacement assumption for long periods. For design purposes the "corner period", marking the transition between constant spectral velocity and constant spectral displacement is a function of the earthquake magnitude and has been tabulated by NEHRP (2003). For this study the corner period was estimated as T_L= 8 s, corresponding to magnitudes ranging from 6.5 to 7.5. Between structural periods of 8 and 10 seconds, the target spectra were extended based on an assumption of constant spectral displacement. Between structural periods of 5 and 8 seconds, the target spectra were interpolated using a smooth curve on a tripartite plot.

PSHA Results

Figure C-3 shows a comparison of average horizontal component, fault normal and fault parallel components conducted for the idealized average shear wave velocity of 900 ft/s. Based on the HAZ38 results, the fault normal component gives higher spectral accelerations at the project site for longer structural periods. Therefore, for conservatism, the fault normal component was used for developing the design spectrum.

Deaggregation of Hazard

Figures C-4a through C-4c show deaggregation of the hazard for peak ground acceleration (PGA), spectral acceleration at a structural period of 0.2 second (SA-0.2s) and spectral acceleration at a structural period of 1 second (SA-1s). For all these structural periods almost the entire hazard comes from events with magnitudes ranging between 6.0 to 7.5 at distances between 0 and 5 km. This hazard is associated with events on the Hayward fault that lies approximately 3.3 km from the project site.

Figures C-5a through 5c show deaggregation of hazard by contributing faults for PGA, SA-0.2s, and SA-1s. As seen from the figures, the Hayward fault dominates the hazard contribution at the project site.

5.0 DETERMINISTIC HAZARD ANALYSIS

The mean deterministic spectrum was obtained from an earthquake occurring approximately 3.3 km from the project site with a magnitude of 7.4, which is the maximum M_{max} value for the rupture of the entire Hayward-Rodgers Creek fault. The deterministic spectrum was modified to account for near-source effects per recommendations by Sommerville (1997). Deterministic spectrum for the fault normal component is shown on Figure C-6.

6.0 DEVELOPMENT OF SPECTRA PER CBC 2007

Site-specific ground motion procedures were undertaken to develop the design spectrum in accordance with CBC 2007 and Chapter 21 of ASCE 7-05. Section 21.2 requires the calculation of the response spectrum for a Maximum Considered Earthquake (MCE) and the design spectrum is estimated as 2/3 of the MCE spectrum.

Evaluating the MCE involves both probabilistic and deterministic analysis. The probabilistic estimate of the MCE corresponds to the lesser of (1) ground motions that have a 2 percent probability of exceedance during a 50-year design life (i.e. return period of 2,475 years) per Section 21.2.1 (ASCE 7-05) and (2) the deterministic MCE. The deterministic MCE is obtained as the larger of (1) 150 percent of the largest median 5 percent damped spectral response acceleration at that period for characteristic earthquakes on all known active faults within the region (ASCE Section 21.2.2) and a lower-bound limit of the deterministic spectrum prescribed by the ASCE 7-05 Section 21.2.2. The deterministic spectrum was obtained from a M 7.4 earthquake occurring approximately 3.3 km from the project site, on Hayward fault.

The development of the MCE using these spectra and the procedures mentioned in ASCE 7-05 is shown on Figure C-7a. The Design Basis Earthquake (DBE) was developed as the larger of (1) 2/3 of the MCE and (2) 80 percent of the ASCE general spectrum as described in ASCE Section 11.4.5. The development of the DBE is shown on Figure C-7b.

The site-specific MCE and DBE spectra described in this report take into account the variation of near-surface stratigraphy at the project site. Therefore, these spectra are applicable directly at the foundation level.

Per ASCE 7-05 Section 21.4, the parameters S_{DS} and S_{D1} can be obtained from the site-specific design spectrum. Per Section 21.4, the S_{D1} value is calculated as the larger of: (1) spectral acceleration at 1 second and (2) two times the spectral acceleration at 2 seconds. Also, the S_{DS} value is calculated as the larger of: (1) spectra acceleration at 0.2 second, and (2) 90 percent of the largest spectral acceleration for any period greater than 0.2 second. Based on this, the design parameters for the project are:

S_{DS} = 1.00 and S_{D1} = 0.70.

The ordinates of the design spectra are presented in the following table:

Table C-2: Horizontal Design Response Spectra Ordinates

Period (seconds)	Design Basis Earthquake (DBE) (g)	Maximum Considered Earthquake (MCE) (g)
0.01	0.60	0.90
0.03	0.60	0.90
0.1	1.00	1.50
0.15	1.00	1.50
0.2	1.00	1.50
0.3	1.00	1.50
0.5	1.00	1.50
0.75	0.80	1.20
1	0.60	0.90
1.5	0.42	0.63
2	0.35	0.53
3	0.25	0.37
4	0.20	0.29
5	0.15	0.22
6	0.12	0.17
7	0.09	0.13
8	0.08	0.11
9	0.07	0.10
10	0.06	0.09

7.0 SEISMIC DESIGN CATEGORY

From the mapped values, the S_S value is 1.73 and the S_1 value is 0.64. Based on the site-specific study the S_{DS} value is 1.00 and the S_{D1} value is 0.70. Per Section 1613 .5 of CBC 2007, the Seismic Design Category is D.

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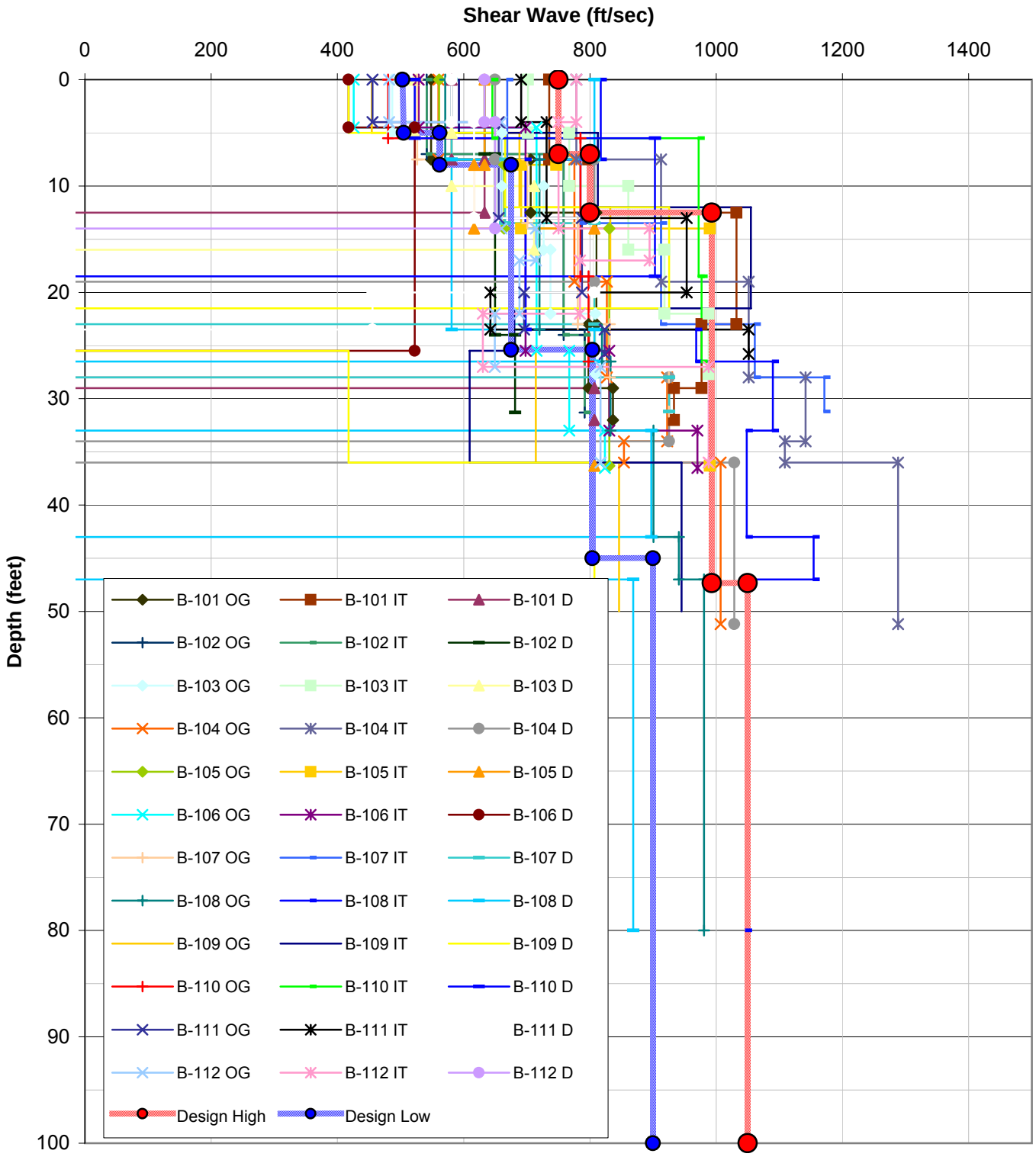
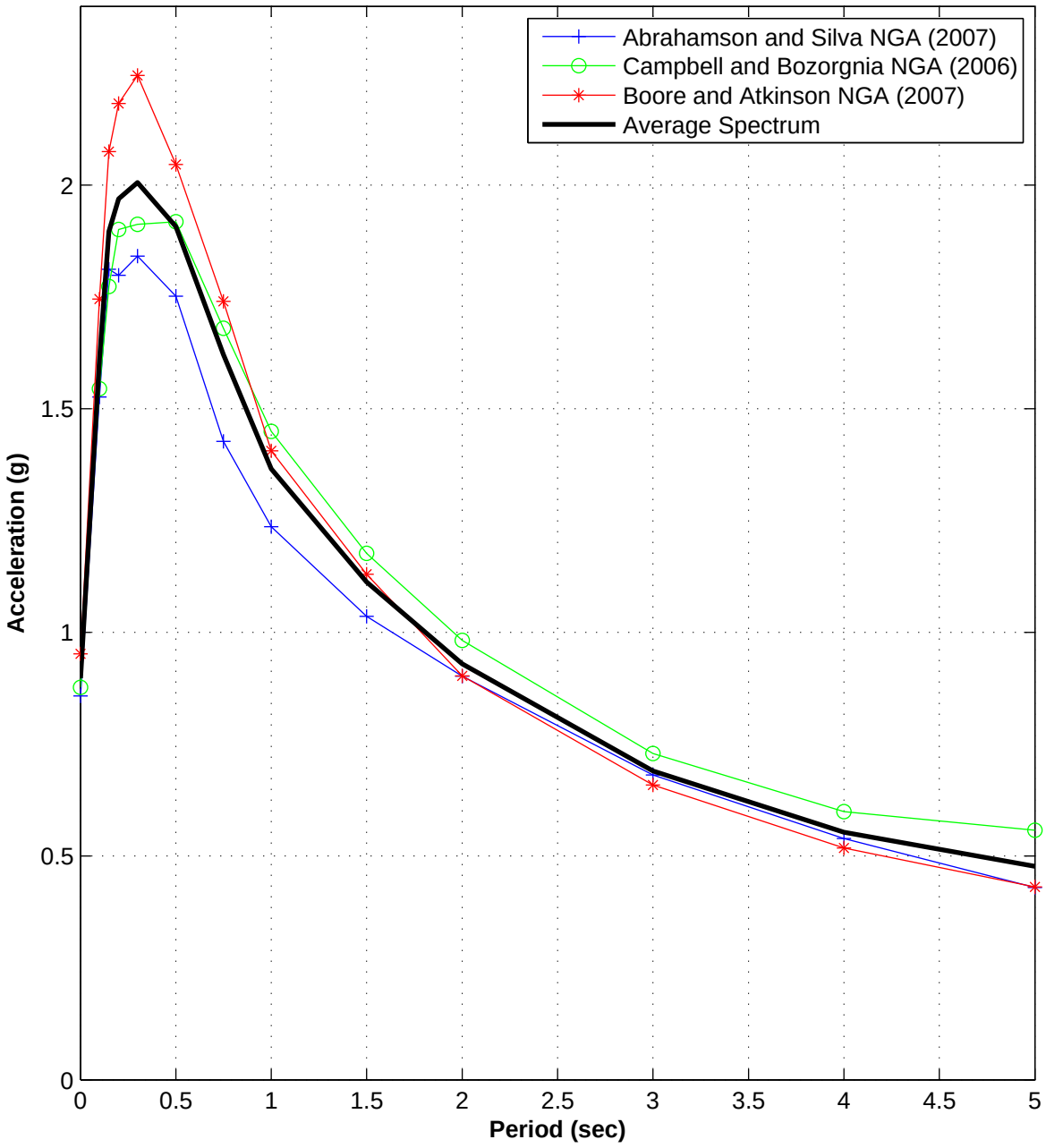


SHEAR WAVE VELOCITY PROFILES FOR FUGRO BORINGS

New Hospital Replacement Project
Oakland, California

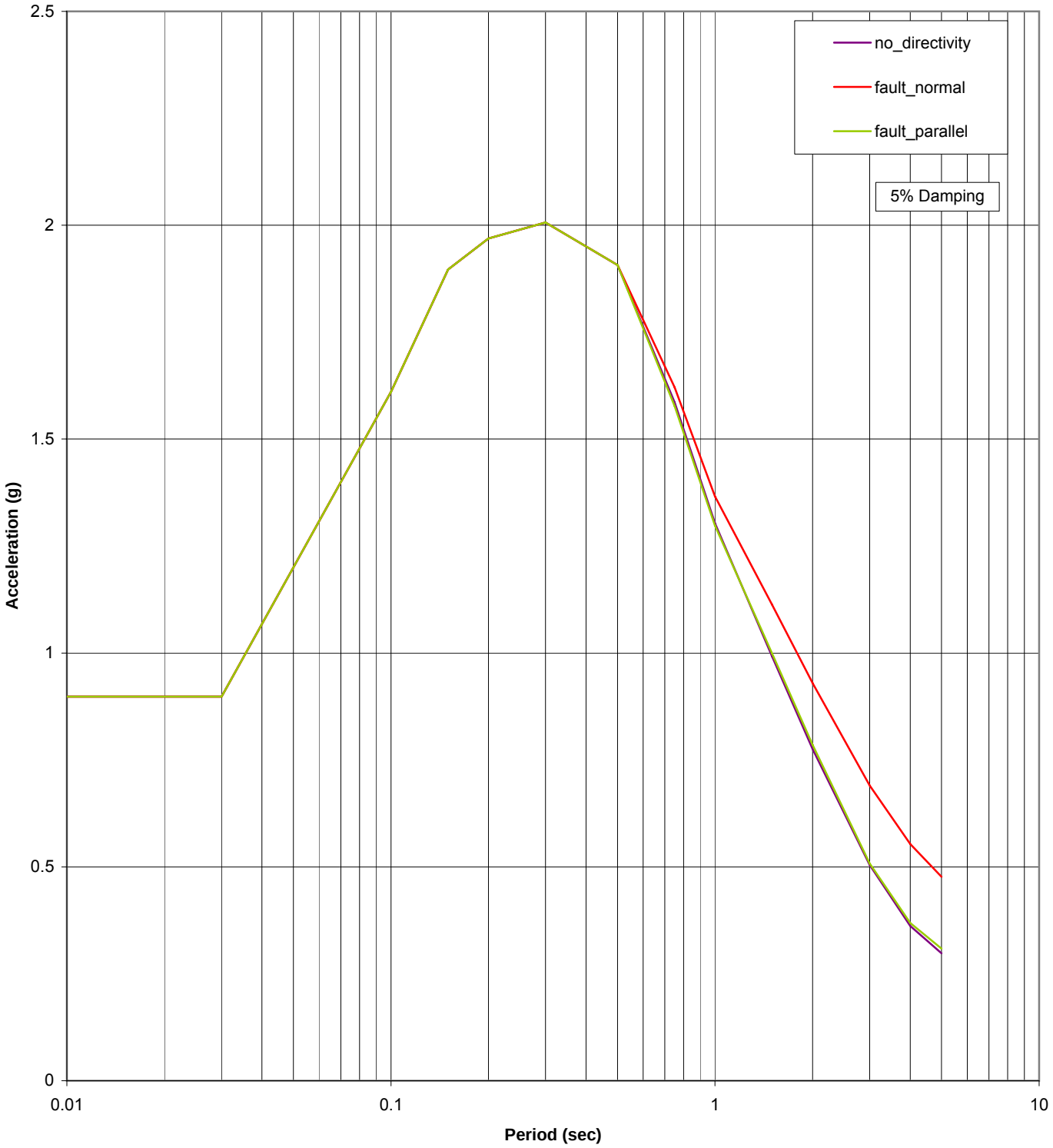
FIGURE C-1



2475-YEAR SPECTRA FROM DIFFERENT ATTENUATION RELATIONSHIPS

New Hospital Replacement Project
Oakland, California

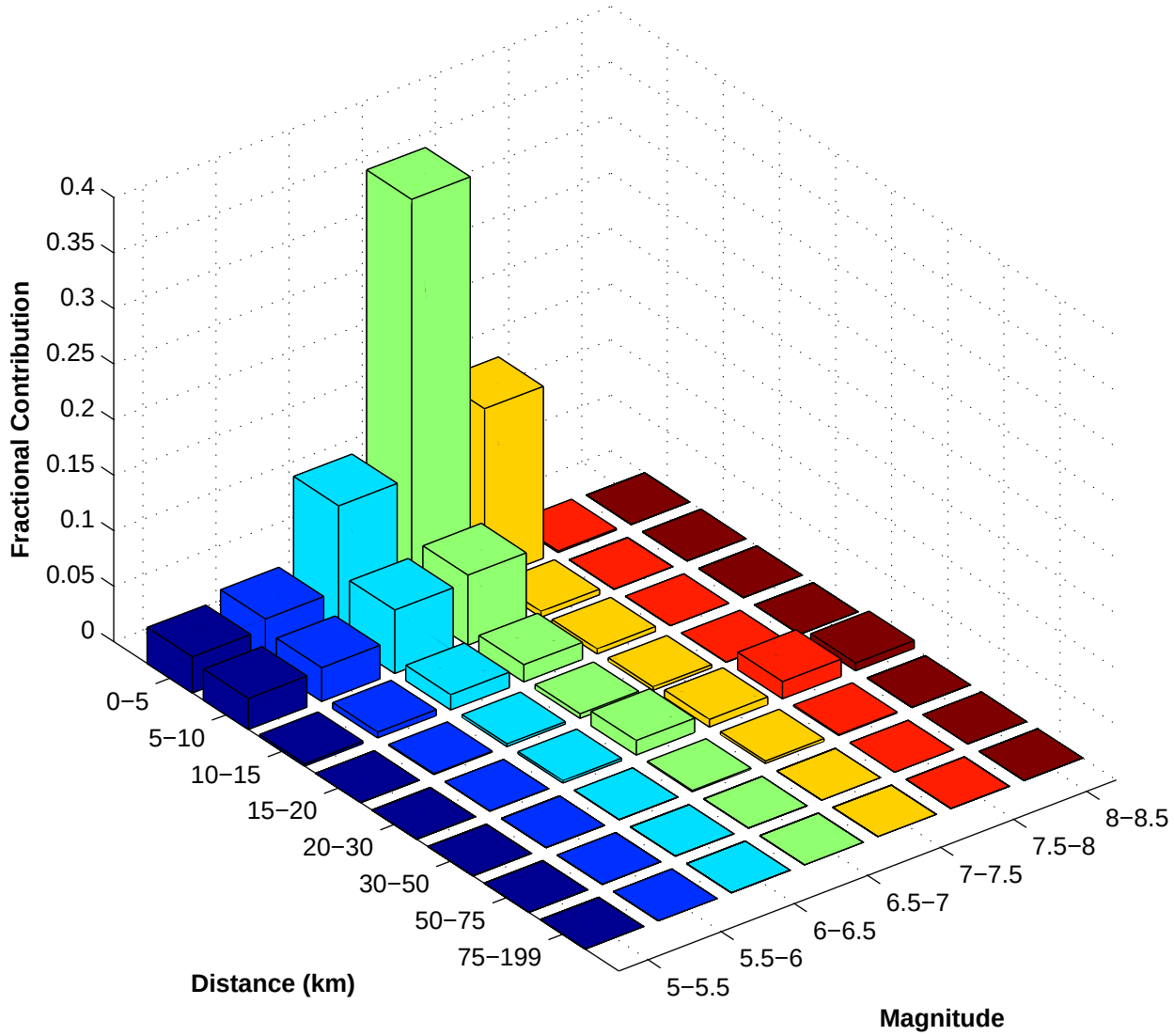
FIGURE C-2



ACCELERATION RESPONSE SPECTRUM FOR 2475-YEAR RETURN PERIOD

New Hospital Replacement Project
Oakland, California

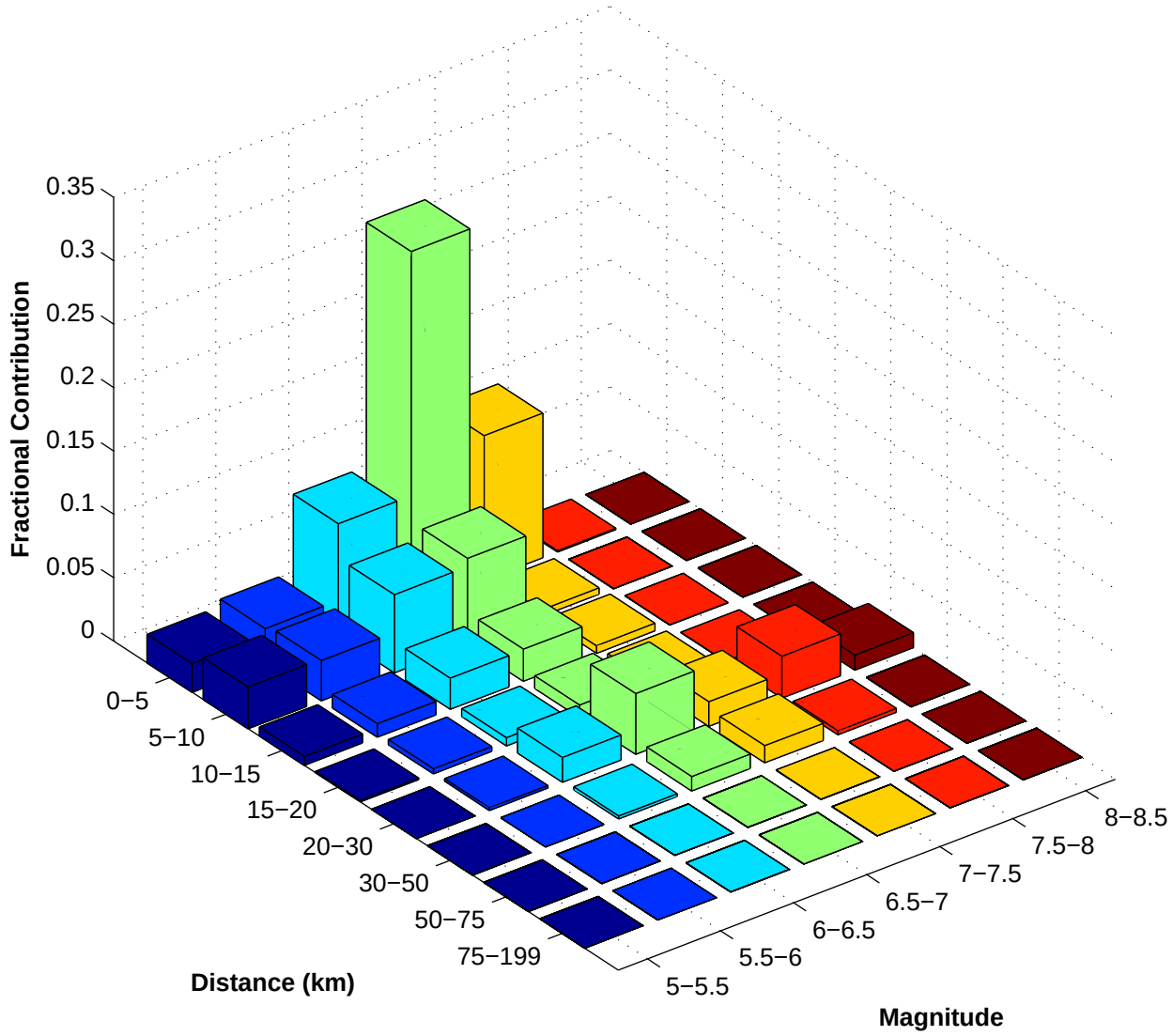
FIGURE C-3



DEAGGREGATION OF HAZARD AT PEAK GROUND ACCELERATION

New Hospital Replacement Project
Oakland, California

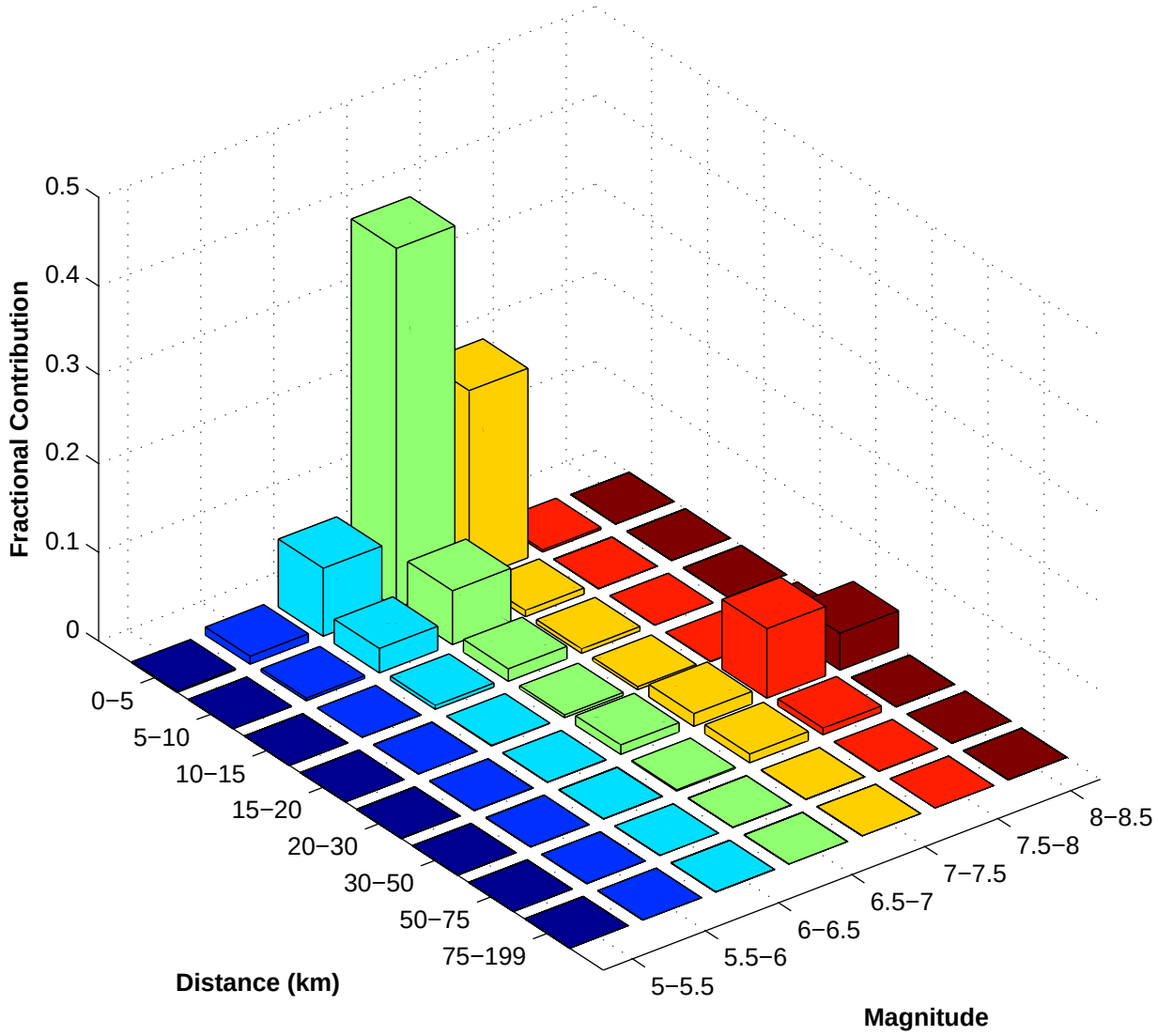
FIGURE C-4a



DEAGGREGATION OF HAZARD AT T = 0.2 SEC

New Hospital Replacement Project
Oakland, California

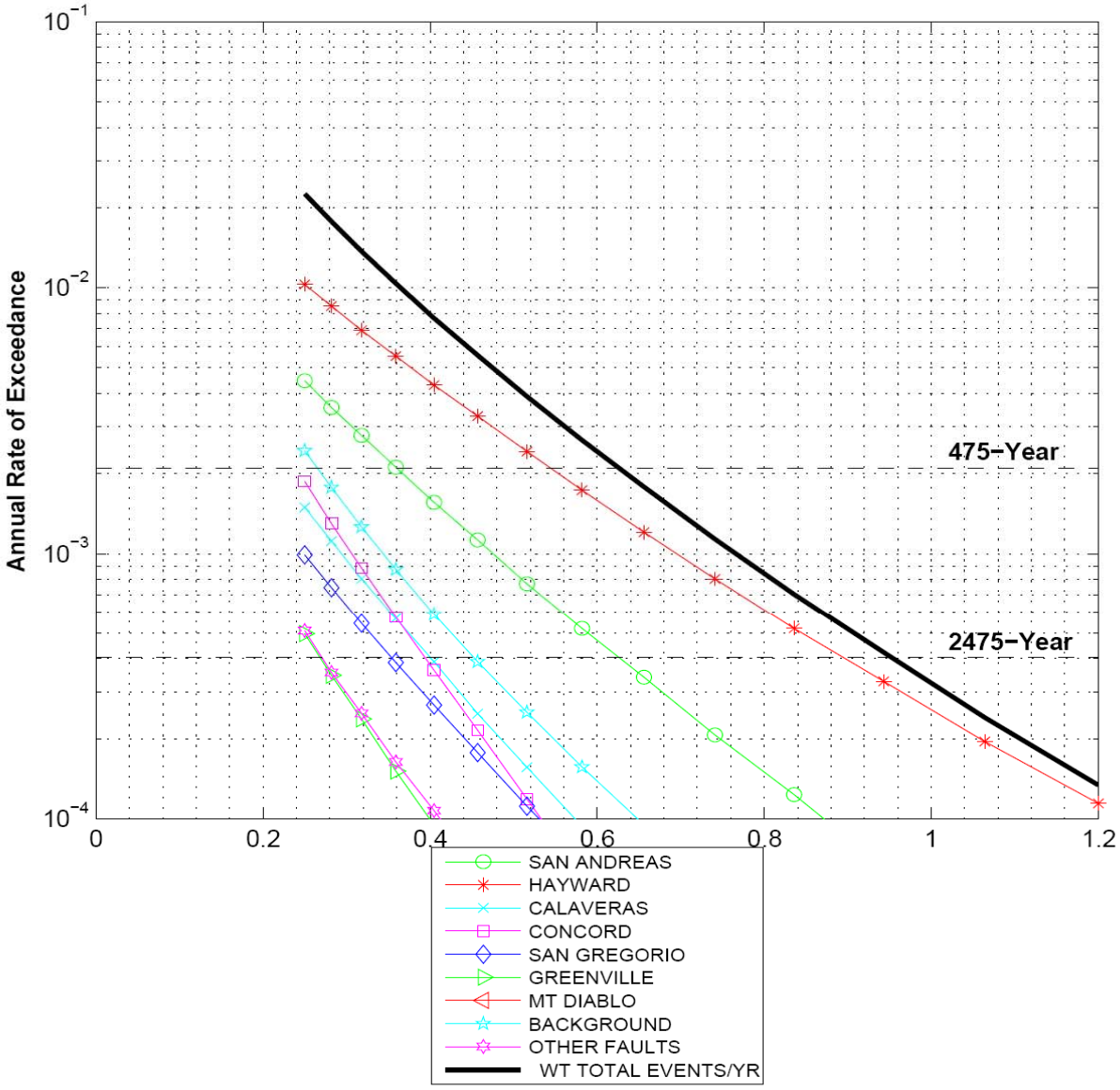
FIGURE C-4b



DEAGGREGATION OF HAZARD AT T = 1.0 SEC

New Hospital Replacement Project
Oakland, California

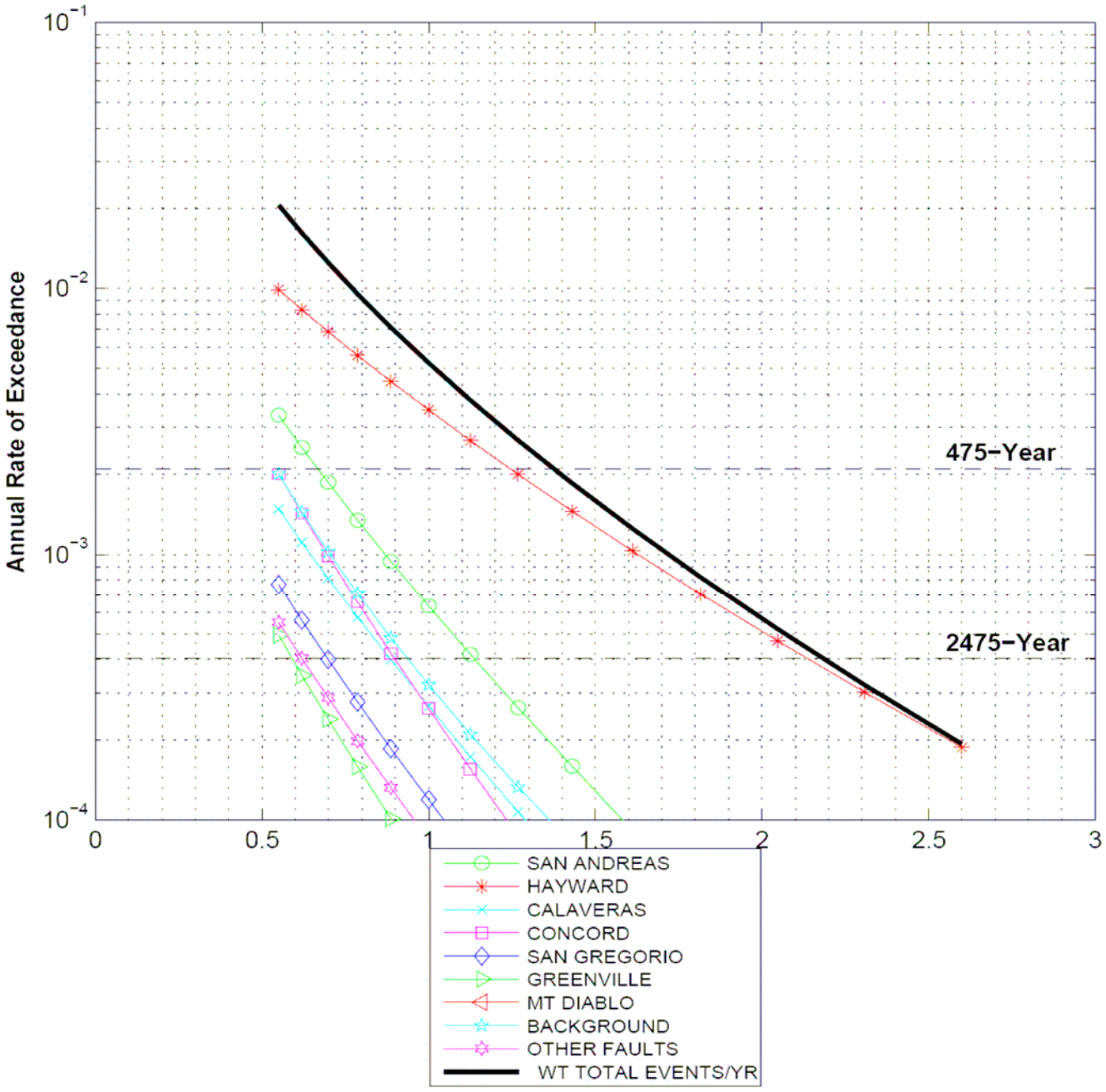
FIGURE C-4c



DEAGGREGATION OF HAZARD BY FAULT AT PEAK GROUND ACCELERATION

New Hospital Replacement Project
Oakland, California

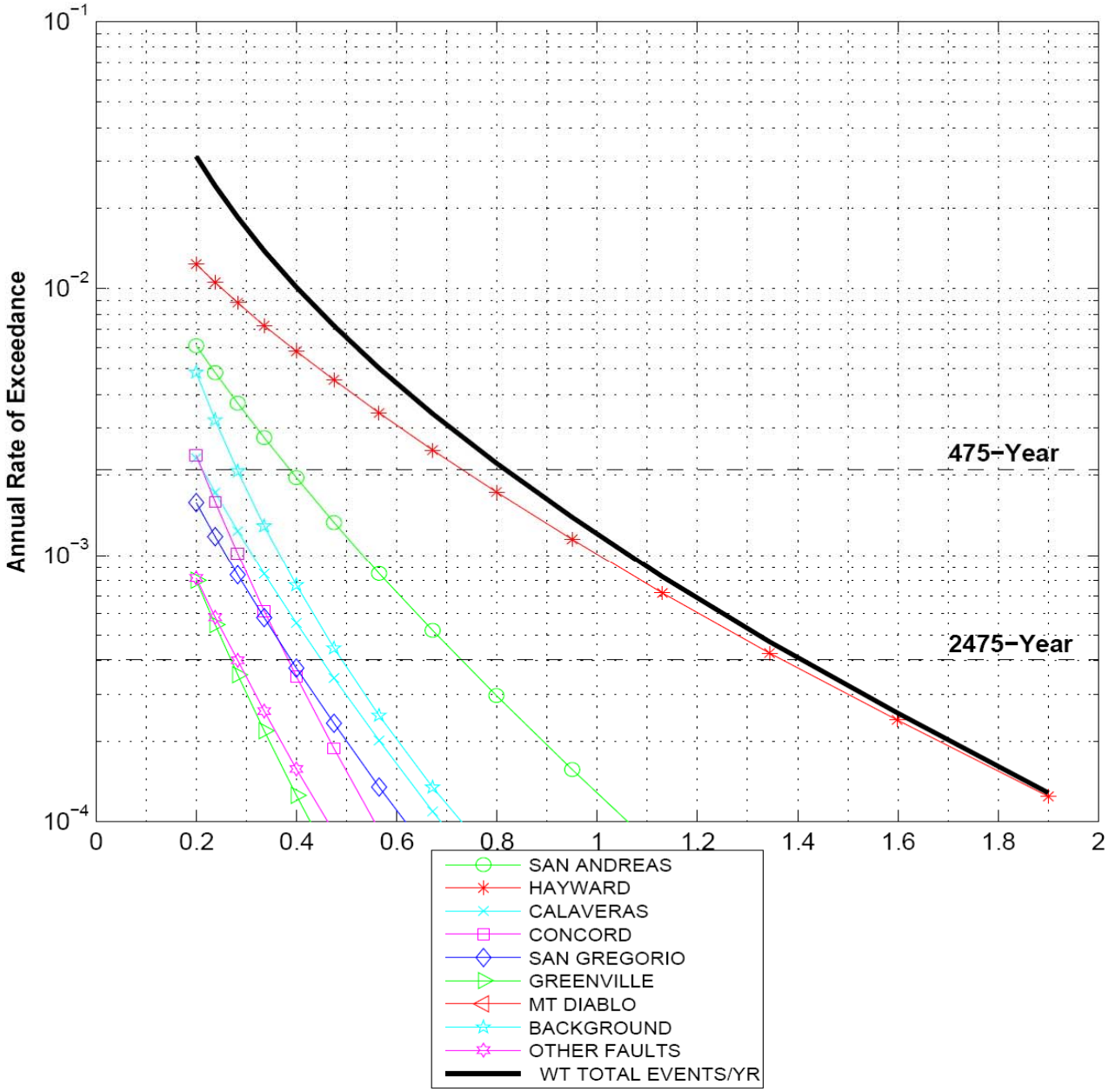
FIGURE C-5a



DEAGGREGATION OF HAZARD BY FAULT AT T = 0.2 SEC

New Hospital Replacement Project
Oakland, California

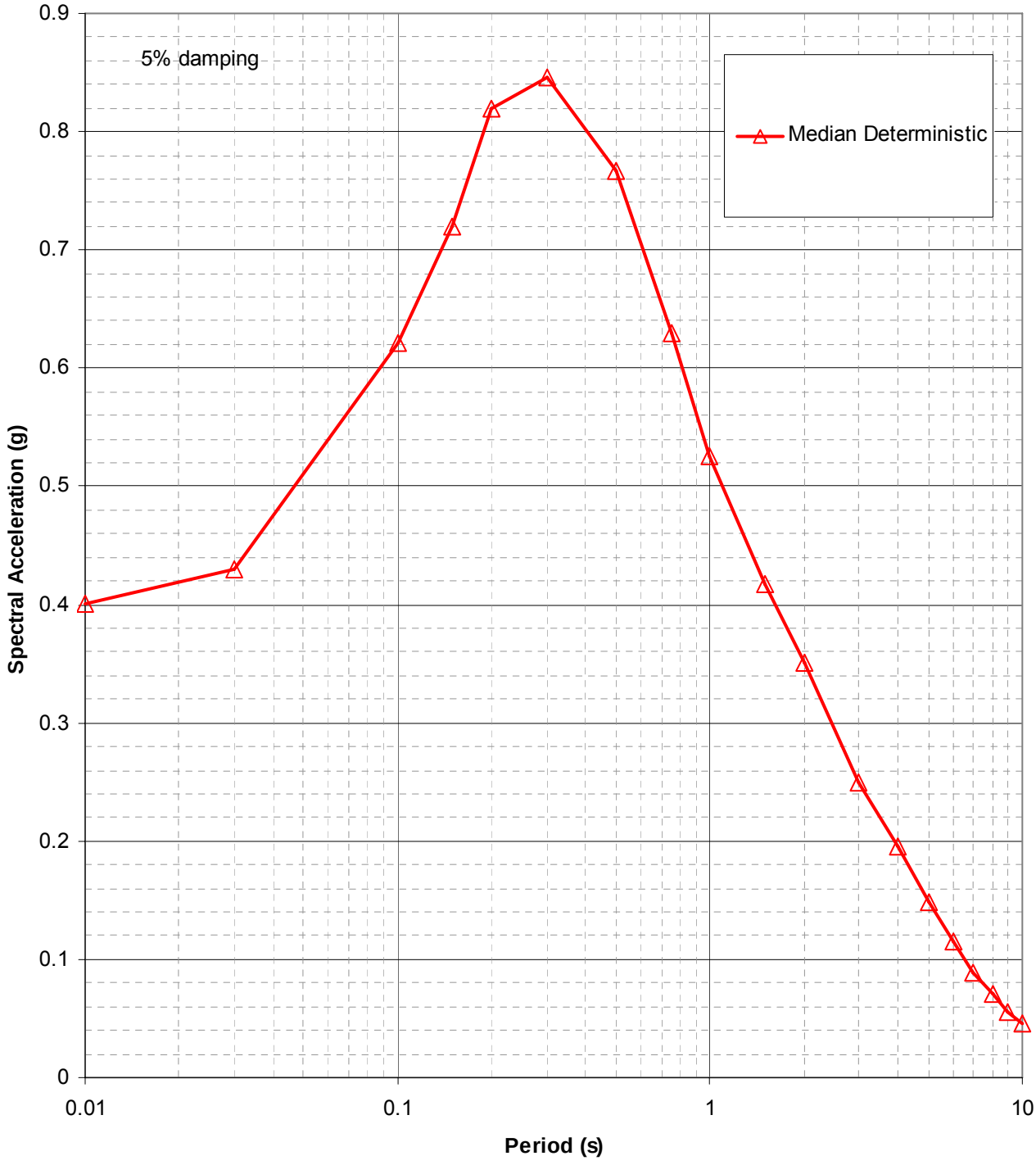
FIGURE C-5b



DEAGGREGATION OF HAZARD BY FAULT AT T = 1.0 SEC

New Hospital Replacement Project
Oakland, California

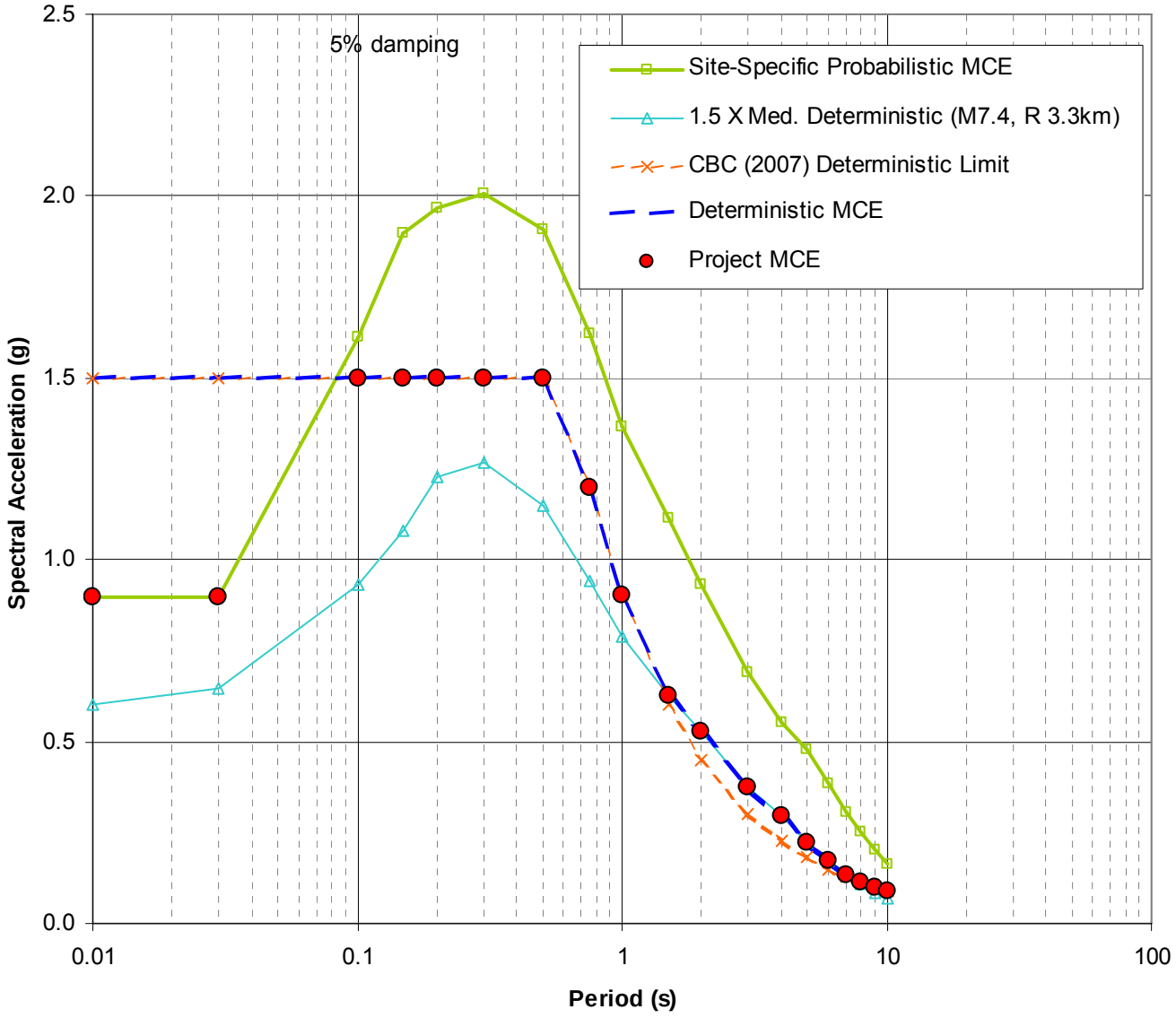
FIGURE C-5c



DETERMINISTIC SPECTRUM FOR FAULT NORMAL COMPONENT

New Hospital Replacement Project
Oakland, California

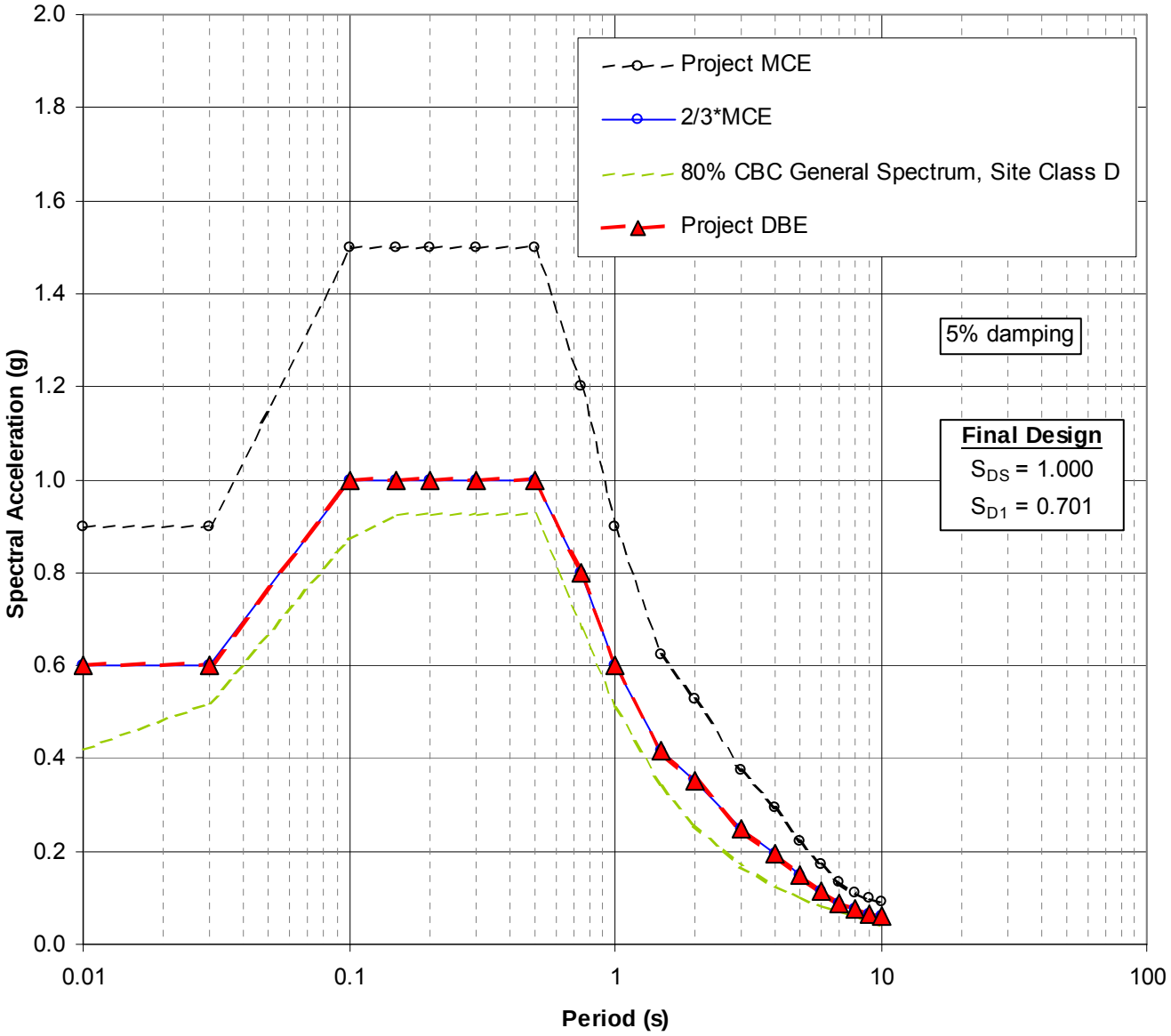
FIGURE C-6



DEVELOPMENT OF MCE SPECTRUM PER CBC 2007

New Hospital Replacement Project
Oakland, California

FIGURE C-7a



DEVELOPMENT OF DBE SPECTRUM PER CBC 2007

New Hospital Replacement Project
Oakland, California

FIGURE C-7b

APPENDIX D
LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

APPENDIX D
LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

INTRODUCTION

This appendix presents a summary of the procedures used to evaluate the potential for liquefaction of soil at the site. The liquefaction evaluations at the site involved the following steps:

1. Evaluating the generalized soil descriptions and laboratory index test results to identify layers that may be potentially liquefiable;
2. Estimating the seismically induced cyclic demands on the potentially liquefiable soil layers;
3. Estimating the liquefaction resistance of the potentially liquefiable soil layers from boring data;
4. Calculating the factor of safety against liquefaction for the potentially liquefiable layers; and
5. Estimating the magnitude of liquefaction induced settlement in layers with a factor of safety less than 1.1

EVALUATION OF POTENTIALLY LIQUEFIABLE SOIL LAYERS

The susceptibility of a granular soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in: a) distribution of grain size, b) soil density, c) aging and cementation, d) fines content, and e) plasticity characteristics of the fines.

Potentially liquefiable layers are identified as relatively loose, submerged, granular sediments. Liquefaction potential is believed to decrease with depth, and current procedures are believed to be overly conservative for depths greater than 20 meters (Davis and Berrill, 1982). Furthermore, aging tends to increase the liquefaction resistance of soil deposits, and Pleistocene soils are considered to have an extremely low potential for liquefaction.

Cohesive materials may also have the potential to liquefy. Generally, if the liquid limit of the cohesive materials is greater than 35 or the soil meets one of the other criteria specified in Seed, et al. (2001), the cohesive layer is not considered to be liquefiable.

Evaluation of the liquefaction susceptibility of coarse-grained layers and deposits was based on the empirical procedure recommended by the National Center for Earthquake Engineering Research (NCEER, 1997) and summarized by Youd et. al. (2001). In the NCEER procedure, potentially liquefiable soil strata are identified as those layers that are relatively loose, submerged granular sediments. Soil strata above the assumed groundwater level were considered not susceptible to liquefaction. A design groundwater depth of 8 feet was used in our analyses.

NCEER recommendations were used to compute liquefaction susceptibility using boring SPT data. The available cyclic shear resistance of the potentially liquefiable strata encountered (based on the normalized SPT N-value) was compared to the estimated cyclic shear stress that would be induced by a given earthquake. The estimated factor of safety against liquefaction is the ratio of the available cyclic shear resistance to the induced cyclic shear stress.

The estimated factors of safety against liquefaction at various depths are a function of the estimated overburden pressure at that depth. In general, higher factors of safety against liquefaction are calculated for higher overburden pressures.

For use with the NCEER procedure, sample depths, fines contents, soil unit weights and field SPT N-values were interpreted and summarized from the borings. The SPT data were then normalized to an overburden pressure of 1 atmosphere and corrected for fines content. A hammer efficiency factor of 1.3 was used to correct the field SPT N-values because the SPT sampling procedure used an automatic trip hammer with a higher efficiency than a standard rope and cathead.

For this evaluation, sublayers were identified at each boring location. Typically, the overall stratigraphic description was obtained from the boring logs, and sublayers were assigned based on SPT blow counts and fines contents. Usually, one sublayer was created for each sample obtained in the boring and that layer was assigned the blow count associated with the sample. A fines content values was assigned to each of those sublayers on the basis of either laboratory tests performed on the sample or the nearest fines content measurement considered representative of the sublayer. Unless bounded by one of the stratigraphic layers shown on the boring logs or the estimated groundwater table, sublayer boundaries were selected midway between adjacent samples.

The seismic hazard analyses for the project (Appendix C) have defined the following design basis earthquake (DBE) values:

- A magnitude 7.4 event on the Hayward fault
- A peak horizontal ground acceleration (PGA) of 0.60g

SUMMARY OF RESULTS AND CONCLUSIONS

The subsurface soils encountered in the borings consist primarily of fine-grained sediment (clay and sandy clay), and have a low susceptibility to liquefaction and dynamic densification. The clayey sand layers contain a significant amount of fines and are typically dense with corrected SPT N-values, (N1)60, greater than 30. These clayey sand layers are unlikely to liquefy during a design seismic event, based on our analyses. However, the liquefaction analyses did identify two isolated, thin layers (B-109 at the new hospital and B-112 at the CUP) that may be susceptible to liquefaction and liquefaction-induced settlement during a design basis earthquake (DBE) event. Our analyses also suggest that isolated liquefaction could occur at these two locations (B-109 and B-112) during a magnitude 7.4 earthquake with a

PGA greater than about 0.3g. The calculation spreadsheets for estimating the liquefaction potential based on the borings are attached in this appendix.

The potential for significant liquefaction and liquefaction-induced settlements at the site is considered low based on the evaluation of the boring data. The underlying soils are predominantly fine-grained and that the identified potentially liquefiable layers: 1) are anticipated to experience relatively minor strains (1 inch or less), 2) do not appear to be laterally extensive layers, and 3) are limited in thickness.

GROUND SETTLEMENT FROM EARTHQUAKE SHAKING

Ground settlement (and other ground effects) during seismic shaking can be caused by liquefaction, lateral spreading and compression of loose fill with voids, rubble, animal burrows or other similar conditions. Settlement from liquefaction or in loose, dry fill can be estimated using empirical procedures. These estimates are based on level ground conditions and are described below. Earthquake-induced settlement can cause downdrag on deep foundations, distress to pavements, gaps between ground supported pavements/structures and deep supported structures and damage to structures founded on shallow foundations.

Liquefaction Induced Settlement

Liquefaction-induced settlements were estimated using the procedure of Tokimatsu and Seed (1984). Using this procedure, volumetric strains were estimated that could occur within submerged layers from dissipation of seismically induced pore pressures. The estimated volumetric strains were then used in conjunction with the corresponding layer thicknesses to evaluate the magnitude of seismically induced settlements at the boring locations. The estimated settlements for the design earthquake (PGA = 0.60g, Magnitude = 7.4) are about ½ inch in B-109 and about 1 inch in B-112. Differential settlement should be anticipated on the same order of magnitude as the total settlements above because these potentially liquefiable materials were identified as isolated layers within a dense layer with a low potential for liquefaction.

Seismically Induced Settlement of Dry Fill

Earthquake shaking can also result in seismically induced settlement of relatively loose, dry, granular materials. However, the upper fill is generally cohesive and stiff and therefore settlement of the dry fill is only expected to occur in areas where voids exist.

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-101
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	13	90	25	N/A	0.0
2	Lean CLAY (CL)	7	125	16	90	20	N/A	0.0
3	Lean CLAY (CL)	8	125	16	90	20	N/A	0.0
4	Clayey SAND (SC)	12	125	49	18		N/A	0.0
5	Clayey SAND (SC)	17	130	32	18		N/A	0.0
6	Clayey SAND (SC)	23	125	29	19		N/A	0.0
7	Lean CLAY (CL)	29	125	20	90	20	N/A	0.0
8	Lean CLAY (CL)	33					N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-101

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	438	438	0.39	31	1.05	0.37		N/A	0.99	169
2	Lean CLAY (CL)	7.0	938	938	0.38	32	1.05	0.36		N/A	0.98	359
3	Lean CLAY (CL)	8.0	1250	1125	0.42	30	1.05	0.40		N/A	0.98	476
4	Clayey SAND (SC)	12.0	1813	1407	0.49	72	1.05	0.46		N/A	0.97	683
5	Clayey SAND (SC)	17.0	2515	1766	0.53	48	1.05	0.50		N/A	0.95	935
6	Clayey SAND (SC)	23.0	3280	2157	0.56	40	1.05	0.53		N/A	0.94	1202
7	Lean CLAY (CL)	29.0	3905	2470	0.57	32	1.05	0.54		N/A	0.92	1404
8	Lean CLAY (CL)	33.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	284	0.14	523	1.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	609	0.30	777	2.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
3	Lean CLAY (CL)	731	0.37	830	2.9E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	Clayey SAND (SC)	914	0.46	1241	2.8E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	Clayey SAND (SC)	1148	0.57	1216	3.8E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	Clayey SAND (SC)	1402	0.70	1269	4.7E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	Lean CLAY (CL)	1605	0.80	1253	5.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	Lean CLAY (CL)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-101

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	7.0	3.5	125.0	13	90	438	0	438	1.70	1.3	1.00	0.75	1.00	22	5.00	1.20	31	N/A	N/A	0.99	0.39	1.05	N/A
2	Lean CLAY (CL)	7.0	1.0	7.5	125.0	16	90	938	0	938	1.46	1.3	1.00	0.75	1.00	23	5.00	1.20	32	N/A	N/A	0.98	0.38	1.05	N/A
3	Lean CLAY (CL)	8.0	4.0	10.0	125.0	16	90	1250	125	1125	1.33	1.3	1.00	0.75	1.00	21	5.00	1.20	30	0.46	1.39	0.98	0.42	1.05	N/A
4	Clayey SAND (SC)	12.0	5.0	14.5	125.0	49	18	1813	406	1407	1.19	1.3	1.00	0.85	1.00	65	3.23	1.07	72	N/A	N/A	0.97	0.49	1.05	N/A
5	Clayey SAND (SC)	17.0	6.0	20.0	130.0	32	18	2515	749	1766	1.06	1.3	1.00	0.95	1.00	42	3.23	1.07	48	N/A	N/A	0.95	0.53	1.05	N/A
6	Clayey SAND (SC)	23.0	6.0	26.0	125.0	29	19	3280	1123	2157	0.96	1.3	1.00	0.95	1.00	34	3.43	1.07	40	N/A	N/A	0.94	0.56	1.05	N/A
7	Lean CLAY (CL)	29.0	4.0	31.0	125.0	20	90	3905	1435	2470	0.90	1.3	1.00	0.95	1.00	22	5.00	1.20	32	N/A	N/A	0.92	0.57	1.05	N/A
8	Lean CLAY (CL)	33.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7 (N1)60cs = a + b(N1)60

FC ≤ 5	a = 0
5 < FC < 35	a = e^(1.76 - (190/FC^2))
FC ≥ 35	a = 5.0
FC ≤ 5	b = 1.0
5 < FC < 35	b = (0.99 + (FC^1.5/1000))
FC ≥ 35	b = 1.2

*8

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-101

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-102
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - CLAY (CL)	0	125	13	90	25	N/A	0.0
2	Lean CLAY (CL)	6	125	14	90	21	N/A	0.0
3	Lean CLAY (CL)	8	125	13	90	21	N/A	0.0
4	Lean CLAY (CL)	13	125	15	90	21	N/A	0.0
5	Lean CLAY (CL)	18	125	21	90	21	N/A	0.0
6	Sandy CLAY (CL)	24	125	13	55	11	N/A	0.0
7	Sandy CLAY (CL)	29	125	21	55	11	N/A	0.0
8	Sandy CLAY (CL)	32					N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-102

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - CLAY (CL)	0.0	375	375	0.39	31	1.05	0.37		N/A	0.99	145
2	Lean CLAY (CL)	6.0	875	875	0.38	30	1.05	0.37		N/A	0.98	336
3	Lean CLAY (CL)	8.0	1313	1157	0.43	25	1.05	0.41		N/A	0.98	499
4	Lean CLAY (CL)	13.0	1938	1470	0.50	28	1.05	0.47		N/A	0.96	728
5	Lean CLAY (CL)	18.0	2625	1814	0.54	38	1.05	0.51		N/A	0.95	974
6	Sandy CLAY (CL)	24.0	3313	2158	0.56	24	1.05	0.53		N/A	0.94	1212
7	Sandy CLAY (CL)	29.0	3813	2409	0.57	33	1.05	0.54		N/A	0.93	1377
8	Sandy CLAY (CL)	32.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - CLAY (CL)	244	0.12	484	1.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	569	0.28	731	2.3E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Lean CLAY (CL)	752	0.38	793	3.1E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	Lean CLAY (CL)	955	0.48	930	3.9E-04	0	0.0E+00		1.0	0.0	0.000	0.00
5	Lean CLAY (CL)	1179	0.59	1137	4.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	Sandy CLAY (CL)	1403	0.70	1062	5.7E-04	below water	#VALUE!	0	1.0	0.0	0.000	0.00
7	Sandy CLAY (CL)	1566	0.78	1259	5.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	Sandy CLAY (CL)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-102

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - CLAY (CL)	0.0	6.0	3.0	125.0	13	90	375	0	375	1.70	1.3	1.00	0.75	1.00	22	5.00	1.20	31	N/A	N/A	0.99	0.39	1.05	N/A
2	Lean CLAY (CL)	6.0	2.0	7.0	125.0	14	90	875	0	875	1.51	1.3	1.00	0.75	1.00	21	5.00	1.20	30	0.45	1.36	0.98	0.38	1.05	N/A
3	Lean CLAY (CL)	8.0	5.0	10.5	125.0	13	90	1313	156	1157	1.32	1.3	1.00	0.75	1.00	17	5.00	1.20	25	0.29	0.88	0.98	0.43	1.05	N/A
4	Lean CLAY (CL)	13.0	5.0	15.5	125.0	15	90	1938	468	1470	1.17	1.3	1.00	0.85	1.00	19	5.00	1.20	28	0.38	1.13	0.96	0.50	1.05	N/A
5	Lean CLAY (CL)	18.0	6.0	21.0	125.0	21	90	2625	811	1814	1.05	1.3	1.00	0.95	1.00	27	5.00	1.20	38	N/A	N/A	0.95	0.54	1.05	N/A
6	Sandy CLAY (CL)	24.0	5.0	26.5	125.0	13	55	3313	1154	2158	0.96	1.3	1.00	0.95	1.00	15	5.00	1.20	24	0.27	0.27	0.94	0.56	1.05	N/A
7	Sandy CLAY (CL)	29.0	3.0	30.5	125.0	21	55	3813	1404	2409	0.91	1.3	1.00	0.95	1.00	24	5.00	1.20	33	N/A	N/A	0.93	0.57	1.05	N/A
8	Sandy CLAY (CL)	32.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8 $$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-102

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-103
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - CLAY (CL)	0	125	11	90	25	N/A	0.0
2	Lean CLAY (CL)	5	125	18	90	16	N/A	0.0
3	Lean CLAY (CL)	8	125	22	90	16	N/A	0.0
4	clayey SAND (SC)	16	125	27	35		N/A	0.0
5	clayey SAND (SC)	22	125	21	22		N/A	0.0
6	clayey SAND (SC)	25	125	31	27		N/A	0.0
7	clayey SAND (SC)	27	125	42	27		N/A	0.0
8	clayey SAND (SC)	35					N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-103

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - CLAY (CL)	0.0	313	313	0.39	27	1.05	0.37		N/A	0.99	121
2	Lean CLAY (CL)	5.0	813	813	0.38	38	1.05	0.37		N/A	0.98	312
3	Lean CLAY (CL)	8.0	1500	1250	0.45	38	1.05	0.43		N/A	0.97	569
4	clayey SAND (SC)	16.0	2375	1689	0.52	44	1.05	0.50		N/A	0.96	885
5	clayey SAND (SC)	22.0	2938	1970	0.55	32	1.05	0.52		N/A	0.95	1083
6	clayey SAND (SC)	25.0	3250	2127	0.56	46	1.05	0.53		N/A	0.94	1191
7	clayey SAND (SC)	27.0	3875	2440	0.57	58	1.05	0.54		N/A	0.92	1393
8	clayey SAND (SC)	35.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - CLAY (CL)	203	0.10	422	1.4E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	528	0.26	764	2.0E-04	N/A	N/A		1.0	0.0	0.000	0.00
3	Lean CLAY (CL)	813	0.41	943	3.0E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	clayey SAND (SC)	1098	0.55	1155	3.8E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	clayey SAND (SC)	1281	0.64	1129	4.8E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	clayey SAND (SC)	1382	0.69	1319	4.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	clayey SAND (SC)	1586	0.79	1517	4.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-103

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - CLAY (CL)	0.0	5.0	2.5	125.0	11	90	313	0	313	1.70	1.3	1.00	0.75	1.00	18	5.00	1.20	27	0.34	1.01	0.99	0.39	1.05	N/A
2	Lean CLAY (CL)	5.0	3.0	6.5	125.0	18	90	813	0	813	1.57	1.3	1.00	0.75	1.00	28	5.00	1.20	38	N/A	N/A	0.98	0.38	1.05	N/A
3	Lean CLAY (CL)	8.0	8.0	12.0	125.0	22	90	1500	250	1250	1.26	1.3	1.00	0.75	1.00	27	5.00	1.20	38	N/A	N/A	0.97	0.45	1.05	N/A
4	clayey SAND (SC)	16.0	6.0	19.0	125.0	27	35	2375	686	1689	1.09	1.3	1.00	0.85	1.00	32	5.00	1.20	44	N/A	N/A	0.96	0.52	1.05	N/A
5	clayey SAND (SC)	22.0	3.0	23.5	125.0	21	22	2938	967	1970	1.01	1.3	1.00	0.95	1.00	26	3.93	1.09	32	N/A	N/A	0.95	0.55	1.05	N/A
6	clayey SAND (SC)	25.0	2.0	26.0	125.0	31	27	3250	1123	2127	0.97	1.3	1.00	0.95	1.00	37	4.48	1.13	46	N/A	N/A	0.94	0.56	1.05	N/A
7	clayey SAND (SC)	27.0	8.0	31.0	125.0	42	27	3875	1435	2440	0.91	1.3	1.00	0.95	1.00	47	4.48	1.13	58	N/A	N/A	0.92	0.57	1.05	N/A
8	clayey SAND (SC)	35.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-103

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-104
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	16	90	26	N/A	0.0
2	Lean CLAY (CL)	8	125	14	90	21	N/A	0.0
3	clayey SAND (SC)	14	125	25	46		N/A	0.0
4	clayey SAND (SC)	18	125	38	46		N/A	0.0
5	clayey SAND (SC)	23	125	34	46		N/A	0.0
6	Clay with Sand (CL)	28	125	38	65		N/A	0.0
7	Sand (SP)	34	125	40	5		N/A	0.0
8	gravelly CLAY (CL)	36	125	40	65		N/A	0.0
9	Sandy CLAY (CL)	40	125	50	65		N/A	0.0
10	Sandy CLAY (CL)	50					N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-104

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	500	500	0.39	37	1.05	0.37		N/A	0.99	193
2	Lean CLAY (CL)	8.0	1375	1188	0.44	26	1.05	0.42		N/A	0.97	522
3	clayey SAND (SC)	14.0	2000	1501	0.50	43	1.05	0.48		N/A	0.96	751
4	clayey SAND (SC)	18.0	2563	1783	0.53	65	1.05	0.51		N/A	0.95	952
5	clayey SAND (SC)	23.0	3188	2096	0.56	54	1.05	0.53		N/A	0.94	1169
6	Clay with Sand (CL)	28.0	3875	2440	0.57	56	1.05	0.54		N/A	0.92	1393
7	Sand (SP)	34.0	4375	2690	0.56	45	1.05	0.54		N/A	0.89	1517
8	gravelly CLAY (CL)	36.0	4750	2878	0.56	57	1.05	0.53		N/A	0.86	1602
9	Sandy CLAY (CL)	40.0	5625	3316	0.53	66	1.05	0.509		N/A	0.81	1772
10	Sandy CLAY (CL)	50.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	325	0.16	593	1.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	772	0.39	817	3.2E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	clayey SAND (SC)	976	0.49	1083	3.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	clayey SAND (SC)	1159	0.58	1347	3.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	clayey SAND (SC)	1362	0.68	1378	4.2E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	Clay with Sand (CL)	1586	0.79	1503	4.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	Sand (SP)	1749	0.87	1467	5.2E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	gravelly CLAY (CL)	1871	0.94	1642	4.9E-04	N/A	N/A		1.0	0.0	0.000	0.00
9	Sandy CLAY (CL)	2156	1.08	1846	4.8E-04	N/A	N/A		1.0	0.0	0.000	0.00
10	Sandy CLAY (CL)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-104

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	8.0	4.0	125.0	16	90	500	0	500	1.70	1.3	1.00	0.75	1.00	27	5.00	1.20	37	N/A	N/A	0.99	0.39	1.05	N/A
2	Lean CLAY (CL)	8.0	6.0	11.0	125.0	14	90	1375	187	1188	1.30	1.3	1.00	0.75	1.00	18	5.00	1.20	26	0.32	0.96	0.97	0.44	1.05	N/A
3	clayey SAND (SC)	14.0	4.0	16.0	125.0	25	46	2000	499	1501	1.15	1.3	1.00	0.85	1.00	32	5.00	1.20	43	N/A	N/A	0.96	0.50	1.05	N/A
4	clayey SAND (SC)	18.0	5.0	20.5	125.0	38	46	2563	780	1783	1.06	1.3	1.00	0.95	1.00	50	5.00	1.20	65	N/A	N/A	0.95	0.53	1.05	N/A
5	clayey SAND (SC)	23.0	5.0	25.5	125.0	34	46	3188	1092	2096	0.98	1.3	1.00	0.95	1.00	41	5.00	1.20	54	N/A	N/A	0.94	0.56	1.05	N/A
6	Clay with Sand (CL)	28.0	6.0	31.0	125.0	38	65	3875	1435	2440	0.91	1.3	1.00	0.95	1.00	42	5.00	1.20	56	N/A	N/A	0.92	0.57	1.05	N/A
7	Sand (SP)	34.0	2.0	35.0	125.0	40	5	4375	1685	2690	0.86	1.3	1.00	1.00	1.00	45	0.00	1.00	45	N/A	N/A	0.89	0.56	1.05	N/A
8	gravelly CLAY (CL)	36.0	4.0	38.0	125.0	40	65	4750	1872	2878	0.83	1.3	1.00	1.00	1.00	43	5.00	1.20	57	N/A	N/A	0.86	0.56	1.05	N/A
9	Sandy CLAY (CL)	40.0	10.0	45.0	125.0	50	65	5625	2309	3316	0.78	1.3	1.00	1.00	1.00	50	5.00	1.20	66	N/A	N/A	0.81	0.53	1.05	N/A
10	Sandy CLAY (CL)	50.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8 $$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-104

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-105
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	14	90	26	N/A	0.0
2	Sandy CLAY (CL)	8	125	11	70	20	N/A	0.0
3	Sandy CLAY (CL)	10	125	13	70	20	N/A	0.0
4	Fat Clay (CH)	14	125	19	85	30	N/A	0.0
5	Lean Clay (CL)	21	125	27	70		N/A	0.0
6	gravelly CLAY (CL)	29	125	34	65		N/A	0.0
7	gravelly CLAY (CL)	36					N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-105

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	500	500	0.39	33	1.05	0.37		N/A	0.99	193
2	Sandy CLAY (CL)	8.0	1125	1063	0.40	23	1.05	0.38		N/A	0.98	430
3	Sandy CLAY (CL)	10.0	1500	1250	0.45	24	1.05	0.43		N/A	0.97	569
4	Fat Clay (CH)	14.0	2188	1595	0.51	33	1.05	0.49		N/A	0.96	818
5	Lean Clay (CL)	21.0	3125	2064	0.56	44	1.05	0.53		N/A	0.94	1148
6	gravelly CLAY (CL)	29.0	4063	2534	0.57	50	1.05	0.54		N/A	0.91	1441
7	gravelly CLAY (CL)	36.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
8	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	325	0.16	571	1.7E-04	N/A	N/A		1.0	0.0	0.000	0.00
2	Sandy CLAY (CL)	691	0.35	736	2.9E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Sandy CLAY (CL)	813	0.41	816	3.5E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	Fat Clay (CH)	1037	0.52	1023	4.0E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	Lean Clay (CL)	1342	0.67	1281	4.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	gravelly CLAY (CL)	1647	0.82	1473	4.9E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	gravelly CLAY (CL)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-105

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	8.0	4.0	125.0	14	90	500	0	500	1.70	1.3	1.00	0.75	1.00	23	5.00	1.20	33	N/A	N/A	0.99	0.39	1.05	N/A
2	Sandy CLAY (CL)	8.0	2.0	9.0	125.0	11	70	1125	62	1063	1.37	1.3	1.00	0.75	1.00	15	5.00	1.20	23	0.25	0.75	0.98	0.40	1.05	N/A
3	Sandy CLAY (CL)	10.0	4.0	12.0	125.0	13	70	1500	250	1250	1.26	1.3	1.00	0.75	1.00	16	5.00	1.20	24	0.28	0.83	0.97	0.45	1.05	N/A
4	Fat Clay (CH)	14.0	7.0	17.5	125.0	19	85	2188	593	1595	1.12	1.3	1.00	0.85	1.00	24	5.00	1.20	33	N/A	N/A	0.96	0.51	1.05	N/A
5	Lean Clay (CL)	21.0	8.0	25.0	125.0	27	70	3125	1061	2064	0.98	1.3	1.00	0.95	1.00	33	5.00	1.20	44	N/A	N/A	0.94	0.56	1.05	N/A
6	gravelly CLAY (CL)	29.0	7.0	32.5	125.0	34	65	4063	1529	2534	0.89	1.3	1.00	0.95	1.00	37	5.00	1.20	50	N/A	N/A	0.91	0.57	1.05	N/A
7	gravelly CLAY (CL)	36.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
8	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8 $$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-105

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-106
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	11	90	26	N/A	0.0
2	Lean Clay (CL)	6	125	18	90	20	N/A	0.0
3	Lean Clay (CL)	8	125	9	90	20	N/A	0.0
4	Lean Clay (CL)	13	125	15	90	20	N/A	0.0
5	Lean Clay (CL)	18	125	12	90	20	N/A	0.0
6	Sandy CLAY (CL)	23	125	23	70		N/A	0.0
7	Sand w/ Gravel (SW)	26	125	45	10		N/A	0.0
8	Clayey Gravel (GC)	33	125	27	33		N/A	0.0
9	Clayey Gravel (GC)	42	125	36	33		N/A	0.0
10	lean CLAY w/ Sand	46	125	36	65		N/A	0.0
11	Lean Clay (CL)	51	125	25	90		N/A	0.0
12	Lean Clay (CL)	61	125	34	90		N/A	0.0
13	Lean Clay (CL)	71	125	35	90		N/A	0.0
14	Lean Clay (CL)	81	125				N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-106

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	375	375	0.39	27	1.05	0.37		N/A	0.99	145
2	Lean Clay (CL)	6.0	875	875	0.38	37	1.05	0.37		N/A	0.98	336
3	Lean Clay (CL)	8.0	1313	1157	0.43	19	1.05	0.41		N/A	0.98	499
4	Lean Clay (CL)	13.0	1938	1470	0.50	28	1.05	0.47		N/A	0.96	728
5	Lean Clay (CL)	18.0	2563	1783	0.53	24	1.05	0.51		N/A	0.95	952
6	Sandy CLAY (CL)	23.0	3063	2033	0.55	39	1.05	0.53		N/A	0.94	1126
7	Sand w/ Gravel (SW)	26.0	3688	2346	0.57	53	1.05	0.54		N/A	0.93	1339
8	Clayey Gravel (GC)	33.0	4688	2847	0.56	40	1.05	0.53		N/A	0.87	1588
9	Clayey Gravel (GC)	42.0	5500	3254	0.54	48	1.05	0.512		N/A	0.82	1750
10	lean CLAY w/ Sand	46.0	6063	3535	0.52	47	1.05	0.496		N/A	0.78	1843
11	Lean Clay (CL)	51.0	7000	4005	0.49	33	1.05	0.466		N/A	0.72	1961
12	Lean Clay (CL)	61.0	8250	4631	0.44	40	1.05	0.421		N/A	0.64	2049
13	Lean Clay (CL)	71.0	9500	5257	0.39	39	1.05	0.373		N/A	0.56	2058
14	Lean Clay (CL)	81.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	244	0.12	463	1.6E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	Lean Clay (CL)	569	0.28	784	2.1E-04	N/A	N/A		1.0	0.0	0.000	0.00
3	Lean Clay (CL)	752	0.38	723	3.5E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	Lean Clay (CL)	955	0.48	930	3.9E-04	0	0.0E+00		1.0	0.0	0.000	0.00
5	Lean Clay (CL)	1159	0.58	969	4.9E-04	0	0.0E+00		1.0	0.0	0.000	0.00
6	Sandy CLAY (CL)	1321	0.66	1216	4.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	Sand w/ Gravel (SW)	1525	0.76	1450	4.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	Clayey Gravel (GC)	1850	0.93	1448	5.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
9	Clayey Gravel (GC)	2115	1.06	1652	5.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
10	lean CLAY w/ Sand	2298	1.15	1711	5.4E-04	N/A	N/A		1.0	0.0	0.000	0.00
11	Lean Clay (CL)	2603	1.30	1610	6.1E-04	N/A	N/A		1.0	0.0	0.000	0.00
12	Lean Clay (CL)	3010	1.51	1851	5.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
13	Lean Clay (CL)	3417	1.71	1953	5.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
14	Lean Clay (CL)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-106

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	6.0	3.0	125.0	11	90	375	0	375	1.70	1.3	1.00	0.75	1.00	18	5.00	1.20	27	0.34	1.01	0.99	0.39	1.05	N/A
2	Lean Clay (CL)	6.0	2.0	7.0	125.0	18	90	875	0	875	1.51	1.3	1.00	0.75	1.00	27	5.00	1.20	37	N/A	N/A	0.98	0.38	1.05	N/A
3	Lean Clay (CL)	8.0	5.0	10.5	125.0	9	90	1313	156	1157	1.32	1.3	1.00	0.75	1.00	12	5.00	1.20	19	0.20	0.60	0.98	0.43	1.05	N/A
4	Lean Clay (CL)	13.0	5.0	15.5	125.0	15	90	1938	468	1470	1.17	1.3	1.00	0.85	1.00	19	5.00	1.20	28	0.38	1.13	0.96	0.50	1.05	N/A
5	Lean Clay (CL)	18.0	5.0	20.5	125.0	12	90	2563	780	1783	1.06	1.3	1.00	0.95	1.00	16	5.00	1.20	24	0.27	0.81	0.95	0.53	1.05	N/A
6	Sandy CLAY (CL)	23.0	3.0	24.5	125.0	23	70	3063	1030	2033	0.99	1.3	1.00	0.95	1.00	28	5.00	1.20	39	N/A	N/A	0.94	0.55	1.05	N/A
7	Sand w/ Gravel (SW	26.0	7.0	29.5	125.0	45	10	3688	1342	2346	0.92	1.3	1.00	0.95	1.00	51	0.87	1.02	53	N/A	N/A	0.93	0.57	1.05	N/A
8	Clayey Gravel (GC)	33.0	9.0	37.5	125.0	27	33	4688	1841	2847	0.84	1.3	1.00	1.00	1.00	29	4.88	1.18	40	N/A	N/A	0.87	0.56	1.05	N/A
9	Clayey Gravel (GC)	42.0	4.0	44.0	125.0	36	33	5500	2246	3254	0.78	1.3	1.00	1.00	1.00	37	4.88	1.18	48	N/A	N/A	0.82	0.54	1.05	N/A
10	lean CLAY w/ Sand	46.0	5.0	48.5	125.0	36	65	6063	2527	3535	0.75	1.3	1.00	1.00	1.00	35	5.00	1.20	47	N/A	N/A	0.78	0.52	1.05	N/A
11	Lean Clay (CL)	51.0	10.0	56.0	125.0	25	90	7000	2995	4005	0.71	1.3	1.00	1.00	1.00	23	5.00	1.20	33	N/A	N/A	0.72	0.49	1.05	N/A
12	Lean Clay (CL)	61.0	10.0	66.0	125.0	34	90	8250	3619	4631	0.66	1.3	1.00	1.00	1.00	29	5.00	1.20	40	N/A	N/A	0.64	0.44	1.05	N/A
13	Lean Clay (CL)	71.0	10.0	76.0	125.0	35	90	9500	4243	5257	0.62	1.3	1.00	1.00	1.00	28	5.00	1.20	39	N/A	N/A	0.56	0.39	1.05	N/A
14	Lean Clay (CL)	81.0	N/A	N/A	125.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-106

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-107
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE

Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	17	80	26	N/A	0.0
2	Lean CLAY (CL)	8	125	16	85	16	N/A	0.0
3	Sandy CLAY (CL)	13	125	25	70	16	N/A	0.0
4	Clayey SAND (SC)	18	125	22	48		N/A	0.0
5	Clayey SAND (SC)	23	125	37	48		N/A	0.0
6	gravelly CLAY (CL)	28	125	41	75	20	N/A	0.0
7	gravelly CLAY (CL)	35					N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-107

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	500	500	0.39	39	1.05	0.37		N/A	0.99	193
2	Lean CLAY (CL)	8.0	1313	1157	0.43	30	1.05	0.41		N/A	0.98	499
3	Sandy CLAY (CL)	13.0	1938	1470	0.50	44	1.05	0.47		N/A	0.96	728
4	Clayey SAND (SC)	18.0	2563	1783	0.53	40	1.05	0.51		N/A	0.95	952
5	Clayey SAND (SC)	23.0	3188	2096	0.56	59	1.05	0.53		N/A	0.94	1169
6	gravelly CLAY (CL)	28.0	3938	2471	0.57	60	1.05	0.54		N/A	0.92	1409
7	gravelly CLAY (CL)	35.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
8	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	325	0.16	603	1.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	752	0.38	839	3.0E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Sandy CLAY (CL)	955	0.48	1075	3.4E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	Clayey SAND (SC)	1159	0.58	1145	4.2E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	Clayey SAND (SC)	1362	0.68	1414	4.1E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	gravelly CLAY (CL)	1606	0.80	1545	4.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	gravelly CLAY (CL)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-107

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	8.0	4.0	125.0	17	80	500	0	500	1.70	1.3	1.00	0.75	1.00	28	5.00	1.20	39	N/A	N/A	0.99	0.39	1.05	N/A
2	Lean CLAY (CL)	8.0	5.0	10.5	125.0	16	85	1313	156	1157	1.32	1.3	1.00	0.75	1.00	21	5.00	1.20	30	0.44	1.33	0.98	0.43	1.05	N/A
3	Sandy CLAY (CL)	13.0	5.0	15.5	125.0	25	70	1938	468	1470	1.17	1.3	1.00	0.85	1.00	32	5.00	1.20	44	N/A	N/A	0.96	0.50	1.05	N/A
4	Clayey SAND (SC)	18.0	5.0	20.5	125.0	22	48	2563	780	1783	1.06	1.3	1.00	0.95	1.00	29	5.00	1.20	40	N/A	N/A	0.95	0.53	1.05	N/A
5	Clayey SAND (SC)	23.0	5.0	25.5	125.0	37	48	3188	1092	2096	0.98	1.3	1.00	0.95	1.00	45	5.00	1.20	59	N/A	N/A	0.94	0.56	1.05	N/A
6	gravelly CLAY (CL)	28.0	7.0	31.5	125.0	41	75	3938	1466	2471	0.90	1.3	1.00	0.95	1.00	46	5.00	1.20	60	N/A	N/A	0.92	0.57	1.05	N/A
7	gravelly CLAY (CL)	35.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
8	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-107

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-108
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	8	80	22	N/A	0.0
2	Lean CLAY (CL)	8	125	11	90	28	N/A	0.0
3	Lean CLAY (CL)	15	125	22	90	28	N/A	0.0
4	Lean CLAY (CL)	20	125	13	90	28	N/A	0.0
5	clayey GRAVEL (GC)	25	125	18	44		N/A	0.0
6	clayey GRAVEL (GC)	30	125	21	44	30	N/A	0.0
7	clayey SAND (SC)	32	125	27	20	11	N/A	0.0
8	clayey SAND (SC)	36					N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-108

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	500	500	0.39	21	1.05	0.37		N/A	0.99	193
2	Lean CLAY (CL)	8.0	1438	1219	0.45	21	1.05	0.43		N/A	0.97	546
3	Lean CLAY (CL)	15.0	2188	1595	0.51	38	1.05	0.49		N/A	0.96	818
4	Lean CLAY (CL)	20.0	2813	1908	0.54	25	1.05	0.52		N/A	0.95	1039
5	clayey GRAVEL (GC	25.0	3438	2221	0.56	30	1.05	0.54		N/A	0.94	1255
6	clayey GRAVEL (GC	30.0	3875	2440	0.57	33	1.05	0.54		N/A	0.92	1393
7	clayey SAND (SC)	32.0	4250	2628	0.57	37	1.05	0.54		N/A	0.90	1487
8	clayey SAND (SC)	36.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	325	0.16	492	2.0E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	792	0.40	775	3.5E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Lean CLAY (CL)	1037	0.52	1066	3.8E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	Lean CLAY (CL)	1240	0.62	1015	5.1E-04	0	0.0E+00		1.0	0.0	0.000	0.00
5	clayey GRAVEL (GC	1443	0.72	1171	5.4E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	clayey GRAVEL (GC	1586	0.79	1265	5.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	clayey SAND (SC)	1708	0.85	1357	5.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-108

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	8.0	4.0	125.0	8	80	500	0	500	1.70	1.3	1.00	0.75	1.00	13	5.00	1.20	21	0.23	0.68	0.99	0.39	1.05	N/A
2	Lean CLAY (CL)	8.0	7.0	11.5	125.0	11	90	1438	218	1219	1.28	1.3	1.00	0.75	1.00	14	5.00	1.20	21	0.23	0.70	0.97	0.45	1.05	N/A
3	Lean CLAY (CL)	15.0	5.0	17.5	125.0	22	90	2188	593	1595	1.12	1.3	1.00	0.85	1.00	27	5.00	1.20	38	N/A	N/A	0.96	0.51	1.05	N/A
4	Lean CLAY (CL)	20.0	5.0	22.5	125.0	13	90	2813	905	1908	1.02	1.3	1.00	0.95	1.00	16	5.00	1.20	25	0.29	0.86	0.95	0.54	1.05	N/A
5	clayey GRAVEL (GC	25.0	5.0	27.5	125.0	18	44	3438	1217	2221	0.95	1.3	1.00	0.95	1.00	21	5.00	1.20	30	N/A	N/A	0.94	0.56	1.05	N/A
6	clayey GRAVEL (GC	30.0	2.0	31.0	125.0	21	44	3875	1435	2440	0.91	1.3	1.00	0.95	1.00	23	5.00	1.20	33	N/A	N/A	0.92	0.57	1.05	N/A
7	clayey SAND (SC)	32.0	4.0	34.0	125.0	27	20	4250	1622	2628	0.87	1.3	1.00	1.00	1.00	31	3.61	1.08	37	N/A	N/A	0.90	0.57	1.05	N/A
8	clayey SAND (SC)	36.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-108

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-109
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	7	80	22	N/A	0.0
2	Lean CLAY	5	125	9	90	20	N/A	0.0
3	Lean CLAY	8	125	14	90	20	N/A	0.0
4	sandy CLAY	12	125	25	66	11	N/A	0.0
5	clayey SAND	21	125	20	45	11	N/A	0.0
6	Lean CLAY	25	125	5	85	12	N/A	0.0
7	sand with clay	36	125	22	12	12	0.60	0.5
8	fat clay	40	125	5	58	30	N/A	0.0
9	clayey SAND	41	125	28	20	11	N/A	0.0
10	clayey SAND	45	125	40	20	11	N/A	0.0
11	clayey SAND	50	125				N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.5

TOTAL SETTLEMENT (inches)	0.5

Table 1: Liquefaction Analysis for B-109

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	313	313	0.39	19	1.05	0.37		N/A	0.99	121
2	Lean CLAY	5.0	813	813	0.38	22	1.05	0.37		N/A	0.98	312
3	Lean CLAY	8.0	1250	1125	0.42	27	1.05	0.40		N/A	0.98	476
4	sandy CLAY	12.0	2063	1532	0.50	43	1.05	0.48		N/A	0.96	773
5	clayey SAND	21.0	2875	1939	0.55	35	1.05	0.52		N/A	0.95	1061
6	Lean CLAY	25.0	3813	2409	0.57	12	1.05	0.54		N/A	0.93	1377
7	sand with clay	36.0	4750	2878	0.56	26	1.05	0.53	1.10	0.5	0.86	1602
8	fat clay	40.0	5063	3035	0.55	11	1.05	0.52		N/A	0.84	1667
9	clayey SAND	41.0	5375	3191	0.54	35	1.05	0.516		N/A	0.82	1727
10	clayey SAND	45.0	5938	3473	0.53	46	1.05	0.500		N/A	0.79	1823
11	clayey SAND	50.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	203	0.10	376	1.6E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	Lean CLAY	528	0.26	633	2.5E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Lean CLAY	731	0.37	801	3.0E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	sandy CLAY	996	0.50	1091	3.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	clayey SAND	1260	0.63	1149	4.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	Lean CLAY	1566	0.78	892	7.7E-04	below water	#VALUE!	0	1.0	0.0	0.000	0.00
7	sand with clay	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	fat clay	1972	0.99	990	8.4E-04	below water	#VALUE!	0	1.0	0.0	0.000	0.00
9	clayey SAND	2074	1.04	1468	5.9E-04	N/A	N/A		1.0	0.0	0.000	0.00
10	clayey SAND	2257	1.13	1683	5.4E-04	N/A	N/A		1.0	0.0	0.000	0.00
11	clayey SAND	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-109

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	5.0	2.5	125.0	7	80	313	0	313	1.70	1.3	1.00	0.75	1.00	12	5.00	1.20	19	0.20	0.61	0.99	0.39	1.05	N/A
2	Lean CLAY	5.0	3.0	6.5	125.0	9	90	813	0	813	1.57	1.3	1.00	0.75	1.00	14	5.00	1.20	22	0.24	0.71	0.98	0.38	1.05	N/A
3	Lean CLAY	8.0	4.0	10.0	125.0	14	90	1250	125	1125	1.33	1.3	1.00	0.75	1.00	18	5.00	1.20	27	0.33	1.00	0.98	0.42	1.05	N/A
4	sandy CLAY	12.0	9.0	16.5	125.0	25	66	2063	530	1532	1.14	1.3	1.00	0.85	1.00	32	5.00	1.20	43	N/A	N/A	0.96	0.50	1.05	N/A
5	clayey SAND	21.0	4.0	23.0	125.0	20	45	2875	936	1939	1.02	1.3	1.00	0.95	1.00	25	5.00	1.20	35	N/A	N/A	0.95	0.55	1.05	N/A
6	Lean CLAY	25.0	11.0	30.5	125.0	5	85	3813	1404	2409	0.91	1.3	1.00	0.95	1.00	6	5.00	1.20	12	0.13	0.13	0.93	0.57	1.05	N/A
7	sand with clay	36.0	4.0	38.0	125.0	22	12	4750	1872	2878	0.83	1.3	1.00	1.00	1.00	24	1.55	1.03	26	0.32	0.32	0.86	0.56	1.05	0.60
8	fat clay	40.0	1.0	40.5	125.0	5	58	5063	2028	3035	0.81	1.3	1.00	1.00	1.00	5	5.00	1.20	11	0.13	0.38	0.84	0.55	1.05	N/A
9	clayey SAND	41.0	4.0	43.0	125.0	28	20	5375	2184	3191	0.79	1.3	1.00	1.00	1.00	29	3.61	1.08	35	N/A	N/A	0.82	0.54	1.05	N/A
10	clayey SAND	45.0	5.0	47.5	125.0	40	20	5938	2465	3473	0.76	1.3	1.00	1.00	1.00	39	3.61	1.08	46	N/A	N/A	0.79	0.53	1.05	N/A
11	clayey SAND	50.0	N/A	N/A	125.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2
*8	$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$		
*9	If PI of Fines>15, Increase CRR by 300%		
*10	rd = 1.0 - 0.00765z	z ≤ 9.15m	
	1.174 - 0.0267z	z > 9.15m	
*11	Magnitude Scaling Factor (NCEER, 1997, mean values)		

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-109

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-110
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE

Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	7	80	22	N/A	0.0
2	Lean CLAY	5	125	7	90	20	N/A	0.0
3	Lean CLAY	8	125	30	90	20	N/A	0.0
4	gravelly CLAY	12	125	20	65	20	N/A	0.0
5	clayey SAND	18	125	28	20	10	N/A	0.0
6	clayey GRAVEL	22	125	31	20	10	N/A	0.0
7	clayey GRAVEL	25					N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-110

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	313	313	0.39	19	1.05	0.37		N/A	0.99	121
2	Lean CLAY	5.0	813	813	0.38	18	1.05	0.37		N/A	0.98	312
3	Lean CLAY	8.0	1250	1125	0.42	52	1.05	0.40		N/A	0.98	476
4	gravelly CLAY	12.0	1875	1438	0.49	36	1.05	0.47		N/A	0.97	706
5	clayey SAND	18.0	2500	1751	0.53	44	1.05	0.51		N/A	0.95	930
6	clayey GRAVEL	22.0	2938	1970	0.55	45	1.05	0.52		N/A	0.95	1083
7	clayey GRAVEL	25.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
8	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	203	0.10	376	1.6E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	Lean CLAY	528	0.26	595	2.6E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Lean CLAY	731	0.37	995	2.4E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	gravelly CLAY	935	0.47	1000	3.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	clayey SAND	1138	0.57	1172	4.0E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	clayey GRAVEL	1281	0.64	1259	4.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	clayey GRAVEL	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-110

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	5.0	2.5	125.0	7	80	313	0	313	1.70	1.3	1.00	0.75	1.00	12	5.00	1.20	19	0.20	0.61	0.99	0.39	1.05	N/A
2	Lean CLAY	5.0	3.0	6.5	125.0	7	90	813	0	813	1.57	1.3	1.00	0.75	1.00	11	5.00	1.20	18	0.19	0.57	0.98	0.38	1.05	N/A
3	Lean CLAY	8.0	4.0	10.0	125.0	30	90	1250	125	1125	1.33	1.3	1.00	0.75	1.00	39	5.00	1.20	52	N/A	N/A	0.98	0.42	1.05	N/A
4	gravelly CLAY	12.0	6.0	15.0	125.0	20	65	1875	437	1438	1.18	1.3	1.00	0.85	1.00	26	5.00	1.20	36	N/A	N/A	0.97	0.49	1.05	N/A
5	clayey SAND	18.0	4.0	20.0	125.0	28	20	2500	749	1751	1.07	1.3	1.00	0.95	1.00	37	3.61	1.08	44	N/A	N/A	0.95	0.53	1.05	N/A
6	clayey GRAVEL	22.0	3.0	23.5	125.0	31	20	2938	967	1970	1.01	1.3	1.00	0.95	1.00	39	3.61	1.08	45	N/A	N/A	0.95	0.55	1.05	N/A
7	clayey GRAVEL	25.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
8	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8 $$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-110

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-111
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE

Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	11	80	22	N/A	0.0
2	Lean CLAY (CL)	4	125	13	90	20	N/A	0.0
3	Lean CLAY (CL)	8	125	16	90	20	N/A	0.0
4	gravelly CLAY (CL)	12	125	25	75	10	N/A	0.0
5	Lean CLAY (CL)	20	125	7	90	10	N/A	0.0
6	clayey SAND (SC)	24	125	36	50	10	N/A	0.0
7	clayey SAND (SC)	30					N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-111

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	250	250	0.39	27	1.05	0.37		N/A	1.00	97
2	Lean CLAY (CL)	4.0	750	750	0.38	30	1.05	0.37		N/A	0.99	288
3	Lean CLAY (CL)	8.0	1250	1125	0.42	30	1.05	0.40		N/A	0.98	476
4	gravelly CLAY (CL)	12.0	2000	1501	0.50	43	1.05	0.48		N/A	0.96	751
5	Lean CLAY (CL)	20.0	2750	1876	0.54	16	1.05	0.52		N/A	0.95	1017
6	clayey SAND (SC)	24.0	3375	2189	0.56	56	1.05	0.54		N/A	0.94	1233
7	clayey SAND (SC)	30.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
8	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	163	0.08	378	1.3E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	488	0.24	677	2.1E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Lean CLAY (CL)	731	0.37	830	2.9E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	gravelly CLAY (CL)	976	0.49	1083	3.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	Lean CLAY (CL)	1220	0.61	867	5.9E-04	below water	#VALUE!	0.00E+00	1.0	0.0	0.000	0.00
6	clayey SAND (SC)	1423	0.71	1424	4.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
7	clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-111

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	4.0	2.0	125.0	11	80	250	0	250	1.70	1.3	1.00	0.75	1.00	18	5.00	1.20	27	0.34	1.01	1.00	0.39	1.05	N/A
2	Lean CLAY (CL)	4.0	4.0	6.0	125.0	13	90	750	0	750	1.63	1.3	1.00	0.75	1.00	21	5.00	1.20	30	0.46	1.37	0.99	0.38	1.05	N/A
3	Lean CLAY (CL)	8.0	4.0	10.0	125.0	16	90	1250	125	1125	1.33	1.3	1.00	0.75	1.00	21	5.00	1.20	30	0.46	1.39	0.98	0.42	1.05	N/A
4	gravelly CLAY (CL)	12.0	8.0	16.0	125.0	25	75	2000	499	1501	1.15	1.3	1.00	0.85	1.00	32	5.00	1.20	43	N/A	N/A	0.96	0.50	1.05	N/A
5	Lean CLAY (CL)	20.0	4.0	22.0	125.0	7	90	2750	874	1876	1.03	1.3	1.00	0.95	1.00	9	5.00	1.20	16	0.17	0.17	0.95	0.54	1.05	N/A
6	clayey SAND (SC)	24.0	6.0	27.0	125.0	36	50	3375	1186	2189	0.96	1.3	1.00	0.95	1.00	42	5.00	1.20	56	N/A	N/A	0.94	0.56	1.05	N/A
7	clayey SAND (SC)	30.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
8	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8 $$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-111

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-112
Date:	3/18/2008
Calculated By:	MP

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.4
Design PGAmax (g)	0.6
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY	0	125	12	80	22	N/A	0.0
2	Lean CLAY (CL)	5	125	9	90	20	N/A	0.0
3	Lean CLAY (CL)	8	125	14	90	20	N/A	0.0
4	clayey SAND (SC)	14	125	18	14		0.68	0.4
5	clayey SAND (SC)	17	125	17	15		0.53	0.7
6	sandy CLAY (CL)	22	125	13	66		N/A	0.0
7	silty SAND (SM)	28	125	36	15		N/A	0.0
8	SAND with Silt (SP-SM)	33	125	32	10		N/A	0.0
9	SAND with Silt (SP-SM)	40					N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	1.1

TOTAL SETTLEMENT (inches)	1.1

Table 1: Liquefaction Analysis for B-112

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY	0.0	313	313	0.39	29	1.05	0.37		N/A	0.99	121
2	Lean CLAY (CL)	5.0	813	813	0.38	22	1.05	0.37		N/A	0.98	312
3	Lean CLAY (CL)	8.0	1375	1188	0.44	26	1.05	0.42		N/A	0.97	522
4	clayey SAND (SC)	14.0	1938	1470	0.50	26	1.05	0.47	1.10	0.4	0.96	728
5	clayey SAND (SC)	17.0	2438	1720	0.53	24	1.05	0.50	1.20	0.7	0.95	907
6	sandy CLAY (CL)	22.0	3125	2064	0.56	24	1.05	0.53		N/A	0.94	1148
7	silty SAND (SM)	28.0	3813	2409	0.57	45	1.05	0.54		N/A	0.93	1377
8	ND with Silt (SP-S	33.0	4563	2784	0.56	37	1.05	0.53		N/A	0.88	1560
9	ND with Silt (SP-S	40.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.05	N/A		N/A	N/A	N/A

Design Earthquake M = 7.4

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY	203	0.10	432	1.4E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	528	0.26	633	2.5E-04	0	0.0E+00		1.0	0.0	0.000	0.00
3	Lean CLAY (CL)	772	0.39	817	3.2E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
5	clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
6	sandy CLAY (CL)	1342	0.67	1045	5.5E-04	below water	#VALUE!	0	1.0	0.0	0.000	0.00
7	silty SAND (SM)	1566	0.78	1389	5.0E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	ND with Silt (SP-S	1810	0.90	1399	5.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
9	ND with Silt (SP-S	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-112

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY	0.0	5.0	2.5	125.0	12	80	313	0	313	1.70	1.3	1.00	0.75	1.00	20	5.00	1.20	29	0.40	1.21	0.99	0.39	1.05	N/A
2	Lean CLAY (CL)	5.0	3.0	6.5	125.0	9	90	813	0	813	1.57	1.3	1.00	0.75	1.00	14	5.00	1.20	22	0.24	0.71	0.98	0.38	1.05	N/A
3	Lean CLAY (CL)	8.0	6.0	11.0	125.0	14	90	1375	187	1188	1.30	1.3	1.00	0.75	1.00	18	5.00	1.20	26	0.32	0.96	0.97	0.44	1.05	N/A
4	clayey SAND (SC)	14.0	3.0	15.5	125.0	18	14	1938	468	1470	1.17	1.3	1.00	0.85	1.00	23	2.20	1.04	26	0.32	0.32	0.96	0.50	1.05	0.68
5	clayey SAND (SC)	17.0	5.0	19.5	125.0	17	15	2438	718	1720	1.08	1.3	1.00	0.85	1.00	20	2.50	1.05	24	0.27	0.27	0.95	0.53	1.05	0.53
6	sandy CLAY (CL)	22.0	6.0	25.0	125.0	13	66	3125	1061	2064	0.98	1.3	1.00	0.95	1.00	16	5.00	1.20	24	0.27	0.27	0.94	0.56	1.05	N/A
7	silty SAND (SM)	28.0	5.0	30.5	125.0	36	15	3813	1404	2409	0.91	1.3	1.00	0.95	1.00	41	2.50	1.05	45	N/A	N/A	0.93	0.57	1.05	N/A
8	ND with Silt □(SP-S	33.0	7.0	36.5	125.0	32	10	4563	1778	2784	0.85	1.3	1.00	1.00	1.00	35	0.87	1.02	37	N/A	N/A	0.88	0.56	1.05	N/A
9	ND with Silt □(SP-S	40.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.05	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8 $$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table 3: Liquefaction Analysis for B-112

APPENDIX F2

Geotechnical and Geohazard Evaluation,
Outpatient Center No. 2

FUGRO CONSULTANTS, INC.

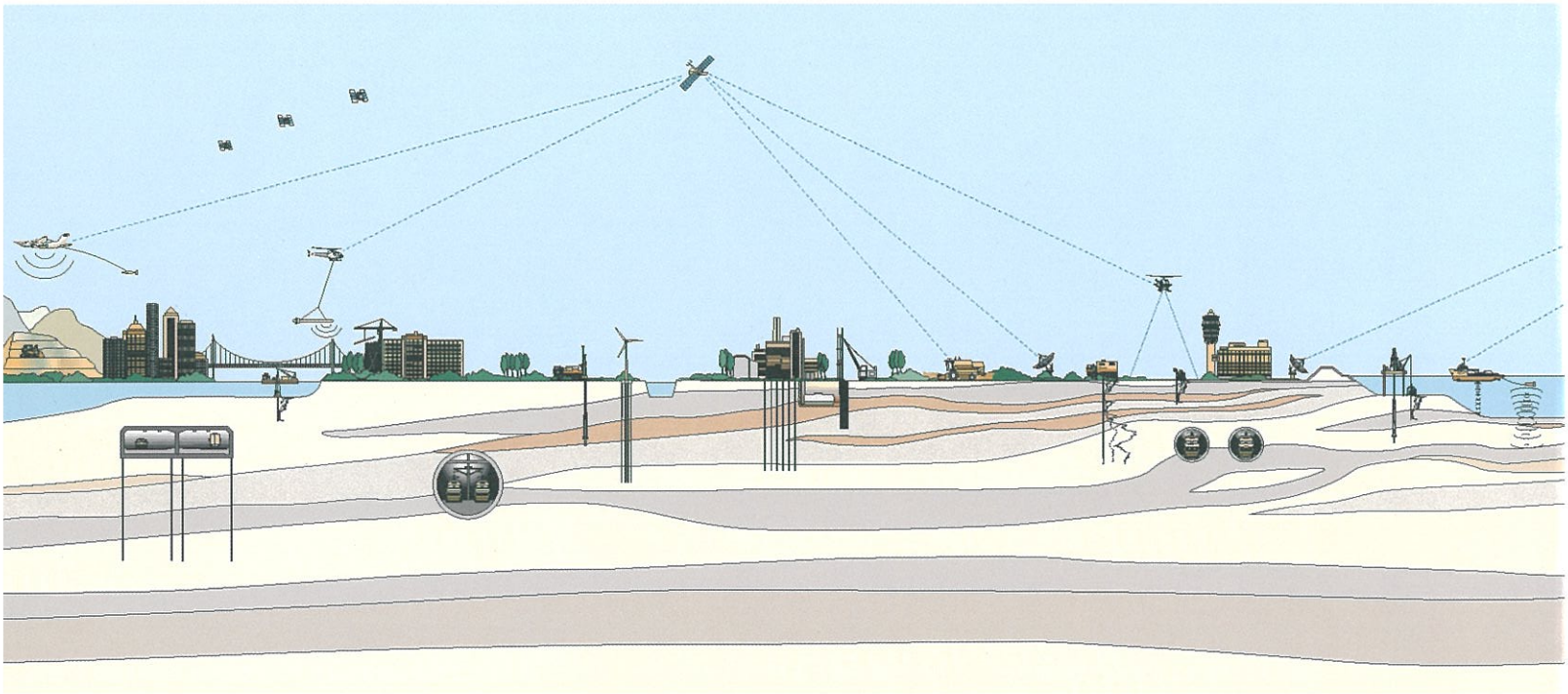
GEOTECHNICAL AND GEOHAZARD EVALUATION
OUTPATIENT CENTER No. 2
CHILDREN'S HOSPITAL & RESEARCH CENTER
OAKLAND, CALIFORNIA

Prepared for:

CHILDREN'S HOSPITAL & RESEARCH CENTER AT OAKLAND
C/O HARRIS & ASSOCIATES

April 2014

Fugro Project No. 04.72120009



FUGRO CONSULTANTS, INC.

1000 Broadway, Suite 440
Oakland, California 94607
Tel: (510) 268-0461
Fax: (510) 268-0137

April 4, 2014
Project No. 04.72120009

Children's Hospital & Research Center at Oakland
747 52nd Street
Oakland, California 94609

Attention: Mr. Doug Nelson

Subject: Geotechnical and Geologic Hazards Evaluation
Outpatient Center No. 2, Children's Hospital & Research Center,
Oakland, California

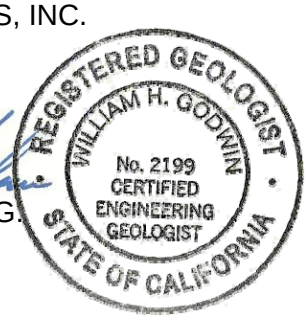
Dear Mr. Nelson:

Fugro Consultants, Inc. is pleased to submit this geotechnical and geologic hazards evaluation report presenting the results of our field explorations, laboratory-testing program, and engineering analyses for the Outpatient Center No. 2 at the Children's Hospital and Research Center in Oakland, California.

We appreciate this opportunity to be of service to you. Please contact us if you have any questions regarding the information presented in this report.

Sincerely,

FUGRO CONSULTANTS, INC.



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(PDF) Frank Penrose, HDR, Inc.

CONTENTS

Page

1.0 INTRODUCTION....1
1.1 Project Description....1
1.2 Scope of Services....2
2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING....2
2.1 Review of Existing Data....2
2.2 Aerial Photograph Review....4
2.3 Field Exploration....6
2.4 Laboratory Testing....6
3.0 GEOLOGIC SETTING....7
3.1 Regional Geology....7
3.2 Regional Seismicity....8
3.2.1 Historic Seismicity....10
4.0 SITE CONDITIONS....11
4.1 Surface Conditions....11
4.2 Site Geology....12
4.3 Subsurface Conditions....12
4.3.1 Outpatient Center - 2....13
4.4 Groundwater....14
5.0 GEOLOGIC HAZARDS EVALUATION....15
5.1 Surface Fault Offset Hazard....15
5.2 Shaking Hazards....16
5.2.1 Liquefaction....16
5.2.2 Dynamic Densification....18
5.2.3 Lateral Spreading....18
5.2.4 Tsunami and Seiche....18
5.2.5 Landsliding....19
5.2.6 Other Shaking Hazards....19
5.3 Other Geologic Hazards and Complicating Site Conditions....19
5.3.1 Flooding and Inundation Due to Dam or Embankment Failure....19
5.3.2 Debris Flow....20
5.3.3 Land Subsidence....20
5.3.4 Soil Expansion....20
5.3.5 Corrosion Potential....21
5.3.6 Naturally Occurring Asbestos (NOA)....21
5.3.7 Radon-222....21
5.4 Conclusions - Geologic Hazards....22
5.5 Other Hazards and Complicated Site Conditions Not Addressed In This Report....22
6.0 DISCUSSIONS AND CONCLUSIONS....22

CONTENTS – CONTINUED

6.1 Expansive Soils 23
6.2 Undocumented Fill 23
6.3 Foundation Support 23
6.4 Construction Considerations 24
6.4.1 Existing Structures and Foundations 24
6.4.2 Earthwork 24
6.4.3 Height Restrictions During Construction 24
6.4.4 Construction Quality Control 24

7.0 RECOMMENDATIONS 24
7.1 Seismic Design Criteria 25
7.1 Site Preparation and Grading 25
7.1.1 Site Preparation 25
7.1.2 Subgrade Preparation 26
7.1.3 Reworking Undocumented Fill 27
7.1.4 Engineered Fill Materials 27
7.1.5 Fill Placement and Compaction 27
7.1.6 Trench Backfill 27
7.1.7 Surface Drainage 28
7.2 Temporary Shoring and Construction Slopes 28
7.2.1 General 28
7.2.2 Temporary Construction Slopes 28
7.2.3 Temporary Shoring 29
7.3 Foundations 29
7.3.1 Drilled Pier Foundations 29
7.3.2 Drilled Pier Construction 30
7.3.3 Pier Caps and Grade Beams 30
7.4 Lateral Loads 31
7.4.1 Long-Term/Static Lateral Loads 31
7.5 Concrete Slabs 33
7.5.1 Interior Slab 33
7.5.2 Exterior Slabs-on-Grade 34
7.6 Retaining / Below Grade Walls 34
7.6.1 Unrestrained Walls 35
7.7 Buried Diesel Fuel Tank 36
7.8 Support of Minor Structural Elements 36
7.9 Additional Geotechnical Services 37

8.0 LIMITATIONS 38

9.0 REFERENCES 39

CONTENTS – CONTINUED

TABLES

Page

Table 1. Aerial Photographs Examined 5
Table 2. Summary of Significant Seismic Sources within 25 km of the Site 10
Table 3. Large Magnitude (M≥6.0) Regional Earthquakes Within 100 Miles of the Site 11
Table 4. 2013 CBC Seismic Design Parameters 25
Table 5. Drilled Pier Design Skin Friction 30
Table 6. Passive Soil Resistance (P) versus Displacement (Δ) Relationship 31
Table 7. Reduction Factors for Drilled Pier Lateral Load Capacity 32
Table 8. Average P-Multipliers for Various Pier Groups 33
Table 9. Lateral Capacity of Conventional Drilled Piers 33
Table 10. Allowable Bearing Pressures for Spread Footings 37

PLATES

Plate

Vicinity Map 1
Topographic Site Map 2
Site Plan 3
Creek Location Map 4
Geologic Map 5
Regional Fault Map 6a
Vicinity Fault Map 6b
Faulting and Historical Seismicity 7
Geologic Cross-Section A-A' 8a
Geologic Cross-Section B-B' 8b
Cross-Section C-C' 8c
CGS Seismic Hazard Zone Map with Alquist-Priolo Earthquake Zones 9
24" Diameter Drilled Piers (Free Head) 10a-10c
24" Diameter Drilled Piers (Fixed Head) 11a-11c
30" Diameter Drilled Piers (Free Head) 12a-12c
30" Diameter Drilled Piers (Fixed Head) 13a-13c
Static Lateral Pressures on Unrestrained Walls 14a
Seismic Lateral Pressures on Unrestrained Walls 14b

CONTENTS – CONTINUED

APPENDICES

APPENDIX A FIELD EXPLORATIONS

Key to Soil Classification and Test Data ..... Plate A-1
Boring Logs ..... B-113 through B-118
Historical Boring Logs

APPENDIX B LABORATORY TESTING PROGRAM

Summary of Laboratory Test Results ..... Plate B-1
Plasticity Chart and Data ..... Plate B-2a
Plasticity Chart and Historic Data ..... Plate B-2b
Gradation Test Data ..... Plate B-3
Cerco Corrosion test results and Evaluation

APPENDIX C SEISMIC HAZARD ANALYSIS

APPENDIX D LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

1.0 INTRODUCTION

This report presents the results of the geotechnical study, and geohazard evaluation conducted by Fugro Consultants, Inc. (Fugro) for the Outpatient Center No. 2 (OPC-2) project for Children's Hospital & Research Center at Oakland (CHRCO). The OPC-2 will be located within the hospital campus roughly bounded by Martin Luther King Jr. Way to the west, Highway 24 to the east and 53nd Street to the north in Oakland, Alameda County, California as shown on Plate 1 – Vicinity Map. Specifically, the OPC-2 will be located northeast of the intersection os Martin Luther King Jr. Way and 52nd Street. The site coordinates are:

Latitude: 37.8375 º
Longitude: -122.2682 º

The project site is plotted on a 7 ½-minute United States Geological Survey (USGS) quadrangle topographic map on Plate 2 – Topographic Site Map.

1.1 PROJECT DESCRIPTION

Based on our conversations with the project design team and a conceptual site plan by HDR, the project architect, the OPC-2 will be constructed immediately adjacent to the existing Outpatient Center No. 1 (OPC-1) and existing hospital Parking Structure, allowing space for a seismic separation. The proposed building will consist of a 6-story structure with an approximate footprint area of 17,000 square feet. The 1st floor (ground floor) of the building will serve as a parking area and will also house utility service rooms.

The 1st floor garage grades will conform closely with existing grades of about Elevation +91 to 93 feet[1]. Additionally, grades will conform to the OPC-1 finished floor Elevation +93.22 feet. The adjacent OPC-1 building and Parking Structure both have basement floors bearing at approximate Elevation +77.2 feet (about 13 to 15 feet below existing site grades). The grade separation between the adjacent basement levels and the lower level of the OPC-2 poses design implications that are discussed in appropriate sections.

Rutherford + Chekene, Inc. (R+C), the project structural engineer, provided a conceptual plan showing axial column dead plus live loads will be on the order of 50 to 1000 kips. The highest vertical loads occur at interior columns.

A bio-swale will be constructed at the Southwest corner of the proposed OCP-2.

It is our understanding the proposed OPC-2 will not be considered as a critical structure under the California Building Code and will not be subject to review by the California Geologic Survey. The project will be subject to OSHPD3 review.

[1] Unless otherwise noted, all elevations in this report are in reference to the City of Oakland Datum

1.2 SCOPE OF SERVICES

The purpose of our geotechnical field exploration and laboratory-testing program was to obtain information on subsurface conditions to evaluate the geotechnical aspects of the portion of the site where OPC-2 will be located. The scope of our services included:

- Compiling and reviewing available geotechnical and geologic data that is contained in our files and is pertinent to the project vicinity. This includes a series of previous reports prepared for various developments at the site;
- Conducting a field exploration program to supplement the available information on subsurface conditions at the proposed development location;
- Evaluate the corrosion potential of near-surface soils;
- Evaluating the geologic hazards at the site;
- Performing engineering analyses of the collected data and developing geotechnical recommendations and design criteria for the proposed OC-2; and
- Preparing this geotechnical report presenting the results of our geotechnical field exploration, laboratory testing program, discussion of geotechnical issues and geologic hazards and our geotechnical recommendations.

2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING

The data review and field exploration program described herein was developed to provide general characterization of the subsurface materials.

2.1 REVIEW OF EXISTING DATA

Prior to conducting our field exploration and laboratory testing, Fugro reviewed relevant information relating to geotechnical, geologic, and seismic issues at the site. Fugro's local predecessor companies completed several geotechnical investigations at the Children's Hospital site since 1974. Historic boring locations from our previous work at the site are shown on Plate 3 – Site Plan. Data from these borings was used in preparing the cross sections presented in this report, and for our engineering analyses. We reviewed the results of these previous studies performed at the site, including the following reports that contain information directly related to the current project:

- Fugro (March 2009) "Phase 1 New Hospital Replacement Project New Patient Tower Phase I": Fugro performed geotechnical and geohazard evaluation for a new patient tower as a part of the new hospital replacement project. The project includes constructing a 5 to 6-story structure at the location of the existing B/C Wing structure on the south side of the main hospital and a retrofit of an existing helipad structure to be a new Services Building. Six borings were advanced to depths of 25 to 75 feet. Fugro later used this report as a basis to issue supplemental geotechnical

recommendations for a Bulk Liquid Oxygen Tank that is under construction at this time.

- Fugro (May 2008) "New Hospital Replacement Project": Fugro performed geotechnical, geological and seismic hazard evaluation for the new hospital replacement project including a 12-story hospital building within a neighboring residential area. Various analyses had been performed as part of the scope satisfying the CGS requirements. However, due to various reasons, the location of the new structure was moved within the campus by proposing the demolition of the B/C wing. This report was used to supplement our current evaluation.
- Fugro (2002) "Western Expansion": Fugro performed a geotechnical study for the western expansion of the hospital. Fugro also completed a supplemental field exploration program including four soil borings and extensive lab testing. The results of this investigation were used to evaluate the liquefaction potential at the site as part of OSHPD Permit Number HS-002307-01. This report was used to supplement our current liquefaction evaluation.
- Harza (1997) "Children's Hospital Helistop": Harza Consulting Engineers (Harza) (local predecessor company acquired by Fugro) completed two soil borings for the hospital helistop. The helistop is located southerly of the area for the Patient Tower project. The information in this report was used as a primary source of subsurface information for the SB retrofit.
- Harza (1994) "Proposed Patient Services Pavilion, Phase I": Harza completed six borings as part of a geotechnical and seismology study for the proposed patient services pavilion. These borings extended to depths up to 51 feet below grade. Laboratory testing was completed on samples collected from these borings, including index and strength tests. Two borings were converted to monitoring wells to observe groundwater levels. This report was used to supplement our current study.
- Harza Kaldveer (1992) "Geotechnical Investigation, Proposed Cath Laboratory": Harza Kaldveer completed two soil borings as part of a geotechnical investigation for a proposed Cardiac .Catheterization Laboratory located east of the Main Hospital Building and west of Dover Street. These borings extended to a maximum depth of 20-1/2 feet below grade.
- Kaldveer (1992) "Geotechnical Investigation, Ambulatory Services Center": Kaldveer Associates, Inc. (Kaldveer) (predecessor company acquired by Harza) completed two soil borings for the proposed Ambulatory Services Center located on the north side of 52nd Street at the Children's Hospital complex. These borings extended to a maximum depth of 59-1/2 feet below grade. Boring PB92-1 and PB-92-2 were located near the east end of the proposed OPC-2 building and are relevant to this study.
- Kaldveer (1992) "Geotechnical Investigation, CHORI Laboratory Addition": Kaldveer completed four soil borings for the proposed Children's Hospital Oakland Research Institute (CHORI) Laboratory Addition building at southern end of the Children's Hospital complex. The borings extended to a maximum depth of 50-1/2 feet below

grade. This investigation was performed concurrently with the proposed Ambulatory Services Center.

- Kaldveer (1990) "Proposed Medical Office Building and Parking Structure": Kaldveer Associates, Inc. (Kaldveer) (predecessor company acquired by Harza) completed four borings as part of a geotechnical study for the parking structure and office building on 52nd Street. One of the borings (PB90-3) was located at the southwest corner of the existing Parking Structure and is relevant to the OPC-2 building. This report was used as a primary source of data in our current investigation.
- Kaldveer (1987) "Foundation Investigation for Nursing Tower": Kaldveer completed two borings and related laboratory testing for the nursing tower on the south side of the Children's Hospital campus.
- Kaldveer (1986) "Foundation Investigation, Proposed Medical Office Building and Parking Structure": Kaldveer completed eight soil borings to a maximum depth of 60-1/2 feet below grade for a proposed medical office building and parking structure. Boring PB86-1 and PB-86-3 were located within the footprint of the existing parking structure and are relevant to this study.
- Kaldveer (1978) "Geotechnical Investigation for Addition to Children's Hospital": Kaldveer completed seven borings to a maximum depth of 60 feet in the location of the addition located adjacent to the existing hospital.

A complete list of references used in this study is included in Section 9.0 – References. Pertinent previous boring logs used to interpret subsurface conditions at the proposed structure are included in Appendix A.

2.2 AERIAL PHOTOGRAPH REVIEW

Six sets of vertical angle, black and white aerial photographs taken between 1930 and 1990 were examined both in stereo pairs and as single photos. Stereo views are used to determine relief and shape of landforms, which are commonly controlled by bedrock type and tectonic movement. Single photographs are reviewed for tonal variations and patterns that often go unnoticed in stereo because of the high relief. The historic photos were obtained from the Pacific Aerial Surveys library. A list of the photographs reviewed is provided in Table 1 below.

Table 1. Aerial Photographs Examined

Flight Date	Approximate Scale	Serial Number
6/12/90	1:12,000	AV3845-7-21/22
5/2/69	1:12,000	AV 902-07-17/18
7/25/63	1:36,000	AV 550-08-19/20
5/3/57	1:12,000	AV 253-08-21/22
3/24/47	1:20,000	AV 11-04-11/12
1930	1:9,500	GY-30-38/39/69/70

Each photoset was examined for changes in vegetation and geologic conditions related to soil movements and bedrock conditions. Interpretations are generally based on three factors: tone, texture and trend. Tonal and textural variations, if present, may be directly related to 1) the rocks or soils underlying the site, 2) water content of the underlying material, 3) natural vegetation (commonly controlled by local geology and water content), or 4) human activities (surface modification) including agriculture. Directional trends are observed as linear features.

The examination of aerial photographs for this investigation included the identification of geologic hazards. Landslides are identifiable through such features as headscarps, concave slopes, lateral drainages, hummocky terrain and lobate toes. Faults are expressed by offset tonal or textural bands, tonal lineaments, isolated lines of springs or trees, abrupt changes in slope or drainage direction, and by streams with anomalously straight or curved courses.

Our photograph review indicates no presence of historic landslides and no features indicative of active faulting at the site. Linear features associated with the Hayward fault are visible (1963 photo) about 2 miles east of the site. No other features relating to active faulting were observed in the vicinity of the project site.

The earliest aerial photographs reviewed for the site were from 1930. At that time, the area surrounding the hospital was developed as a predominately residential neighborhood. The site appears to be developed with an early hospital tower. However, the tower in the photo is not in the configuration of today. To the south of the site, in the vicinity of the intersection of Martin Luther King Way and 47th Street, Temescal Creek is visible. The location of Temescal Creek is shown on Plate 4 – Creek Location Map, and discussed in Section 3.1 below. The close proximity of this creek implies the presence of paleochannels beneath the site, created from a meandering path. No additional features indicative of geologic hazard are visible within or trending towards the subject property. No significant changes were made before the 1947 photos.

The 1957 aerial photograph shows the existing Children's Hospital tower building as the structure appears today. The hospital is surrounded on all sides by residential property. Temescal Creek is visible on the same path as earlier photos. No features indicative of any geologic hazard are visible within or trending towards the subject property.

By 1963 the main u-shaped hospital tower has been constructed. This structure is in the same configuration today, and residential neighborhoods continue to surround the site. Temescal Creek is visible to the south; however, more of the creek path has been diverted into underground culverts.

By 1969 additional hospital towers have been constructed directly northwest of the main tower. Caltrans has constructed Highway 24 in the right-of-way directly east of the hospital. The elevated roadway includes retaining wall and embankment slope with an approximate inclination of 2:1 (horizontal to vertical).

The 1990 photograph was the earliest photograph reviewed after the 1989 Loma Prieta Earthquake. No features were readily observed as being related to earthquake-induced damage, such as faulting, liquefaction, or lateral spreading.

2.3 FIELD EXPLORATION

Fugro's field investigation for the project consisted of a site reconnaissance, utility locating, and advancing two CPTs. The CPT were located in the field by our engineer based on existing site features and, in combination with historical borings, to provide adequate coverage for the proposed Outpatient Center.

Fugro Consultants advanced the CPT soundings on February 16, 2012, and are numbered CPT-1 and CPT-2. Both CPTs were advanced to depths of about 60 feet below ground surface (bgs). A pore pressure dissipation test was performed in CPT-2 to provide and estimate of the groundwater table elevation. The test was performed at a depth of about 39.4 feet below ground surface (bgs). The CPTs were backfilled with cement grout in accordance with Alameda County drilling permit requirements.

In addition to the CPTs, a Fugro engineer sampled near surface soil to a depth of approximately 2½ feet bgs using a hand auger, designated as location HA-1 and HA-2. Near surface samples were transported to appropriate laboratories for classification and corrosion testing.

Logs of the CPTs are included in Appendix A. The CPT logs depict location specific subsurface conditions. The approximate locations of the CPTs were estimated by taping from existing landmarks and the locations should be considered accurate only to the degree implied by the method used. Elevations shown on the boring logs are in reference to the City of Oakland Datum, and were estimated from a topographical survey of the CHRCO campus prepared by SANDIS, Inc. The subsurface and ground water conditions encountered during our field investigation are summarized in Sections 4.3 and 4.4, respectively.

2.4 LABORATORY TESTING

The near-surface soil samples collected during our field investigation were transported to Fugro's soil mechanics laboratory in Oakland, California for classification testing. We performed

one Atterberg limits test on a sample collected from the hand auger location B-2. The Atterberg limits test result is presented in Appendix B

Corrosion testing was conducted on three soil samples at CERCO Analytical, Inc., in Pleasanton, California. Corrosion potential is discussed further in Section 5.3.5 – Corrosion Potential. Results of the corrosion testing are provided in Appendix C.

3.0 GEOLOGIC SETTING

3.1 REGIONAL GEOLOGY

The site is located within the San Francisco Bay alluvial plain. This area, along with the Diablo Mountain Range, and the Santa Cruz Mountains are within the Coast Range Geologic Province, a belt of sedimentary, volcanic, and metamorphic rocks that extend from southern California to Oregon. The structural geology of the Coast Range is complex and dominated by transpressive stress caused by the interaction between the Pacific and North American plates. This stress, concentrated along faults within the San Andreas fault system, results in the series of northwest-trending ridges and valleys that characterize the area. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates, subduction of the Pacific Plate beneath the North American Plate, and subsequent strike-slip faulting along the Proto-San Andreas, and San Andreas fault system. Strike-slip motion along the plate boundaries replaced subduction several million years ago (Atwater, 1970).

Geologic formations in the San Francisco Bay region range in age from Jurassic (190 to 135 million years ago) to recent Holocene (less than 11 thousand years ago). The oldest rocks are deformed Mesozoic sedimentary and volcanic rocks of the tectonically accreted Mesozoic Franciscan Complex and the contemporaneous Great Valley Sequence. During the last glacial maximum, significant relief of the bedrock was formed as a result of incision of creeks and streams to reach the elevation of the global sea level; approximately 140 feet lower than sea level today. As sea level rose with the onset of the current interglacial, lowland areas in the San Francisco Bay Area were subject to the deposition of a transgressive sequence of alluvial sediments, ranging in age from Pleistocene to the present. Younger alluvial, marine and marsh deposits have accumulated in the valleys in the region as a result of weathering in the surrounding mountains and sea level rise.

The regional geology, as mapped by Graymer (2000) is presented on Plate 5 – Geologic Map. Geologic units that are exposed within a mile of the site include the sandstone and quartz diorite members of the Novato Quarry terrane of the Franciscan Complex (Kfn, Kfgm), and surficial alluvial sediments (Qhaf, Qhl, Qpaf). Holocene aged alluvial fan deposits (Qhaf) underlie the site. Graymer (2000) described this formation as "alluvial fan and fluvial deposits, brown or tan, medium dense to dense, gravelly sand or sandy gravel that generally grades upward to sandy or silty clay." An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site as shown on Plate 5 – Geologic Map. The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. The fault does not appear to impact the Pleistocene aged alluvial

deposits mapped in the vicinity. The fault location is poorly constrained, and is not considered active, nor has the fault been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

Radbruch and Case (1967) map the site as being underlain by the Temescal Formation (Qtc). The Temescal Formation fills old channel meanders (of uncertain location) that were cut into the underlying San Antonio Formation (Qsu), and as such do not uniformly underlie the area. The thickness of the Temescal Formation varies from 3 to 18 feet. The nomenclature "Temescal Formation" was first used by Lawson (1914) to comprise several presumably contemporaneous alluvial units of different origin, lithology, and physical properties. The source material in the vicinity of the project location is assumed to be the San Antonio Formation, which consists of "clay, silt, sand, and gravel, some pebbles". Most beds contain flakes or pebbles of white Claremont chert. Clays of this formation are often montmorillonite, and highly plastic. The maximum thickness of the San Antonio Formation is unknown.

Sowers (1993, rev. 2000) mapped modern day hydrology of Oakland and its vicinity, and where available, historic creek paths were included. Temescal Creek was mapped as being contained in a culvert, passing directly south of the project location (Plate 4 – Creek Location Map). As described in Section 4.3, Subsurface Conditions, paleochannels resulting from the meandering path of the creek underlie portions of the project site.

The regional geomorphology is generally controlled by the active Hayward fault, which is located approximately 2 miles east–northeast of the project. Local geomorphology is controlled by fluvial deposition associated with Temescal Creek.

3.2 REGIONAL SEISMICITY

Geologists and seismologists recognize the San Francisco Bay Area as one of the most active seismic regions in the United States. Dominated by the San Andreas fault system, the Bay Area is comprised of mostly northwest-trending strike-slip faults driven by the interaction of the Pacific and North American Plates. Movement between these two plates is predominantly accommodated on the San Andreas, Hayward-Rogers Creek, Calaveras, San Gregorio, and Concord-Green Valley faults. The major fault in the system is the San Andreas fault, a major rift in the earth's crust that extends for at least 750 miles.

In 2003, the Working Group on California Earthquake Probabilities (WGCEP), in conjunction with the United States Geological Survey (USGS), published an updated report evaluating the probabilities of significant earthquakes occurring in the Bay Area over the next three decades (2002-2031). WGCEP found a 62 percent probability that at least one magnitude 6.7 or greater earthquake will occur in the San Francisco Bay region before 2031. This probability is an aggregate value that considers seven principal Bay Area fault systems and unknown faults (background values). The San Francisco Bay region continues to be seismically active. The principal active faults in the Bay Area include the San Andreas, Hayward, Calaveras, and the San Gregorio faults. Earthquakes occurring along these faults are capable

of generating strong ground shaking at the project site. The locations of significant Bay Area faults are shown on Plate 6a – Regional Fault Map, and Plate 6b – Vicinity Fault Map.

The closest active fault to the site is the Hayward fault. The Hayward fault was originally mapped by Lawson (1908), as part of his report to the State Earthquake Investigation Commission regarding the April 18, 1906 San Francisco earthquake. This report included a "comparison with other severe earthquakes in the same region", where he compiles data obtained from newspapers and eyewitness accounts. The fault was named after the city of Haywards (now Hayward), the site of the greatest damage.

The Hayward fault is the southern part of an extensive fault system that includes the Hayward, Rodgers Creek, Healdsburg, and Maacama faults. This system of faults extends over 280 kilometers (km) from San Jose to Mendocino County (Steinbrugge and others, 1987). The Hayward fault extends from the southwestern margin of the East Bay Hills for a distance of 105-km from San Pablo Bay in the north to an area near Mount Misery, east of San Jose, where slip is transferred to the Calaveras fault via a complex system of oblique slip and reverse faulting. The fault is divided into three segments: the 42-km long northern segment which extends from the northwestern margin of San Pablo Bay to the vicinity of San Leandro, the 34-km long southern segment, which extends from San Leandro to the Warm Springs district of Fremont, and the 28-km long southeast Extension segment which extends from the Warm Springs District to the vicinity of Mount Misery (Bryant and Cluett, 2000). The segment closest to the project site is the northern Hayward fault. Based on earthquake focal mechanisms, the Hayward fault is near vertical. However, secondary fault traces are common and exhibit west-vergent structures that may merge at depth with the Hayward fault. Geomorphic evidence for the northern segment of the Hayward fault includes, offset stream channels, sag ponds, and distress to cultural features as a result of fault creep.

A definitive recurrence interval has not been developed for this segment, but Lienkaemper and others (1997) concluded there have been at least four surface rupturing earthquakes in the last 2,250 radiocarbon years. The most recent event has been dated as after 1640 AD, but before 1776 AD. The 1836 magnitude 6.8 earthquake was originally believed to have occurred on this segment of the Hayward fault, however, recent work by Toppozada and Borchardt (1998) re-evaluated this earthquake to locate the epicenter further south, east of Monterey Bay.

The latest assessment of the U.S. Geological Survey Working Group on California Earthquake Probabilities (2003) has given the northern segment of the Hayward fault an 11 percent probability of exceedance, and the Hayward-Rodgers Creek fault (in its entirety) a 27 percent probability of exceedance, in the 30 year period ending in 2031 for a magnitude 6.7 or greater earthquake. Holocene displacement on the Hayward fault is predominantly right lateral, although varying degrees of vertical slip are indicated by the presence of escarpments along the fault's surface trace. Lienkaemper and Borchardt (1992) evaluated the aseismic slip rate for the Hayward fault. They found the area with the highest aseismic slip, 9 mm/yr (along the southern Hayward fault segment in the vicinity of Fremont, California), and assumed that it represented the minimum slip rate of the fault at depth. The historic surficial slip rate measured in the Oakland area only accounts for approximately 4 mm/yr of this total. It is assumed, therefore,

that the Hayward fault in this area is locked at depth and has the potential to rupture in large events.

The approximate distances of the site to the closest known mapped active faults are summarized in Table 2 below. The distances to faults were estimated using Boore et al. (1997). Fault parameters were adapted from the California Geologic Survey (CGS) fault database (2003).

Table 2. Summary of Significant Seismic Sources within 25 km of the Site

Fault	Distance from Site km (miles)	Estimated Maximum Moment Magnitude ($M_{w, max}$)
Hayward-Rodgers Creek	3.4 (2.1)	7.3
California Gridded	5.0 (3.1)	7.0
Mount Diablo Thrust	22.0 (13.6)	6.7
Calaveras	22.8 (14.1)	7.0
Green Valley Connected	25.2 (15.6)	6.8
San Andreas	25.7 (15.9)	8.0

Earthquakes on these or other active faults (including unmapped faults) could cause strong ground shaking at the site. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site, and other factors.

The project site is not located within an Alquist-Priolo Earthquake Fault Zone (discussed in further detail in Section 5.1 below).

3.2.1 Historic Seismicity

Since the first historical reports from the late 1700s, major earthquakes have been recorded along the San Andreas fault system in the Bay Area. Table 3 presents large magnitude ($M_w \geq 6.0$) earthquakes that have occurred within 100 km of the site between 1800 and 2007. The locations of the closest historic earthquakes to the project are shown on Plate 7 – Faulting and Historical Seismicity. The computer program EQSEARCH, v3.00 (Blake, 2001) was used to compile the seismicity information presented in Table 3.

Table 3. Large Magnitude (M≥6.0) Regional Earthquakes Within 100 Miles of the Site

Epicenter Location	Date	Moment Magnitude	Distance (mi)	Distance (km)	Compass Direction to Epicenter
San Francisco	June 21, 1808	6.0	12.9	20.7	Southwest
Hayward	October 21, 1868	7.0	13.2	21.3	South
San Francisco	April 18, 1906	8.25	15.8	25.4	Southwest
San Francisco Peninsula	June 1, 1838	7.0	17.9	28.8	Southwest
Antioch	May 19, 1889	6.25	23.0	37.0	East
Mare Island	March 31, 1898	6.5	26.0	41.9	Northwest
San Jose	December 26, 1858	6.25	30.8	49.6	Southwest
Vacaville	April 19, 1892	6.5	41.5	66.8	Northeast
South Santa Cruz Mountains	October 8, 1865	6.5	42.2	68.0	South
1984 Morgan Hill	April 24, 1984	6.1	47.5	76.4	Southeast
1911 Morgan Hill	July 1, 1911	6.5	49.5	79.7	Southeast
Winters	April 21, 1892	6.25	49.9	80.4	Northeast
Loma Prieta	October 18, 1989	7.1	59.2	95.3	South
Gilroy	June 20, 1897	6.25	71.6	115.2	Southeast
Pajaro Gap	April 24, 1890	6.25	74.4	119.7	South
Pajaro Gap	October 18, 1800	7	79.4	127.8	South
Pacific Ocean	October 22, 1926	6.1	84.9	136.6	South

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The topography of the area surrounding Children's Hospital is shown on Plate 2 – Topographic Site Map. Note that Plate 2 presents a USGS topographic map, with elevations referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). The standard conversion from NGVD29 to North American Vertical Datum, 1988 (NAVD88) is to add 2.38 feet to elevations referencing the NGVD29 datum. The local topography and surface conditions at each proposed structure are discussed below.

The proposed Outpatient Center No. 2 will be located on the portion of the CHRCO campus northeast of the intersection of 52nd Street and Martin Luther King Jr. Way. The site is bounded by 52nd Street to the south, Martin Luther King Jr. Way (MLKJ Way) to the west, the existing hospital parking garage to the north, and the existing outpatient center to the east. The main hospital occupies the area south of 52nd Street and is connected to the existing outpatient

center and parking garage by a bridge at the second story. The geotechnical reports for the existing hospital structures were referenced previously in this report.

The project site is currently occupied by two parking garage access driveways and a narrow lot that contains a two-story wood-frame residential structure at its east end and driveway access from MLK Way to the west. The residential lot is bounded the north, east, and south sides by the two parking garage driveways and 52nd Street. Between the fenced residential lot boundary and the aforementioned roads are landscaping strips containing mature trees and small bushes. The majority of the residential lot consists of landscaping areas with the exception of the building footprint and paved driveway. We understand that the lot is owned by CHRCO and that the structure is currently being used as an office building.

According to the site survey, the CHRCO site is relatively flat, with a gentle slope increasing from about Elevation 88 feet at MLK Way to the west to about Elevation 92 feet at the eastern end, adjacent to the existing outpatient center. For all practical purposes, the site is nearly flat from the north to the south.

4.2 SITE GEOLOGY

The CHRCO site is located on a gradually sloping outwash plain located between the Berkeley Hills to the east and San Francisco Bay to the west. The native soils at the site consist of Quaternary age alluvial deposits likely including materials from the San Antonio and Temescal Formations, according to Radbruch and Case (1967) and Helley, Lajoie and Burke (1972). Graymer (2000) maps the site as Holocene age alluvial and fluvial deposits, as shown on Plate 5 – Geology Map.

4.3 SUBSURFACE CONDITIONS

Our subsurface investigation indicates that about 2 to 6 feet of clay fill with varying amounts of sand and gravel exists at the proposed development locations. Beneath the fill, the alluvial stratigraphy encountered across the site typically consisted of a layer of stiff lean clay, over interlayered clayey sands to sandy clays with varying amounts of gravel over stiff lean clay. The clayey sands encountered within the interbedded layer are interpreted to be the paleochannels associated with a meandering paleo-Temescal Creek. These deposits appear to be at various stages of development of the rubification process (e.g., the reddening of soils through the release and precipitation of iron as an oxide during weathering). In general, there appear to be two paleochannel deposits that may or may not be laterally continuous across the site (see Plates 8a and 8b). One is shallow and appears to be relatively laterally continuous with a greater percentage of fines, than the other deeper, more poorly graded deposit. These deposits may or may not connect at depth. One previous 150-foot deep boring on 52nd Street, PB-1 (74), shows lean clay below 50 feet to a depth of 150 feet with two clayey sand layers encountered between 102 feet and 108½ feet and 118 feet and 121 feet. The subsurface conditions at the site north of 52nd Street of the proposed structure are discussed below. The geologic units are described in order of increasing depth. Detailed descriptions of the deposits encountered in each of the borings are presented on the boring logs in Appendix A. Interpreted subsurface profiles are shown on Plates 8a through 8c.

The attached CBT and boring logs and related information depict location specific subsurface conditions, encountered during our field exploration. The passage of time could result in changes in the subsurface conditions.

4.3.1 Outpatient Center - 2

The subsurface stratigraphy at the proposed Outpatient Center - 2 north of 52nd Street is generally characterized by four layers, described below in order of increasing depth: I) undocumented fill; II) stiff to very stiff clay to silty clay/clayey silt alluvium with thin lenses of sands; III) very stiff to hard clay to clayey silt alluvium and IV) dense to very dense sand and gravel to the maximum depth explored in this portion of the campus of about 68 feet below existing site grades.

The undocumented fill was encountered below the surface topsoil and extended to depths ranging from 2 feet to 6 feet. This fill is generally characterized as dark brown to black, firm to stiff, lean clay and silt with varying amouts of sand and trace organics. The fill has a moderate to high expansion potential, with measured Plasticity Index ranging from 22 to 28 percent. Fugro does not have documentation indicating that these fills were properly compacted during placement.

Beneath the fill/topsoil is a layer of stiff to very stiff lean clay with varying amounts of sand. This clay layer was consistently observed to be about 30 to 36-feet thick in the borings and CPTs. Water content measured on samples in this layer ranged from 18 to 33 percent, with an average of about 23 percent. Undrained shear strength (S_u) measurements in this layer from unconfined compression tests and CPT correlations range from about 1.4 kips per square foot (ksf) to over 4 ksf. In Boring PB 92-1, medium dense gravels and sands wth varying amounts of silt and clay were encountered at a depth of about 13 feet, grading back to the clay profile by 29 feet below grade; However, CPT-01 was pushed relatively close to PB 92-1 and these gravels and sands were not found.

Below the stiff clay layer, we observed interlayered sandy clays and clayey sands with varying amounts of gravel. The individual clayey sand and sandy clay layers range in thickness from about 5 to 10 feet. Gradual transitions were often observed between the sandy material and the clayey material. Both the sand and the clay layers contain a trace to significant amounts of gravel. The sandy units were typically medium dense to dense, and the clay units were typically firm to stiff.

Below the interlayered sands and clays, we observed a very stiff to hard lean clay layer. This layer was typically encountered about 40 to 50 feet below grade in the more southerly borings. Approximately 50 feet below site grade, very stiff, lean clays are interbedded with dense to very dense sand or gravel lenses with significant fines (silt or clay) content. Recorded SPT N-values below 50 feet were very high, typically in excess of 30 blows per foot.

4.4 GROUNDWATER

We performed a dissipation test at CPT-02 to evaluate the groundwater depth at the time of our investigation. The dissipation test indicated a groundwater depth of approximately 17 feet bgs, or approximately Elevation 73 feet.

During Harza's 1992 investigation for the Ambulatory Services Center (location of OP1-2 building), groundwater was recorded at depths of 23 feet at Boring EB-1 (PB92-1 on Plate 3). During Harza's 1986 investigation for the Medical Office Building and Parking Structure, groundwater was measured at a depth of 12 feet bgs in Boring PB86-1 after leaving the boring open for 4 hours.

Groundwater levels have been observed to vary significantly across the the CHRCO campus during previous subsurface investigations. Groundwater was encountered as shallow as about 8½ feet below grade in historical Boring B-101 and as deep as about 40 feet below grade in Boring B-104 during Fugro's February, 2008 investigation. During Fugro's August, 2008 investigation, groundwater was encountered in Borings B-113, B-115, B-116 and B-118, at between 20 feet and 25 feet below grade. Groundwater was typically measured shortly after completing the boreholes because the borings were backfilled immediately after drilling with cement grout. Note that the borings may not have been left open for a sufficient period of time to establish equilibrium ground water conditions.

As part of the geotechnical investigation for the Patient Services Pavilion, Harza (1994) installed two monitoring wells at the existing hospital. One Monitoring Well, EB-1 (OW), is located near the southern courtyard and the other Monitoring Well, EB-6 (OW), is located in the physicians parking lot near the entrance gate. Water levels were recorded in September of 1992 (dry period), approximately 50 days after drilling, and in March of 2008 (wet period) as part of this study. Groundwater in EB-1 (OW) was measured at 22 feet (Elevation 79 feet) in 1992 and at 21 feet (Elevation 80 feet) in 2008. For EB-6 (OW), groundwater was measured at 16 feet (Elevation 86.5 feet) in 1992 and 12.5 feet (Elevation 90 feet) in 2008.

Historically, groundwater levels at the Children's Hospital site have varied considerably. Observed levels range from 6 feet below grade in Boring PB-3 (86b) (K876-3, 1986), at the Family House east of Dover Street, to 23 feet below grade in Boring PB-5(86a) (K928-1, 1986), located in the current parking garage area. Boring PB-5 (86a) was converted into a temporary observation well and groundwater was recorded at a depth of 17 feet below grade three weeks after drilling.

The CGS Seismic Hazard Zone Report for the Oakland West Quadrangle (1991) presents a map that shows approximate depths to historic high ground water. Groundwater depth contours presented on this map suggest historic high ground water at the Children's Hospital site to be on the order of about 7.5 feet below grade.

Based on the depths of observed shallow groundwater in historical borings and well measurements, and to account for varying site topography (typically less than 2 feet), we have estimated a design ground water depth of 8 feet below ground surface for use in our analyses

for the proposed OPC-2 building. This depth corresponds to about Elevation +84 feet CPT-01 and Elevation +82 feet at CPT -02. Therefore, we recommend a design groundwater elevation of +84 feet. Fluctuations in the groundwater level may occur from changes in seasons, variations in rainfall, and other factors.

5.0 GEOLOGIC HAZARDS EVALUATION

Geologic hazards can be grouped into seismic and non-seismic categories. Seismic hazards can be further subdivided into surface fault offset, ground shaking, seismic ground deformation and water movements (tsunamis & seiches). Seismic ground deformation includes liquefaction, dynamic densification and landsliding. Certain types of ground deformation and water movement, however, can also occur under non-seismic conditions, so there is some overlap between seismic and non-seismic hazards. In general, the list of potential geologic hazards can also include such rare occurrences as mine collapse, hydrocompaction and peat oxidation. These hazards are rare and do not warrant discussion in connection with the project site as peat deposits were not encountered in the explorations. In addition, there is no mention of mines underlying the site in the literature we reviewed.

Historical evidence and the results of current seismological research indicate that at least one moderate to severe earthquake will occur sometime during the design life of the proposed development. Detailed discussions of severe ground shaking and other geologic hazards with respect to the site are presented below.

5.1 SURFACE FAULT OFFSET HAZARD

The potential for surface fault rupture at the site is low. No known active fault traces are mapped crossing the Children's Hospital Oakland site, based on existing geologic maps and reports including Graymer (2000) and Helley and Graymer (1997). The site is not located within a Fault-Rupture Hazard Zone as designated by the Alquist-Priolo Earthquake Fault Zoning Act (1972), as mapped on the official Earthquake Fault Hazard Zone Maps issued by the State of California (1982). The site is approximately 2 miles west of the closest active fault zone (Hayward fault) depicted on the Special Studies Zones map (CGS, 1982), as shown on Plate 9 – CGS Seismic Hazard Zone Map. The Fault Activity Map from the CGS digital database (CGS, 2000), as shown on Plates 6a (regional view) and Plate 6b (vicinity view), shows the Hayward fault to the east of the site.

An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site (Plate 5 – Geologic Map). The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. Based on Graymer (2000), this fault does intersect the ground surface within Pleistocene aged alluvial deposits near the site vicinity. The fault location is poorly constrained, and is not considered active, nor has it been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

No indications of surface fault displacement were found in our reconnaissance of the site. While the possibility of new fault breaks developing during future earthquakes cannot be precluded, historic occurrences of surface fault rupture have generally followed pre-existing active fault traces.

5.2 SHAKING HAZARDS

Strong ground shaking at the Children's Hospital site will likely occur during a moderate to severe earthquake occurring on one of the active Bay Area faults. Strong ground shaking can cause the structures to shake and also has the potential to induce other phenomena that can indirectly cause damage to structures. These phenomena include: soil liquefaction, seismically induced waves such as tsunamis and seiches, inundation from dam or embankment failure, landsliding, lateral spreading, differential compaction, and ground cracking. Detailed discussions of these phenomena with respect to the site are presented in the subsequent paragraphs.

Youd and Hoose (1978) compiled data related to damage caused by historic earthquakes in the San Francisco Bay region, including the 1906 San Francisco and 1868 Hayward Earthquakes. To the south, in the vicinity of the Alameda and Oakland border, "… the [1906] earthquake did not produce, relatively speaking, much destruction in these [Oakland, Alameda, and Berkeley water systems] works." Damage was limited to minor settlement and pipeline breaks, stream bank failures, and minor lateral displacements on the order of 8 inches. No features were noted in the project area as a result of the 1906 San Francisco Earthquake. In 1868, "…portions of the wharves were carried away in some instances, while walls were cracked in almost every house". No features were noted in the Children's Hospital vicinity or surrounding areas as being related to liquefaction.

Liquefaction data are also available for the 1989 Loma Prieta Earthquake. Tinsley and others (1998) mapped liquefaction features nearby in Berkeley and Emeryville. Damage included sand boils, cracks, ground settlement, lateral spreading and pipeline breaks. "Lateral spreading and settlement caused extensive pavement damage to the freeway road surface. Pavement cracking was oriented parallel to the shoreline, with a total lateral movement of 30 to 120 mm. Several cracks were more than 50 m long." No other features indicative of liquefaction or lateral spreading were identified in the vicinity of the Children's Hospital site as a result of the 1989 Loma Prieta Earthquake.

5.2.1 Liquefaction

The susceptibility of a soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in a) distribution of grain size, b) soil density, c) aging, d) cementation, e) fines content, and f) plasticity characteristics of the fines. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface, a depth usually considered to be less than about 50 feet.

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers located close to the ground surface. When very loose to medium dense saturated granular soils are shaken, their tendency to contract and compress may lead to the development of excess pore pressures. If the seismic shaking is strong enough and duration long enough, the buildup in pore pressure can produce a significant loss of shear strength. Downward movement (settlement) can occur as a result of soil strength loss below foundations bearing on liquefied material, or as a result of the excess pore water pressure dissipation and redistribution of the soil particles to a denser configuration.

Liquefaction is said to occur when the excess pore pressure equals the initial effective stress in the soil. If the shaking continues after the onset of liquefaction, ground distress may occur (e.g., sand boils, settlement, lurching, and lateral deformation). Liquefaction may also cause a loss of capacity of shallow or deep foundations, loss of lateral capacity or stiffness, and lateral ground spreading.

The liquefaction susceptibility of the project site is considered to be moderate, based on the Association of Bay Area Governments (ABAG) Liquefaction Susceptibility Map (2000). Witter and others (2006) mapped the quaternary deposits and evaluated the liquefaction susceptibility for the San Francisco Bay Area. The map shows the soils at the site as Holocene aged alluvial fan deposits, and as such, the liquefaction susceptibility is dependent on the depth to groundwater in the vicinity. The liquefaction susceptibility is considered moderate for groundwater depths of 10-30 feet below ground surface and high for groundwater less than 10 feet below ground surface. The site has a moderate to high liquefaction susceptibility as mapped by Witter and others (2006). The site is located in an area designated as a zone of required investigation for liquefaction by California Geological Survey (CGS), as shown on the Seismic Hazard Zones Maps, West Oakland Quadrangle (CGS, 2003), included as Plate 9 – CGS Seismic Hazard Zone Map.

We performed CPT-based liquefaction analyses using the procedures recommended by the National Center for Earthquake Research, and summarized by Youd et al (2001). The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.73g based on a probabilistic seismic hazard analyses. An anticipated earthquake magnitude (maximum moment magnitude) of 7.26, representing the upper bound M_{max} for a full rupture of the Hayward-Rogers Creek fault, was used in conjunction with the PGA value assigned in the liquefaction analyses. The liquefaction-induced settlements were estimated using the procedure of Ishihara and Yoshimine (1992).

The calculations of liquefaction potential and corresponding settlement are included in Appendix D. Our liquefaction analyses show that the materials beneath the proposed OPC-2 are generally cohesive and/or dense enough to have a low potential for liquefaction. This low potential for liquefaction in the new borings agrees well with previous liquefaction studies around the CHRCO campus that also found low potential for liquefaction. Our previous liquefaction studies included a detailed study for the western expansion of the hospital (OSHPD Permit Number HS-002307-01). This previous study included four soil borings and extensive lab testing to evaluate the liquefaction potential at the site. The study showed the clayey sand layers to be "non-liquefiable" according to the procedure by Youd et al (2001). Additionally,

liquefaction analyses of the borings performed as part of the geotechnical investigation for the medical office and parking structure by Kaldveer (1990) indicated the sand layers are medium dense to dense and liquefaction potential is remote.

5.2.2 Dynamic Densification

During a major earthquake the potential for seismically-induced settlement or dynamic densification of non-saturated soil has been observed. Soils generally susceptible to seismically-induced settlement are relatively clean, loose to medium dense, cohesionless soils. The soils encountered in our borings do not appear to be susceptible to seismically-induced settlement. Based on the densities and soil types encountered our explorations, the potential for seismically-induced settlement above the groundwater level is low.

5.2.3 Lateral Spreading

Lateral Spreading occurs where the contact between a layer of liquefiable material and the material below is sloped. Saturated sands lose their strength during an earthquake and become fluid-like and mobile. As a result, the ground may undergo large permanent displacements that can damage underground utilities and well-built surface structures. Lateral spreading involves displacement of large blocks of ground down gentle (greater than 0.1 percent) slopes or towards the open face of slopes such as a stream channel. In areas where there is no open face, buckling of the overburden is often observed.

Historic earthquakes in the Bay Area have produced lateral spreading features. However, large-scale lateral spreading is considered unlikely at this site because the site is essentially level and the probability for liquefaction at the site is considered low (discussed in Section 5.2.2). In addition, the transitions between cohesive layers and granular layers were observed to be gradual and occur at varying depths. Liquefaction-induced lateral spreading along the contact between cohesive materials and granular materials is unlikely. Liquefaction induced lateral spreading into buried paleochannels is unlikely because the paleochannels are typically dense and have a low potential for liquefaction, as discussed above.

5.2.4 Tsunami and Seiche

During a major earthquake, strong waves such as tsunamis or seiches may be generated in large bodies of water and may cause damage to structures affected by them. A tsunami (or seismic sea wave) is an open ocean phenomenon caused by faulting, volcanism or other abrupt movements on the ocean floor. A seiche is a wave that occurs in an enclosed basin as a result of fault displacement in the basin bottom, large landslides into the basin, or from periodic oscillation or sloshing of the water in the basin in response to ground shaking.

The potential for tsunami and seiche hazards at this site is considered remote. The site is located at an elevation of about 100 feet according to the project survey, and about 1.8 miles east of San Francisco Bay. There are no large enclosed bodies of water in the vicinity of the site. Damaging tsunamis are not common on the California coast. Most California tsunamis are associated with distant earthquakes (often those in Alaska or South America), not with local

earthquakes. Devastating tsunamis have not occurred in historic times in the Bay Area. Because of the lack of reliable information about the kind of tsunami run-ups that have occurred in the prehistoric past, there is considerable uncertainty over the extent of tsunami run-up that could occur at the site. However, the Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region (Ritter & Dupre, 1972) does not indicate that the site is in an area that may be inundated by a tsunami with a run-up of 20 feet at the Golden Gate Bridge.

5.2.5 Landsliding

A landslide is a movement of a mass of soil down a steep slope when the soil cannot support the weight of overlying soil or rocks. Landslides vary in size and rate of movement. Slides can occur slowly over time or suddenly. Areas susceptible to landslides are those where masses of soils are weakly supported because of natural erosion, changes in ground water or surface water patterns, or human activities such as undercutting. Heavy rains or seismic shaking can also trigger landslides.

A large earthquake in the San Francisco Bay Region may cause movement of active slides and could trigger new slides. However, no significant slopes are present at the site, and evidence of previous or active slides was not observed during our site investigation. The site is not within a mapped area of existing or potential slope instability. The site is not located within a State of California designated seismically induced landslide hazard zone, as shown on Plate 9 – CGS Hazard Zone Map. The site is not located within a City of Oakland landslide Hazard Zone (Miles and others, 2001). The potential for landsliding at the site is considered low.

5.2.6 Other Shaking Hazards

Ground shaking during an earthquake could cause objects within buildings that are not rigidly attached to the building structure (such as desks and bookshelves) to undergo some differential movements with respect to the structure. Building construction should include designs that reduce such potential differential movements and the adverse effects of such movements where they cannot be prevented.

We have also considered the possibility of the occurrence of other seismic hazards. Differential compaction is considered unlikely because of the abundance of very stiff clayey soils and bedrock and low potential for liquefaction. Ground cracking and lurching can be caused by any of the phenomena discussed above.

5.3 OTHER GEOLOGIC HAZARDS AND COMPLICATING SITE CONDITIONS

5.3.1 Flooding and Inundation Due to Dam or Embankment Failure

The site is located at an elevation of approximately 100 feet (relative to NAVD88) and about 1.5 miles west of San Francisco Bay. The potential for flooding from San Francisco Bay is remote. The site is not within the flood plain of any nearby water body. The National Flood Insurance Program designates flood prone areas. The site is designated as Zone C, an area of

minimal flooding (FEMA, 1982). Flooding resulting from ponding of locally heavy rainfall was not included in our evaluation.

Dams and reservoirs, which hold large volumes of water, represent a potential hazard from failure caused by ground shaking. The site is in the path of inundation from failure of Lake Temescal dam, approximately 2 miles (3.2km) northeast of the project site (owned by the East Bay Regional Park District). The state's Division of Safety of Dams last inspected this dam in July 2002. At the time, no issues necessitating corrective action were presented and the dam was "judged satisfactory for continued operation" (City of Oakland General Plan, 2004).

5.3.2 Debris Flow

The project area is about 2 mile from the nearest zone designated as a debris-flow source area (Oakland Hills) in the Principal Debris-Flow Sources Areas in Alameda County map (USGS, 1997), and is therefore not likely to be impacted by debris flows.

5.3.3 Land Subsidence

Land subsidence is a gradual settling or sudden sinking of the Earth's surface caused by subsurface movement of earth materials. The principal causes of subsidence are aquifer-system compaction, drainage of organic soils, underground mining, hydrocompaction, sinkholes, and thawing permafrost. The conditions needed for these hazards are not known to exist within the project area.

5.3.4 Soil Expansion

The near surface clay fill soils observed within the CHRCO campus have Plasticity Index (PI) ranging from about 15 to 28 percent. The sample tested from hand auger location HA-2 indicated a PI of 23. These PI values indicate a moderate to high expansion potential. In addition, an expansion index test was performed as part of the previous study for the adjacent office building and parking structure (Kaldveer, 1990). This test showed an Expansion Index of 115 in the fill material, indicating a high expansion potential. These results suggest that the clayey soils can be classified as "expansive" as defined in Section 1803.5.3 of the 2013 California Building Code.

Expansive pressures on the floor slabs for the OPC-2 are a concern since the building will be constructed approximately at grade. In order to reduce the potential impact on foundations resulting from swelling and shrinkage of expansive materials, we conclude that the perimeter of the foundations should be deepened to bear below the depth subject to swelling and shrinking. In addition, a layer of non-expansive engineered fill should be provided beneath the interior slabs-on-grade. Expansive materials surrounding the structure may be subjected to volume changes during seasonal fluctuations in moisture content, which could affect minor slabs-on-grade and landscaping hardscape such as sidewalks and exterior slabs. We have provided recommendations to reduce the effects of swelling and shrinkage of these materials in later sections of this report.

At the southwest corner where the bio-swale is proposed, special precautions will be required to avoid water migrating below the grade beams and the sidewalk. The grade beams adjacent to the bio-swale should be extended a minimum of 3 feet below the bottom of the bio-swale and into native clayey soil. In addition, a cut-off should be provided adjacent to the sidewalk that also extends 3 feet below the bottom of the bio-swale and into the native clayey soils.

5.3.5 Corrosion Potential

Corrosion testing was conducted on samples of the surficial material from hand auger location HA-1 and HA-2 at a depth of about 1¼ feet to 2-feet at each location. The corrosion testing was conducted at CERCO Analytical Laboratory in Pleasanton, California. The corrosivity testing included resistivity analysis, chloride ion concentration, sulfate ion concentration, pH testing, and redox potentials. Based on the resistivity measurement, the soil samples were classified as "moderately corrosive." The corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. The results of the laboratory tests and details of the results are presented in the letter reports provided by CERCO Analytical, Inc., attached in Appendix C.

5.3.6 Naturally Occurring Asbestos (NOA)

Inhalation of asbestos fibers may cause cancer. Most commonly, asbestos occurrences are associated with serpentinite and partially serpentinized ultramafic rocks. Asbestos occurs naturally in certain geologic settings in California. Exposure and disturbance of rock and soil that contains asbestos can result in the release of fibers to the air and consequent exposure to the public. Asbestos most commonly occurs in ultramafic rock that has undergone partial or complete alteration to serpentine rock (proper rock name serpentinite) and often contains chrysotile asbestos. In addition, tremolite, another form of asbestos, can be found associated with ultramafic rock, particularly near faults. Sources of asbestos emissions include: unpaved roads or driveways surfaced with ultramafic rock, construction activities in ultramafic rock deposits, or rock quarrying activities where ultramafic rock is present.

The bedrock underlying the site has the potential of containing asbestos fibers; however, bedrock is anticipated to be at a depth of about 300 feet below grade and construction activities should not penetrate through the Quaternary alluvium overburden and expose ultramafic bedrock. Our exploration did not expose alluvium derived from ultramafic material. The potential for NOA hazards at the site is low.

5.3.7 Radon-222

Radon gas is a naturally occurring radioactive gas that is invisible and odorless. Radon forms from the radioactive decay of small amounts of uranium and thorium naturally present in rocks and soils. Some rock types, such as black shales and certain igneous rocks, can have uranium and thorium in amounts higher than is typical for the earth's crust. Increased amounts of radon will be generated in the subsurface at locations where these rocks are found. Because radon is a gas, it can easily move through soil and cracks in building slabs or basement walls

and concentrate in a building's indoor air. Areas with higher amounts of radon in the underlying rocks and soil are likely to have higher percentages of buildings with indoor radon levels in excess of U.S. Environmental Protection Agency guidelines, and incidences of very high indoor radon levels are more likely in these areas. Long term exposures to elevated levels of radon can increase the risk of lung cancer.

In California, the Department of Public Health (CDPH), Radon Program, collects radon test data for buildings throughout the state. The data are maintained in a digital database. The database is used to help CDPH determine areas with excessive indoor radon levels, determine areas that may need testing, and inform the public of the results. This database is organized by zip code, and is **not** to be considered a site specific investigation. The United States Environmental Protection Agency (EPA) recommends that individuals avoid long-term exposures to radon concentrations above 4 picocuries per liter (pCi/L). The Oakland zip code 94118 has had three tests conducted, none returning with levels exceeding 4 pCi/L.

5.4 CONCLUSIONS - GEOLOGIC HAZARDS

Based on the results of our review and evaluation, the primary geologic hazards at the Children's Hospital campus include the high expansion potential of the clay soils, a corrosive soil environment, and the potential for moderate to strong ground shaking. Details regarding specific geologic hazards are presented above. Practically all structures within the San Francisco Bay Area will experience similar ground shaking effects during a moderate to strong earthquake.

5.5 OTHER HAZARDS AND COMPLICATED SITE CONDITIONS NOT ADDRESSED IN THIS REPORT

Under the "Exceptional Geologic Hazards and Complicated Site Conditions" of CGS Note 48 are a number of exceptional items not typically applicable statewide including:

- Phase I & II Environmental Site Assessment;
- Hazardous Materials;
- California Environmental Quality Act;
- Groundwater Quality;
- Onsite Septic Systems; and
- Non Tectonic Faulting and Hydrocollapse of Alluvial Fan Soils;

These exceptional items have not been evaluated in detail as part of this report, and are considered outside of the current scope of Fugro's work.

6.0 DISCUSSIONS AND CONCLUSIONS

We believe that construction of the OPC-2 building is feasible from a geotechnical and geological standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections.

6.1 EXPANSIVE SOILS

As discussed in Section 5.3.4, the moderate to high expansion potential of the clayey surface soils encountered onsite is typically a consideration for foundation design. These materials could be subjected to volume changes during seasonal fluctuations in moisture content. However, we anticipate the OPC-2 will consist of a slab-on-grade with a layer on a layer of "non-expansive fill" with column loads and grade beams supported entirely by the deep foundations. Grade beams should enclose the entire perimeter of the building to reduce the potential for expansion pressures on interior slabs-on-grade. Note that special design considerations will apply for the design of exterior slabs, as discussed in Section 7.6.3 - Exterior Slabs on Grade.

6.2 UNDOCUMENTED FILL

Our subsurface investigation indicates that about 2 to 6 feet of clay fill with varying amounts of sand and gravel exists at the proposed development locations. This information agrees well with the historic borings drilled at this site that also indicate a fill layer. We recommend the fill be reworked to conform to engineered fill requirements as discussed below (see section 7.2.3). We assume that the OPC-2 will be supported on deep foundations and not be affected by the undocumented fill conditions.

6.3 FOUNDATION SUPPORT

The primary consideration for foundation design of the OPC-2 building are the immediately adjacent basement levels for the OPC-1 and Parking Structure and the relatively higher column loads for the proposed OPC-2 building. To reduce the potential of overloading the adjacent basement wall, we recommend that the building be supported on a drilled pier foundation with grade beams extending below the depth of potentially expansive soil. A spread footing foundation system is not feasible because of the magnitude of the structural loads.

We explored a split-level shallow foundation option consisting of using excavation and replacement with Controlled Low Strength Material (CLSM) to extend footing bearing pressures to the bottom of adjacent existing basements and bearing the remaining footings near existing grades. However, this option is not desirable due to the potential for relatively large differential consolidation settlements between the split level footings.

We do not believe that driven piles are a feasible foundation option because the active hospital and residential neighborhood are sensitive to noise and vibration. Proprietary deep foundation alternatives may also be feasible, but research of these is beyond the scope of this study.

Drilled pier recommendations for the proposed OPC-2 are discussed in Section 7.4.

6.4 CONSTRUCTION CONSIDERATIONS

6.4.1 Existing Structures and Foundations

An existing house and buried utilities exist at the proposed OPC-2 require demolition as part of the construction. The contractor and project structural engineer should be aware that buried obstructions are likely to be encountered during construction of the OPC-2, and that foundation layout should take into account the potential for the existing obstructions.

In order to prevent "hard spots" beneath any future development at the site, all foundations should be removed to a depth at least 5 feet below the planned final grade, or a minimum three feet below the bottom of new pier caps, grade beams, or other conflicting new foundation elements, whichever is deeper (note that this requirement is not meant to extend to three feet below the bottom of the piers, which will be tipped below existing foundation elements.) The contractor should prepare detailed documentation of the location, size and depth of all foundations that are not completely removed.

6.4.2 Earthwork

If earthwork is performed during the dry season, moisture conditioning will be required to raise the in-situ moisture contents to near optimum moisture content (per ASTM D1557). If earthwork is performed during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

6.4.3 Height Restrictions During Construction

We understand that the hospital may maintain operations during construction. It is possible that during construction, height restrictions for construction cranes or rigs may be imposed to maintain clearance for air traffic accessing the existing helistop/helipad structure. If height restrictions will be imposed, they should be included in construction and bid documents.

6.4.4 Construction Quality Control

Careful quality control should be maintained during construction so that the recommendations presented in this report are achieved. Construction observation is necessary to check that the quality control measures are being properly implemented.

7.0 RECOMMENDATIONS

The recommendations presented in this report should be incorporated in the design and construction of the project to help minimize soil or foundation related problems. Detailed earthwork and foundation recommendations for use in design and construction of the project are presented below. We recommend that our firm review the final design and specifications to check that the earthwork and foundation recommendations presented in this report have been

properly interpreted and implemented. Fugro can assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications.

7.1 SEISMIC DESIGN CRITERIA

The proposed structure should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practice. This section presents seismic design parameters for use with the 2013 California Building Code (CBC). Based on the site geology and available subsurface information, the site may be characterized as Site Class D, described as a "stiff soil" profile. Seismic design parameters are presented in Table 5 below. The 2013 CBC Seismic Parameters presented in Table 4, below, were determined in accordance with ASCE 7-10 Section 11.4.1.

Table 4. 2013 CBC Seismic Design Parameters

Latitude: 37.8375 Longitude: -122.2682	ASCE 7-05 Table/Figure	Factor/Coefficient	Value
Short-Period MCE at 0.2s	Figure 22-1	S_s	2.01g
1.0s Period MCE	Figure 22-2	S_1	0.82g
Soil Profile Type	Table 20.3-1	Site Class	D
Site Coefficient	Table 11.4-1	F_a	1.0
Site Coefficient	Table 11.4-2	F_v	1.5
Adjusted MC Spectral Response Parameters	Equation 11.4-1	S_{MS}	2.01g
	Equation 11.4-2	S_{M1}	1.23g
Design Spectral Acceleration Parameters	Equation 11.4-3	S_{DS}	1.34g
	Equation 11.4-4	S_{D1}	0.82g
Long Period Trasition	Section 11.4-5	T_L	8 sec

Based on the above seismic design parameters, structures of Occupancy Category I, II, and III shall be assigned a Seismic Design Category "E" (ASCE 7-10 Section 11.6).

7.1 SITE PREPARATION AND GRADING

7.1.1 Site Preparation

The site should be cleared of all obstructions, including concrete, asphalt pavement, buried foundations, slabs, utility lines, trees and associated root systems, and debris. Removed concrete, asphalt concrete, and baserock may be reused as fill, provided the material is broken up to meet the requirements in Section 7.1.4 - Engineered Fill Materials. Holes resulting from the removal of root systems of larger trees could extend to depths of 3 feet, and laterally to the drip line of each tree. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with suitable material

compacted to the requirements in Section 7.1.5 - Fill Placement and Compaction. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

As previously discussed, the existing wood-frame structure will be demolished prior to the OPC-2 construction. We recommend all foundations and any below grade walls be removed to a minimum depth of 5 feet below final planned grade, or a depth of 3 feet below new foundation elements such as pier caps and grade beams, whichever is deeper. The contractor should prepare detailed documentation of the location, size and depth of all foundations that are not completely removed.

Prior to backfilling any basement level, holes should be drilled in the floor slab of the basement to prevent water from becoming trapped in the buried concrete structure. If the basement level is not completely removed, the buried concrete will act as a "bathtub" and will collect water. In order to prevent this water buildup, we recommend that holes be created in the floor slab to allow water to pass through. We recommend the holes be installed on a grid pattern with a maximum center-to-center spacing of 10 feet. The holes should be a minimum of 6-inches in diameter.

After clearing, the portions of the site containing surface vegetation or organic laden topsoil should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by the Geotechnical Engineer at the time of construction. Stripped materials should be removed from the site, or stockpiled for later use in landscaping, if approved by the owner.

7.1.2 Subgrade Preparation

Following excavation to the required grades, soil subgrades in areas to receive engineered fill (as defined in Section 7.1.4) or slabs-on-grade should be scarified to a depth of at least 6 inches, moisture conditioned to at least slightly above optimum moisture content, and compacted to at least 95 percent of the soil's maximum dry density. The top 6 inches of subgrade in areas to receive pavements or shallow spread footings should be moisture conditioned and compacted to at least 95 percent of the soil's maximum dry density. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. If the subgrade is allowed to become dry, it should be moisture conditioned to eliminate shrinkage cracks.

In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry. Conversely, it may require that scarification and aeration be performed in any soils that are too wet. There was no indication of weak or unstable soils in our borings; however, after the removal of existing building and pavements, the exposed subgrade materials may be above their optimum moisture content, and may be locally unstable. If areas of unstable soils are encountered at the time of construction, the geotechnical engineer should review conditions in the field and provide recommendations for stabilization procedures.

7.1.3 Reworking Undocumented Fill

As previously discussed, about 2 to 6 feet of undocumented fill was observed in our borings at the site. Where the fill is not completely removed by excavation, the undocumented fill material should be scarified totally removed. Excavated material may be used as engineered fill elsewhere on the site, provided the material is placed in accordance with the requirements of Section 7.1.5 – Fill Placement and Compaction.

7.1.4 Engineered Fill Materials

All fill placed at the site should consist of engineered fill meeting the requirements presented in this report, except for landscaping materials which are placed on level ground. On-site soil below the stripped layer and having an organic content of less than 3 percent by volume can be used as fill where needed except where "non-expansive" import is required beneath slabs-on-grade. All engineered fill placed at the site, including on-site soils, should not contain rocks or lumps larger than 4 inches in greatest dimension and contain no more than 15 percent larger than 2.5 inches.

"Non-expansive" fill should be predominantly granular, have an organic content of less than 3 percent by volume, should have a liquid limit less than 40 percent, have a plasticity index not exceeding 15, and should contain no environmental contaminants or debris. All imported fill should consist of "non-expansive" fill, and satisfy California Environmental Protection Agency's Department of Toxic Substances Control (Cal EPA DTSC) guidelines for clean, import fill material.

7.1.5 Fill Placement and Compaction

All engineered fill should be compacted to at least 95 percent of the soil's maximum dry density as determined by ASTM Designation D1557 (latest edition). The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent of the soil's maximum dry density. Fill material should be spread and compacted in lifts not exceeding 8 inches in pre-compacted thickness. The moisture content of the natural on-site expansive clayey soils reused as fill should be 2 to 3 percent above the optimum moisture content for the soil at the time of compaction. In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet.

7.1.6 Trench Backfill

Pipeline trenches should be backfilled with materials satisfying the criteria described above for fill, placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts may be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Onsite soil used for trench backfill should be compacted to at least 90 percent of the soil's maximum dry density by mechanical means only (jetting should not be permitted). Sand can be used for

trench backfill if the sand is compacted to at least 95 percent of the soil's maximum dry density and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill below slab and pavements should be compacted to at least 95 percent of the soil's maximum dry density.

Where utility trenches backfilled with sand enter building areas, they should be backfilled by an impermeable soil plug that is at least 2-feet thick. The plug should be located at the perimeter of the foundation. The plug should be keyed at least 12 inches into the sides and bottom of the trench and should extend to within 12 inches of the finished grade. This should help to minimize moisture change in the moderately to highly expansive clays beneath the slabs. Where sand backfilled utility trenches cross planter areas and pass below pavements or concrete sidewalks, they should be plugged as described above to minimize soil volume change below asphalt and concrete areas.

7.1.7 Surface Drainage

Positive surface gradients should be provided adjacent to the building to direct surface water away from foundations and slabs toward suitable discharge facilities. Water from roof downspouts should be carried away from the building in solid pipes and discharged into suitable drainage facilities. Surface water should not be allowed to pond adjacent to the structure or on pavements.

7.2 TEMPORARY SHORING AND CONSTRUCTION SLOPES

7.2.1 General

Construction may require either temporary slopes or shoring for excavations as needed. If site constraints are not present, temporary construction slopes may be used. However, where site constraints are present, or at the contractor's option, temporary shoring should be used. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary construction slopes and shoring system. In selecting the temporary slope, the contractor should evaluate the potential of shallow soil movement, which may potentially jeopardize the existing nearby facilities.

7.2.2 Temporary Construction Slopes

Fugro recommends that the contractor, or his specialty subcontractor, design temporary construction slopes to conform to the OSHA "Guidelines for Excavations and Temporary Sloping." The contractor, or responsible subcontractor, should determine temporary slope inclinations based on the subsurface conditions exposed at the time of construction.

For planning purposes, the onsite soils should be categorized as OSHA Type C.

If temporary slopes are left open for extended periods of time, exposure to weathering and rain could have detrimental effects such as sloughing and erosion. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes and that the temporary slopes are protected from excessive drying or saturation during

construction. No temporary excavation slopes should extend within 10 feet of existing foundations.

7.2.3 Temporary Shoring

If open-excavation is not possible because of site constraint and proximity of existing or new buildings or streets, we recommend that a temporary shoring system be used during any needed excavations. Such shoring should be sufficiently rigid to minimize damage to the adjacent facilities (streets, utilities, foundations, neighboring buildings, etc.) caused by deflection of the sides of the excavation. The temporary shoring system must be designed to include loads from the OPC-2 foundations and foundations of neighboring structures.

Fugro recommends that the contractor, or a specialty subcontractor, design and install the temporary shoring. We recommend that the geotechnical and structural engineers review the final temporary shoring plan to ensure compliance with the anticipated soil conditions encountered at the site. In addition, we recommend that a geotechnical engineer's representative observe the installation of the temporary shoring systems. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary shoring system.

7.3 FOUNDATIONS

7.3.1 Drilled Pier Foundations

Drilled pier foundations should be used to support the OPC-2 building. The drilled piers should be cast-in-place, straight piers that develop their load carrying capacity through skin friction in the stiff clay soils and interlayered clayey sands and sandy clays. Piers located in a group should have a minimum center-to-center spacing of three times the shaft diameter. The minimum pier diameter should be 24 inches. Pier reinforcing should be based on structural requirements.

The proposed OPC-2 will be constructed immediately adjacent to the existing Parking Structure and OPC-1 building, both of which contain a basement level. The foundations for the OPC-1 and Parking Structure extend down to approximately Elevation 77 and 79, respectively. To limit impacts to the adjacent buildings, piers constructed along exterior of the building adjacent to these structures should be designed as free-standing columns above the bottom of the existing foundations (i.e. ignore axial and lateral load resistance from soil above the bottom of the existing foundations). In addition, for piers near the buried diesel fuel tank, the frictional capacity above a 1.5 to 1.0 projected from the bottom of the fuel tank excavation should be ignored.

The design depth of the piers should be determined using an ultimate skin friction of 1,200 pounds per square foot (psf) in the native, upper stiff clay and the interlayered clayey sand and sandy clay from Elevation +88 down to Elevation +50. An ultimate skin friction value of 2,000 psf may be used in the very stiff to hard clay and dense granular lenses below

Elevation +50 feet. A minimum factor of safety of two should be used for design of friction piers for dead plus live loads. Table 5 below summarizes recommended skin friction for drilled piers.

Table 5. Drilled Pier Design Skin Friction

Depth Below Ground Surface (feet NAVD88)	Soil Type	Ultimate Skin Friction (psf)	Allowable Skin Friction (psf)
Above Elevation +88	Undocumented Fill	0	0
+88 to +51	Stiff Clay and Interlayered Clayey Sand and Sandy Clay	1,200	600
Below Elevation +51	Stiff Clay and Interlayered Clayey Sand and Sandy Clay	2,000	1,000

Our recommended allowable skin friction values may be increased by one-third for all load combinations that include wind or seismic loads. Uplift loads may be resisted using 80 percent of the allowable skin friction values.

Static total and differential settlements of the OPC-2 supported on drilled piers should not be significant (i.e., less than ½ inch) and within tolerable limits for the proposed structure.

7.3.2 Drilled Pier Construction

Pier excavations extending below a depth of about 10 feet from existing ground surface could encounter groundwater and may require special considerations during construction. Excavations may need to be cased, or alternatively, polymer drill slurry could used to protect the holes from caving. All pier excavations adjacent to the existing Parking Structure or OPC-1 building should be cased to prevent caving of the hole caused by nearby footing loads. Concrete should be placed in the bottom of the drilled holes using a tremie pipe. Additionally, concrete should be designed with a high slump, equal to or greater than 6 inches, to facilitate construction and reduce the potential for free water to be trapped in the pier excavation. Concrete should be placed in all piers the same day that the excavations are completed.

We recommend that the excavation of all piers be performed under the direct observation of the geotechnical engineer to confirm that the pier foundation is constructed in general accordance with the requirements presented above.

7.3.3 Pier Caps and Grade Beams

Pier caps and grade beams should be used to tie together the foundation piers. Pier caps and grade beams within 3 feet of the ground surface should be designed to resist an uplift pressure of 2,500 pounds per square foot (from highly expansive soils), with uplift resistance provided by the drilled piers. Pier caps and grade beams beneath 3 feet below the nearest adjacent soil grade can be considered in a zone with reduced potential for soil expansion and

do not need to be designed for expansion pressures. Grade beam reinforcement should be designed to meet structural requirements.

7.4 LATERAL LOADS

7.4.1 Long-Term/Static Lateral Loads

.A long-term passive pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for lateral load resistance against the face of grade beams and pier caps perpendicular to the direction of loading where the foundation is poured neat against engineered fill or undisturbed native material. Friction resistance should not be used on the bottom of grade beams or pier caps

The equivalent fluid pressure of 300 pcf can also be assumed to act against twice the project diameter of the drilled pier shaft on the upper 10 feet of the piers. Additionally, any portion of pier caps and grade beams within the upper 3 feet below grade should be neglected in the passive resistance design because of the expansive nature of the soils. If piers are spaced closer than six diameters on center, the recommended passive pressure should be reduced to account for group effects. We can provide group reduction factors during design once the estimated pier spacing and load requirements are better known.

Note that lateral deflections corresponding to a Δ/H of 0.04 (where Δ equals horizontal deflection and H equals the height of the pier cap or grade beam) are required to mobilize full passive resistance. Compatible deflection-resistance relationships must be considered when evaluating the relative contributions of passive resistance from foundation elements. The table below presents the recommended load-deflection curves to mobilize the passive resistances for different lateral deflections.

Table 6. Passive Soil Resistance (P) versus Displacement (Δ) Relationship

Deflection Ratio (Δ/H)	Passive Soil Resistance Ratio (P/P_{max})
0.00	0.0
0.01	0.5
0.04	1.0
1.00	1.0

Temporary Seismic Lateral Loads – Conventional Drilled Piers

Displacement, Moment, and Shear diagrams for free and fixed head 24 and 30 inch diameter drilled piers are presented on Plates 10a, 10b, and 10c; 11a, 11b and 11c; 12a, 12b, 12c, and 13a, 13b, and 13c respectively. Table 9 presents the maximum shear (lateral capacity) and moment values for the piers, assuming 500 kips axial compression load. Friction

resistance along the bottom of the pier caps and grade beams should be ignored in evaluating the lateral seismic resistance.

Group Effects

Our lateral capacity analyses were based on a single pier condition without considering pier group effects. For closely spaced piers, the shear planes in the soil overlap and the lateral resistance for a pier within the group is less than that of a single pier. We note that the leading piers are generally less impacted by group effects, and tend to draw higher loads. To account for the reduction of soil resistance because of group effects, we recommend multiplying the lateral loads corresponding to a given deflection by the reduction factors in the Table 7. Reduction factors, or p-multipliers, are a function of center-to-center spacing where D is the pier diameter. P-multipliers should be applied to trailing piers in the direction of loading.

As an example, a 1 x 6 pier row with a center-to-center spacing of 6 and loaded in the direction parallel to the pier row would use a p-multiplier of 1.0 for the lead pier and 0.7 for all trailing piers. The same group loaded perpendicular to the pier row would use a p-multiplier of 1.0 for all piers. Linear interpolation may be used for other pier spacing.

Table 7. Reduction Factors for Drilled Pier Lateral Load Capacity

Center-to-Center Spacing	p-multiplier
8D	1.0
6D	0.7
4D	0.4

For larger groups, we recommend multiplying the lateral loads corresponding to a given deflection by the factors in the table below. We note that the leading piers are generally less impacted by group effects, and tend to draw higher loads. The leading row will have a higher multiplier and the trailing row will have a lower multiplier than shown in the table below. For simplicity, we have presented the average p-multipliers for the proposed pier groups. If needed, more rigorous analyses of the response of each pier group under the proposed loads can be performed after the actual loads and pier layouts are determined.

Table 8. Average P-Multipliers for Various Pier Groups

Number of Rows**	PIER SPACING (S/D) 2	2.5	3	4	5
2	0.52	0.61	0.68	0.79	0.88
3	0.40	0.50	0.59	0.72	0.82
4	0.34	0.45	0.54	0.69	0.80
5	0.31	0.42	0.52	0.67	0.78

** Number of pier rows in the direction of loading

For example, a 3 x 4 pier group with a center-to-center pier spacing (S/D) of 3 would use a p-multiplier of 0.59 in the analyses for loads applied in the direction of (perpendicular to) the three pier rows. The same group would have a p-multiplier of 0.54 applied to the analyses for loads applied in the direction of (perpendicular to) the four pier rows. For new and existing piers, D may be considered to be the average pier diameter, if the pier sizes are different.

Table 9. Lateral Capacity of Conventional Drilled Piers

Load Case	Maximum Moment	Depth to Fixity
24" Free-Head, 42 kip Load	180 kip-feet	28 feet
24" Fixed-Head, 42 kip Load	170 kip-feet	25 feet
30" Free-Head 42 kip Load	180 kip-feet	30 feet
30" Fixed-Head 42 kip Load	170 kip-feet	28 feet

7.5 CONCRETE SLABS

7.5.1 Interior Slab

The ground floor of the outpatient clinic 2 should be a structural slab totally support on the grade beams and drilled piers. Appropriate slab reinforcement should be provided in accordance with the anticipated loading of the slab.

The structural slab and waterproofing may be placed directly on native subgrade soil. The subgrade should be proof-rolled (i.e., compacted) to provide a firm surface for constructing steel reinforcement and placement of slab concrete.

Sufficient slab reinforcing should be provided for an effective plasticity index of 28. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

If migration of moisture through the floor slab is undesirable, a moisture retarder system should be provided between the slab and subgrade. A high quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., .012 perms) per ASTM E 96 (e.g., 15-mil thick "Stego Wrap Class A") may be used to reduce the potential for moisture migration through the slab. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing. The vapor retarder or barrier should be placed directly under the slab, or at the structural engineer's option, the retarder may be covered with 2 inches of sand. Sand, if used, should be lightly moistened just prior to placing the concrete.

7.5.2 Exterior Slabs-on-Grade

As previously discussed, the onsite moderate to highly expansive surface soils could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs, sidewalks, and pavements should be anticipated. This movement could result in damage to the slabs, sidewalks, and pavements that might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

Exterior slabs such as sidewalks could be reinforced with steel reinforcing bars in lieu of wire mesh to minimize the impact of expansion pressures.

Walkways and pavement curbs and gutters should be supported directly on properly prepared native soils. Eliminating rock base beneath slabs will reduce the potential for migration of landscape irrigation water into pavement and walkway subgrade. Curbs should extend to the bottom of the pavement and baserock layer. One to two days prior to placing concrete, subgrade soils should be soaked to increase their moisture content to 3 to 5 percent above laboratory optimum moisture (ASTM D1557). The water content of subgrade soils should be verified by field-testing by the Geotechnical Engineer prior to placing concrete.

To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and/or a "drip" irrigation watering system be used. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the tree from slabs or pavements to reduce the effects of tree roots on these improvements.

7.6 RETAINING / BELOW GRADE WALLS

Minor retaining walls, such as landscape walls, the elevator pit and shoring for the diesel fuel tank must be designed to resist lateral earth pressures and any additional lateral loads caused by surcharging or seismic pressure. If the walls are designed to resist lateral loads through passive pressure, we recommend they be designed for the equivalent fluid pressures

discussed in Section 7.4 - *Lateral Loads*. If the retaining walls are not designed to resist passive lateral loads, they should be designed as discussed below.

The following recommendations were developed based on the clayey soils encountered at the OPC-2 locations. Retaining wall backfill less than 5 feet deep should be compacted using light compaction equipment to at least 90 percent of the soil's maximum dry density. Backfill greater than 5 feet deep should be entirely compacted to at least 95 percent of the soil's maximum dry density. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

7.6.1 Unrestrained Walls

Unrestrained walls (walls that are free to move/rotate) should be designed for active earth pressure. We recommend unrestrained walls be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot (pcf). Schematics of static and seismic lateral pressures on unrestrained retaining walls can be found on Plates 12a and 12b, respectively. Equivalent fluid pressures are based on:

- Walls backfilled with onsite, reworked lean clay fill compacted to the requirements in this report;
- The walls are fully back drained to prevent the development of hydrostatic pressure;
- The wall are capable of moving/rotating sufficiently for active pressure conditions to develop; and
- Level backfill. Walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.

In additional to soil pressure, a uniform seismic pressure of 30H (psf) should be applied to the entire wall hight, where H is the height of backfill above the top of the wall footing, in feet.

Unrestrained walls should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load. The design surcharge should include the anticpated surcharge caused by vehicular traffic and construction equipment. Surcharge loads from adjacent structures need to be considered if the proposed walls extend below the zone of influence of adjacent foundations. The zone of influence of adjacent foundations can be defined as the area below an imaginary 1.5:1 (horizontal to vertical) line extending downward from the bottom of footings near the wall.

Wall Drainage

The recommended lateral pressures for unrestrained walls herein assume walls are fully back drained to prevent the buildup of hydrostatic pressures. Adequate drainage could be provided by means of eithr weep holes with permeable materal installed behind the walls or by means of a system of subdrains.

For the subdrain system, the top of the perforated pipe should be below the bottom of the adjacent slap or grade at the toe of the wall. Drains should consist of a drain rock layer at least 12 inches thick that extends to within 2 feet of the ground surface. Four-inch diameter perforated plastic pipe should be installed (with perforations down) along the base of the walls on two-inch-thick bed of drain rock. The pipe should be sloped to drain by gravity to a suitable drainage facility. Drain rock should conform to Caltrans specifications for Class 2 permeable material. A more open-graded material, such as 3/4-inch crushed rock, could be used provided the rock is wrapped in a geotextile filter fabric (Mirafi 140 N or equivalent) to reduce the migration of fine-grained soils into the drain rock. Paving or a 2-foot thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration. Drainpipes should outlet to an appropriate drainage facility.

Alternatively, wall back-drainage can be provided by prefabricated drainage material (such as Miradrain 6000 or an approved alternative). The drainage material can be installed on the back (soil) face of the basement wall and should terminate at a 4-inch diameter perforated plastic pipe surrounded by at least 6 inches of drain rock as defined above.

7.7 BURIED DIESEL FUEL TANK

The buried diesel fuel tank will be founded about 10 feet below grade. Therefore, the tank must be designed to resist the hydrostatic uplift forced. An appropriately design hold down slab can be used to resist the hydrostatic uplift force. The fiberglass tank should be designed in accordance with the manufacturer recommendation.

7.8 SUPPORT OF MINOR STRUCTURAL ELEMENTS

Spread footings may be used to support minor structural elements such as canopies on the perimeter of the OPC-2 and the CMU walls for one-story small mechanical rooms. Conventional continuous and isolated spread footings should bear entirely on properly compacted imported non-expansive engineered fills or on firm, undisturbed native soils below the required 3-foot thick layer of non-expansive fill. No footings should bear in the undocumented fill material, where the consistency and density can be highly variable. The undocumented fill should be reworked and placed as engineered fill according to the requirements in this report, or the footings should be deepened to extend through the undocumented fill layer. All footing excavations should be examined by our representative prior to placing reinforcement.

Spread footings should be founded at least 18 inches below lowest adjacent finished grade. Footings located adjacent to other footings (i.e. existing structures) or buried utilities should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trench. Where possible, footings should not bear within the utility easement. The canopy footing adjacent to the bio-swale should be deepened such that the top of the footing is below the base of the bio-swale. Alternatively, the canopy can be supported on a drilled piers designed in accordance with the recommendation in Section 7.3.1.

Footings dimensions should be based on structural requirements. However, we recommend continuous footings be a minimum of 18-inches wide and spread footings be a minimum of 24-inches wide for ease in construction.

The following allowable bearing values may be used for spread footings. These bearings pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Table 10. Allowable Bearing Pressures for Spread Footings

Load Case	Factor of Safety	Allowable Bearing Pressure (psf)
Dead Load	3	200
Dead plus Live Loads	2	300
Total Loads (including wind or seismic)	1.5	400

Footings designed with the above bearing pressures should have long-term settlement limited to about 1/4 to 1/2 inch.

Long-term resistance to lateral loads may be provided by friction along the base of spread footings. A friction coefficient of 0.30 times the dead load may be used to evaluate the allowable frictional resistance along the bottom of footings bearing on subgrades prepared as discussed above. Friction resistance should not be used on the bottom of grade beams.

In addition to friction, a long-term passive pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for lateral load resistance against the fact of footings and grade beams perpendicular to the direction of loading where the foundation is poured neat against engineered fill or undisturbed native material. Unless confined by concrete or pavement, the passive resistance of the uppermost one foot of soil should be neglected.

7.9 ADDITIONAL GEOTECHNICAL SERVICES

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations. The analyses, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from the subsurface explorations conducted for the project, and upon the conditions existing when services were conducted. Variations of subsurface conditions from those analyzed or characterized in the report are possible, as may become evident during construction. In that event, it may be advisable to revisit certain analyses or assumptions.

We recommend that Fugro be retained to provide geotechnical services during site grading and foundation installation to observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to

modify design if unanticipated subsurface conditions are encountered. During construction, our field engineer should observe and/or test the following:

- Soil conditions exposed by site grading and foundation excavations, to check that they are consistent with those encountered during the field exploration,
- Pier foundation construction, and
- Fill placement and compaction, including backfill of utilities.

8.0 LIMITATIONS

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted, local and current geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The analyses and recommendations contained in this report are based on the data obtained from the subsurface explorations conducted for this study and relevant previous explorations at the site. These explorations indicate subsurface conditions only at specific locations and times, and only to the depths explored. Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our analyses of the observed conditions. If conditions other than those described in this report are encountered, we should be notified so that we can provide additional recommendations, if warranted.

This report has been prepared for the exclusive use of Children's Hospital and Research Center Oakland and their consultants for specific application to the Children's Hospital development as described herein. In the event that there are any changes in the ownership, nature, design, or location of the proposed project, or if any future additions are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) the project changes are reviewed by Fugro, and 2) conclusions and recommendations presented in this report are modified or verified in writing. Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

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PLATES

Source: Aerial photograph obtained from Google Earth, 04/28/2008

Site Location:
Latitude: 37.8375°
Longitude: -122.2682°

N

1:24,000

0 1,000 2,000 4,000 Feet

VICINITY MAP
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 1

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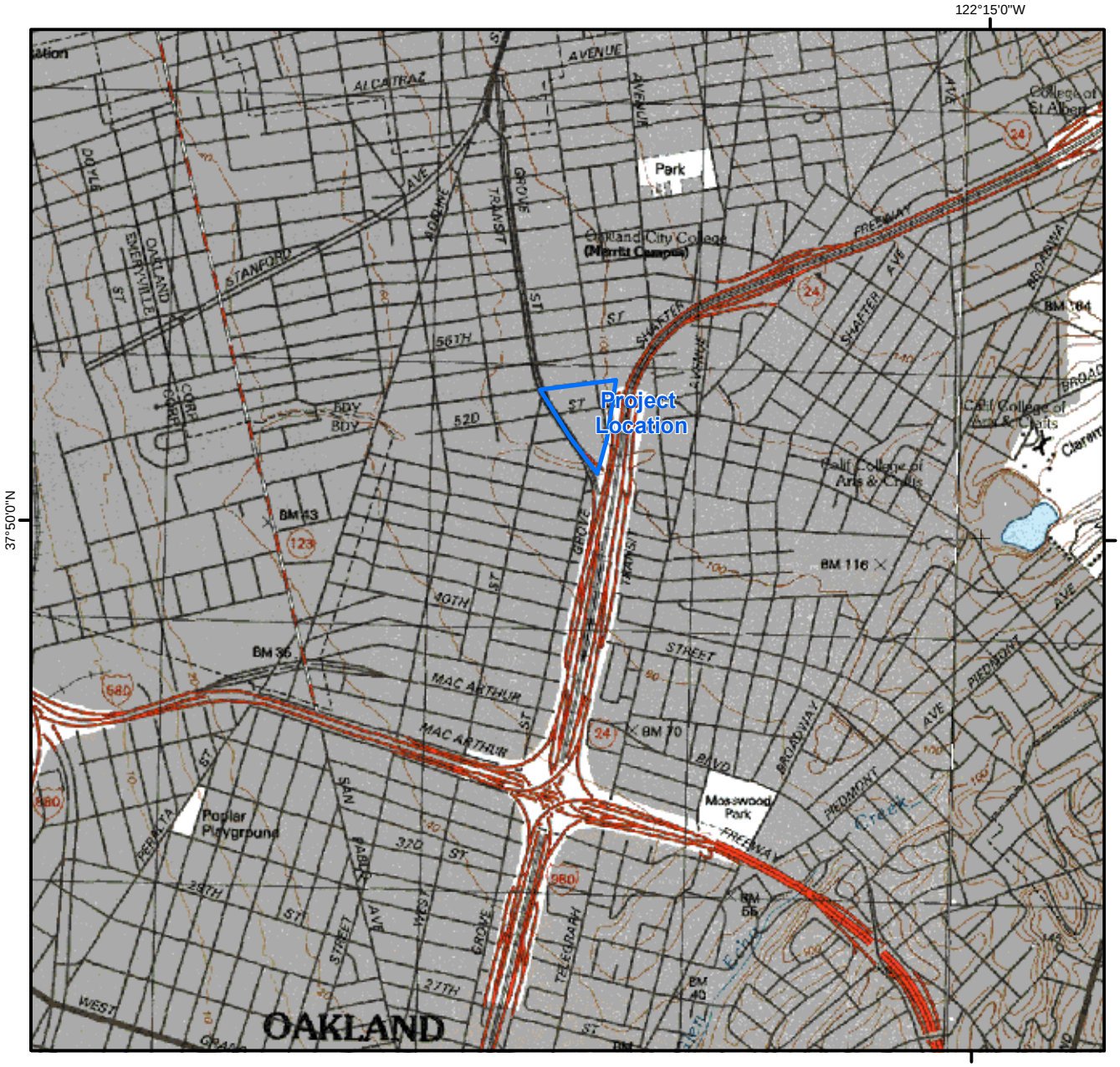


Source: USGS, Oakland West Quadrangle, California, 7.5-Minute Series (Topographic), 1993, Scale 1:24,000

Site Location:
Latitude: 37.8375°
Longitude: -122.2682°

N

1:24,000

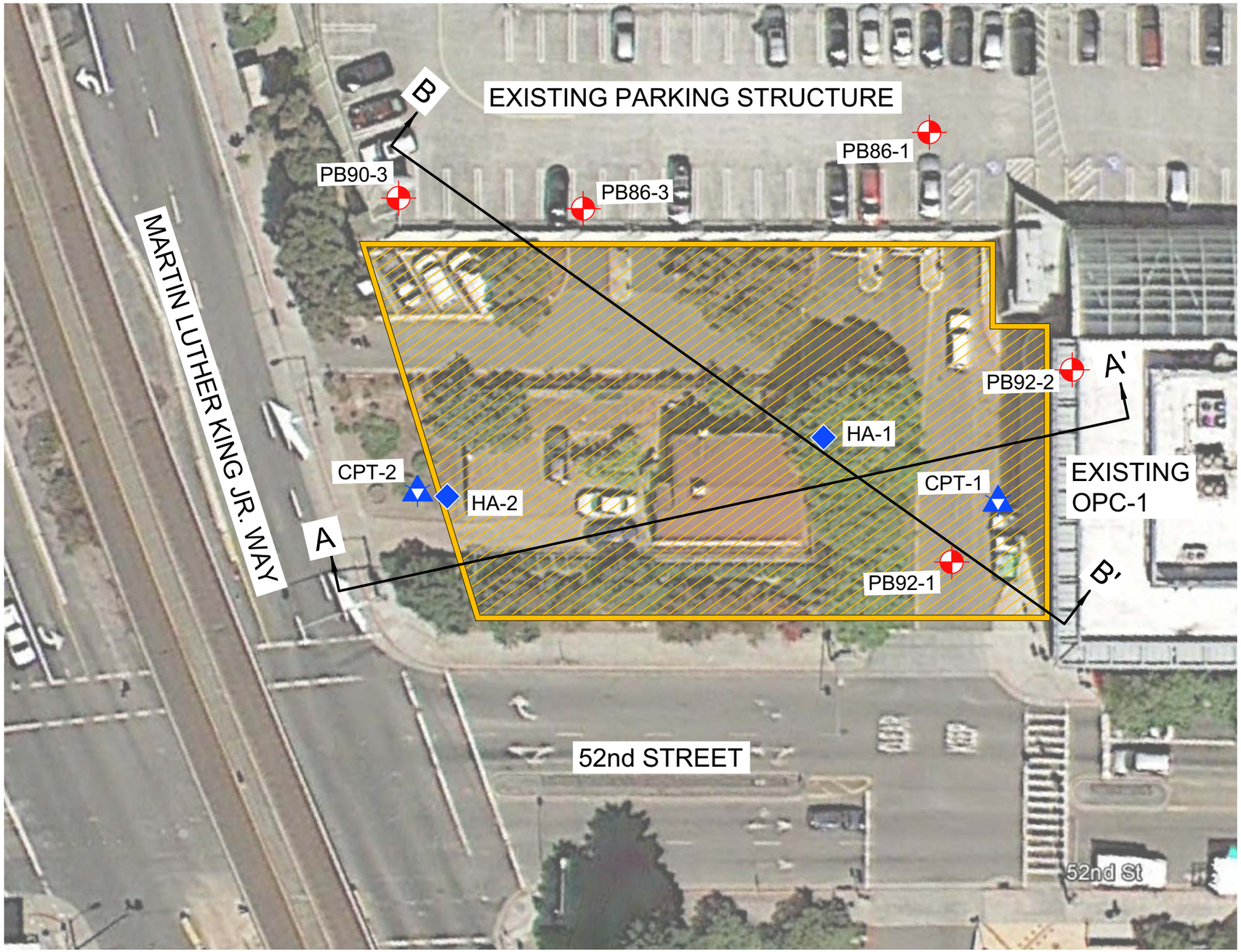
0 1,000 2,000 4,000 Feet

TOPOGRAPHIC SITE MAP

CHRCO Outpatient Center No. 2
Oakland, California

PLATE 2

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate2-Topography.mxd, 04/21/08, ksheil



LEGEND

- Proposed Outpatient Center No. 2 Building Footprint
- CPT-1 — Approximate Location of Cone Penetration Test (CPT) Sounding
- HA-1 — Approximate Location of Near-Surface Corrosion Test Sample
- PB92-2 — Previous Boring by Kaldveer Associates for Ambulatory Services Center (1992)
- PB90-3 — Previous Boring by Kaldveer Associates for Medical Office Building and Parking Structure (1990)
- PB86-1 — Previous Boring by Kaldveer Associates for Medical Office Building and Parking Structure (1986)
- B B' — Geologic Cross-section

NORTH

0 30 60
FEET

BASE MAP SOURCE: Google Earth Pro and Conceptual Layout provided by Rutherford + Chekene on June 6, 2013

SITE PLAN
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 3

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Children's Hospital and Research Center Oakland
Project No. 04.72120009

122°18'0"W 122°17'0"W 122°16'0"W 122°15'0"W 122°14'0"W

37°51'0"N
37°50'0"N
37°49'0"N

Project Location

Source: Creek and Watershed Map of Oakland and Berkeley; Sowers; 2000.

Legend

- Creeks
- Former creeks, buried or drained - dashed where channel was ephemeral
- Underground culverts & storm drains
- Engineered channels
- Flood control channels > 100 feet wide
- Water spreads over the ground
- Water seeps out of the ground
- Present watersheds
- Artificial bodies of water
- Bay or slough
- Tidal marsh, circa 1850
  - Now water
  - Now filled or drained land
- Tidal marsh formed after 1850
- Fresh water marsh, modern red=artificial, blue=natural
- Willow groves, circa 1850

N

1:24,000

0 1,000 2,000 4,000 Feet

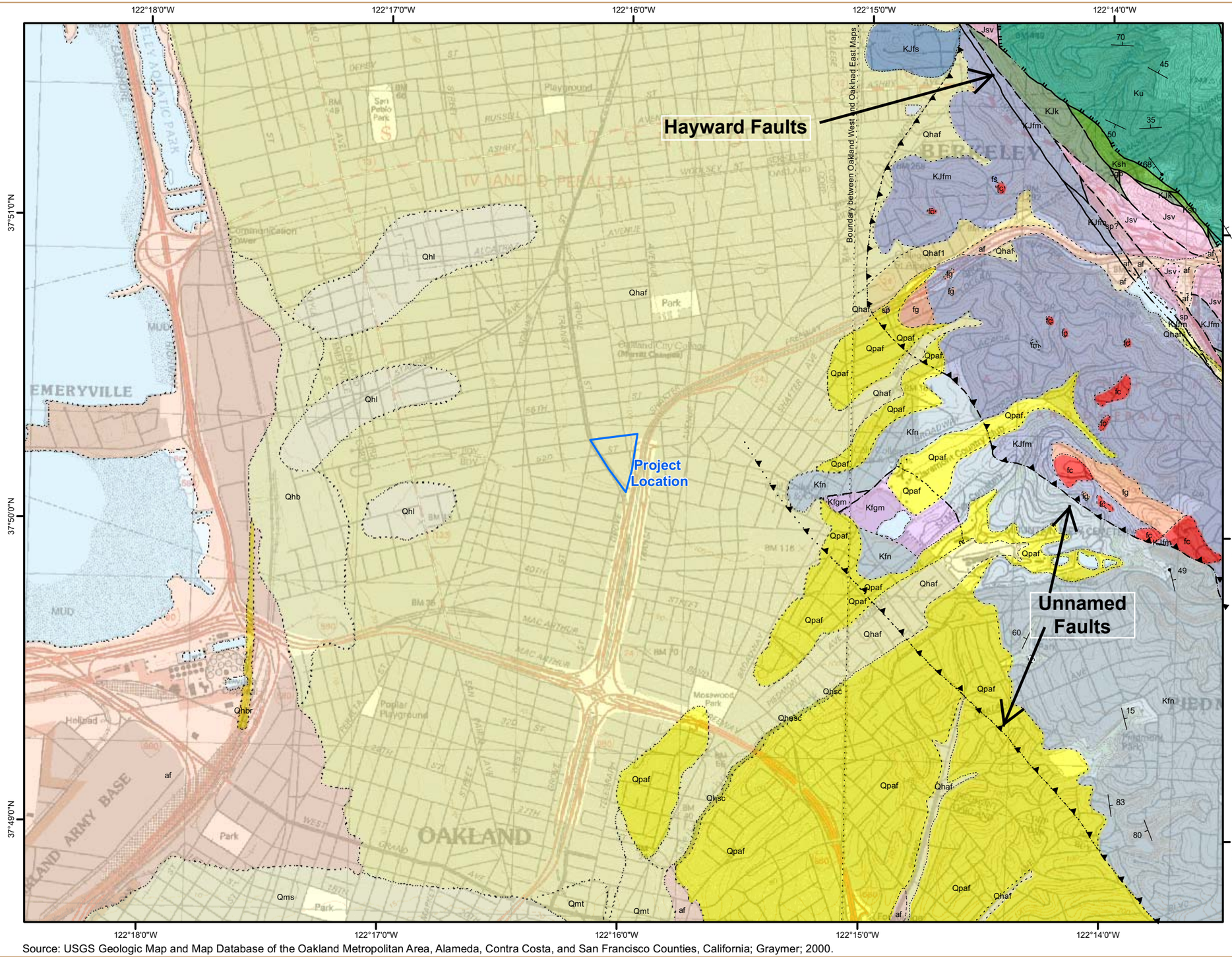
CREEK LOCATION MAP

CHRCO Outpatient Center No. 2
Oakland, California

PLATE 4

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate4-Creek_Map.mxd, 04/23/08, ksheil

Children's Hospital and Research Center Oakland
Project No.04.72120009



Legend

Contact

Faults (dashed where approx. located, small dashes where inferred, dotted where concealed, queried where location is uncertain)

Fault

Reverse or Thrust Fault

Normal Fault

70 Strike and Dip of Bedding

49 Strike and Dip of Bedding, top indicator observed

Geology

water

Surficial Deposits

af	Artificial Fill (Historic)	Qhl	Natural Levee Deposits (Holocene)
Qhasc	Artificial Stream Channels (Historic)	Qhsc	Stream Channel Deposits (Holocene)
Qhaf1	Younger Alluvial Fan Deposits (Holocene)	Qms	Merrit Sand (Holocene and Pleistocene)
Qhaf	Alluvial Fan and Fluvial Deposits (Holocene)	Qmt	Marine Terrace Deposits (Pleistocene)
Qhb	Basin Deposits (Holocene)	Qpaf	Alluvial Fan and Fluvial Deposits (Pleistocene)
Qhbr	Beach Ridge Deposits (Holocene)		

Great Valley Complex

Ku	Undivided Great Valley Complex Rocks (Cretaceous)
Ksh	Siltstone and Shale
KJk	Knoxville Formation (Early Cretaceous and Late Jurassic)
Jsv	Keratophyre and Quartz Keratophyre (Late Jurassic)

Coast Range Ophiolite

Jgb	Gabbro

Franciscan Complex

Kfn	Sandstone of the Novato Quarry Terrane of Blake and Others (1984) (Late Cretaceous)	fc	Serpentinite Blocks
Kfgm	Fine-grained Quartz Diorite (Late Cretaceous (?))	sp	Chert Blocks
KJfs	Franciscan Complex Sandstone, Undivided (Late Cretaceous to Late Jurassic)	fg	Greenstone Blocks
KJfm	Franciscan Complex, Melange (Cretaceous Late Jussaric)	fm	Schist Blocks
fs	Graywacke and Meta-Graywacke Blocks		

N

1:24,000

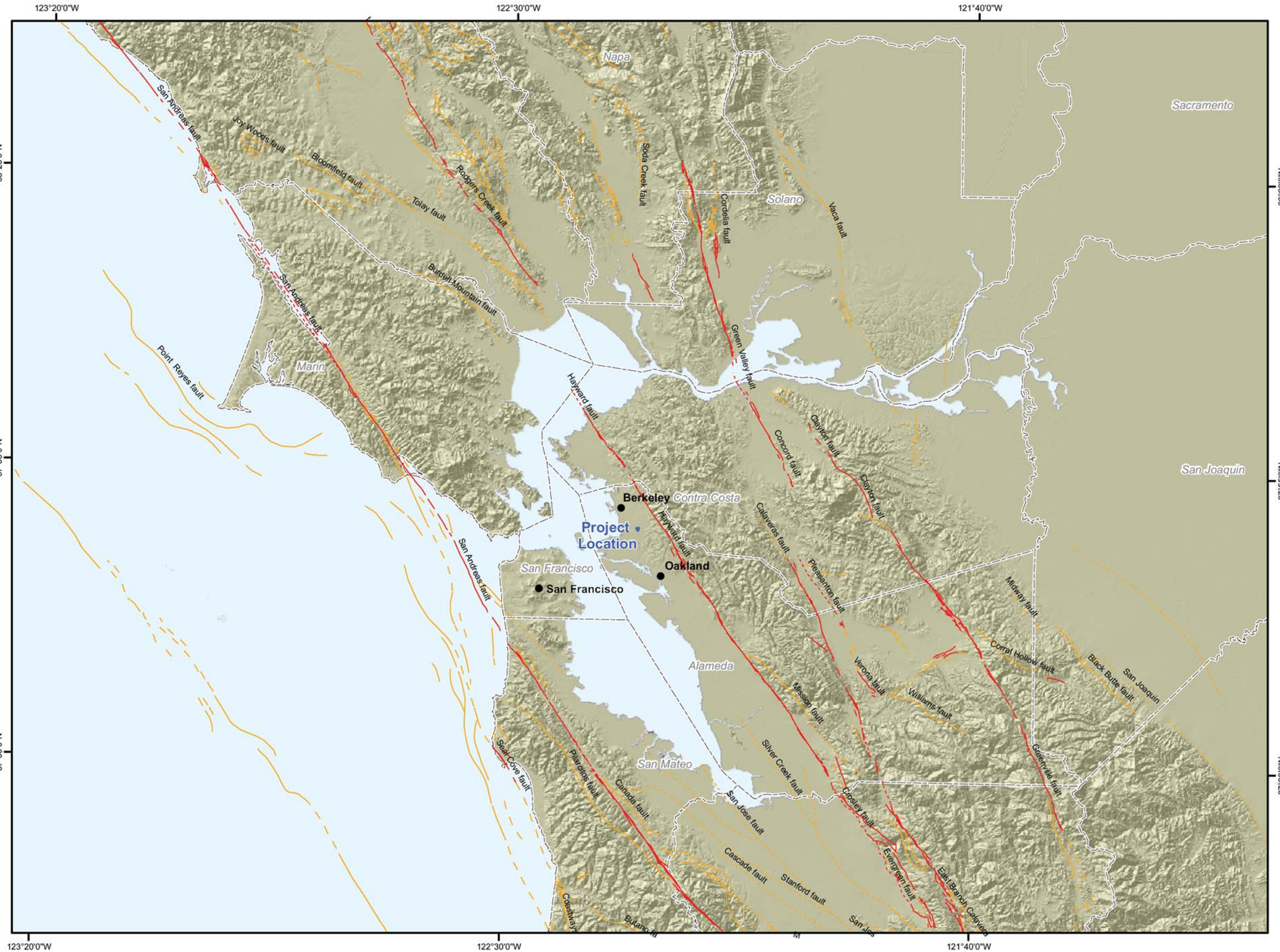
0 1,000 2,000 4,000 Feet

GEOLOGIC MAP
CHRCO Oupatient Center No. 2
Oakland, California

PLATE 5

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate5-Geology.mxd, 04/23/08, ksheil

Source: USGS Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California; Graymer; 2000.



N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate6a-Regional_Faults.mxd, 04/23/08, ksheil

Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Sources:

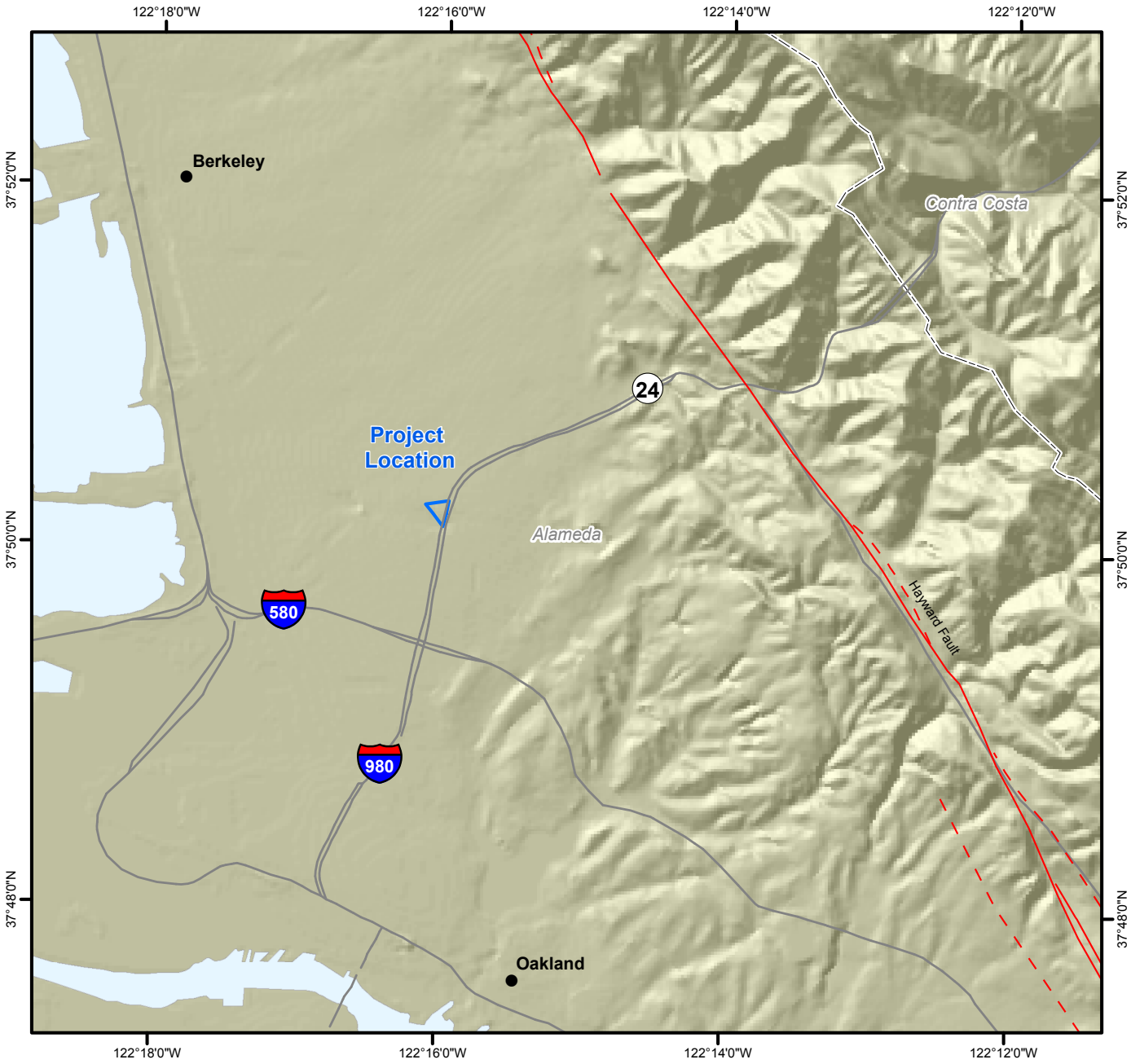
a) Bryant, 2005

b) Jennings, 1994

REGIONAL FAULT MAP

CHRCO Outpatient Center No. 2
Oakland, California

PLATE 6a



Source: Bryant, 2005; Jennings, 1994

Legend

- Major Roads
- County Limits

Faults

- Active Fault, dashed where inferred
- Potentially Active Fault, dashed where inferred

N

1:63,360

0 0.5 1 2 Miles

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\October_2008\mxd\New_Patient_Tower\Plate6b-Vacinity_Faults.mxd, 10/17/08, ksheil

VICINITY FAULT MAP

CHRCO Outpatient Center No. 2
Oakland, California

PLATE 6b

123°20'0"W
122°30'0"W
121°40'0"W
38°20'0"N
37°55'0"N
37°30'0"N

San Andreas fault
Joy Woods fault
Bloomfield fault
Tolay fault
Rodgers Creek fault
Point Reyes fault
Burdell Mountain fault
Hayward fault
Soda Creek fault
Cordelia fault
Vaca fault
Green Valley fault
Concord fault
Clayton fault
Calaveras fault
Pleasanton fault
Midway fault
Ortigalita fault
Coast Range Thrust
Cooper Hollow fault
Black Butte fault
Verona fault
Williams fault
Mission fault
Silver Creek fault
Greenville fault
Quien Sabe fault
San Jose fault
Cascade fault
Stanford fault
Evergreen fault
Berrocal fault
Coyote fault
Monte Vista fault
Shannon fault
Sargent fault
Los Gatos fault
Seal Cove fault
Pilarcitos fault
Canada fault
Butano fault

Napa
Solano
Sacramento
Marin
Contra Costa
San Francisco
Alameda
San Mateo
San Joaquin

5.6 (1969)
5.7 (1969)
5.2 (2000)
5.5(1891)
6.2(1898)
5.5(1902)
6(1889)
5.4 (1955)
5.3(1870)
Berkeley
Project Location
5.2(1889)
Oakland
San Francisco
5.4(1899)
8.2(1906)
5.3 (1957)
7(1838)
5.3(1856)
5.5(1856)
6.8(1868)
5.7(1864)
5.3(1864)
5.5(1903)
6.1(1858)
5.7 (1986)
5.4 (2007)
5.3 (1988)
5.5 (1955)
5.5(1903)
5.5(1891)
6.3(1865)
6.2 (1984)
5.8 (1980)
5.1 (1980)
5.6(1861)
5.4 (1980)
5.8(1866)
5.9(1881)

Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Historical Earthquake Magnitude (1836 - March 2008)
Magnitudes equal to and greater than 5 are labeled.

- 3.0 - 3.9
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7+

Sources:

1) Earthquake Data:
   a) Earthquake epicenters from ANSS Composite Catalog Search, 1898 to 2008, <www.ncedc.org/anss/> (downloaded March 2008)

   b) USGS Earthquake Hazards Program Search, 1735 to 1974, <http://neic.usgs.gov/neis/epic/epic_circ.html> (downloaded March 2008)

2) Faults:
   a) Bryant, 2005

   b) Jennings, 1994

N

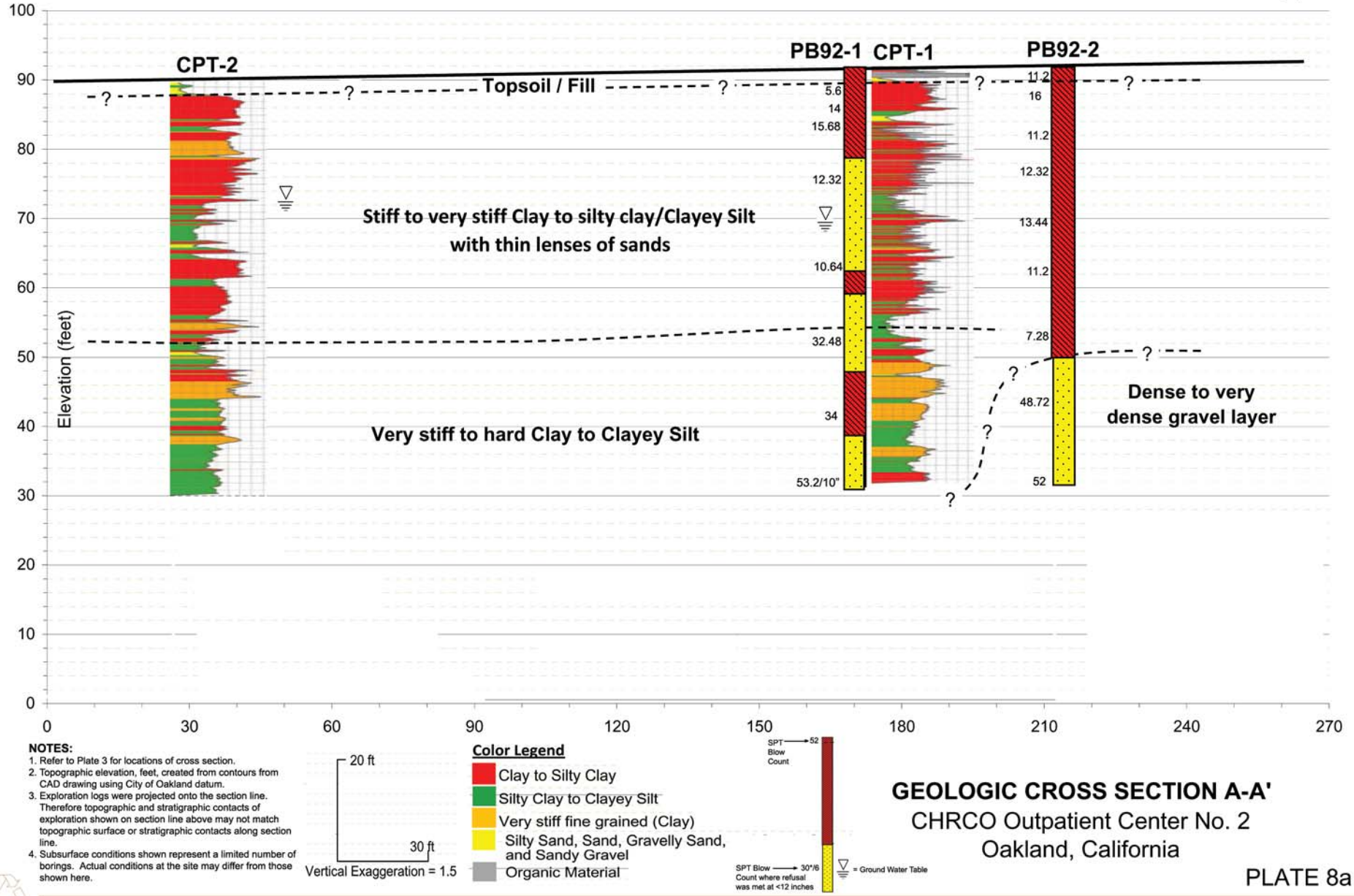
1:633,600

0 5 10 20 Miles

REGIONAL FAULTING AND HISTORICAL SEISMICITY
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 7

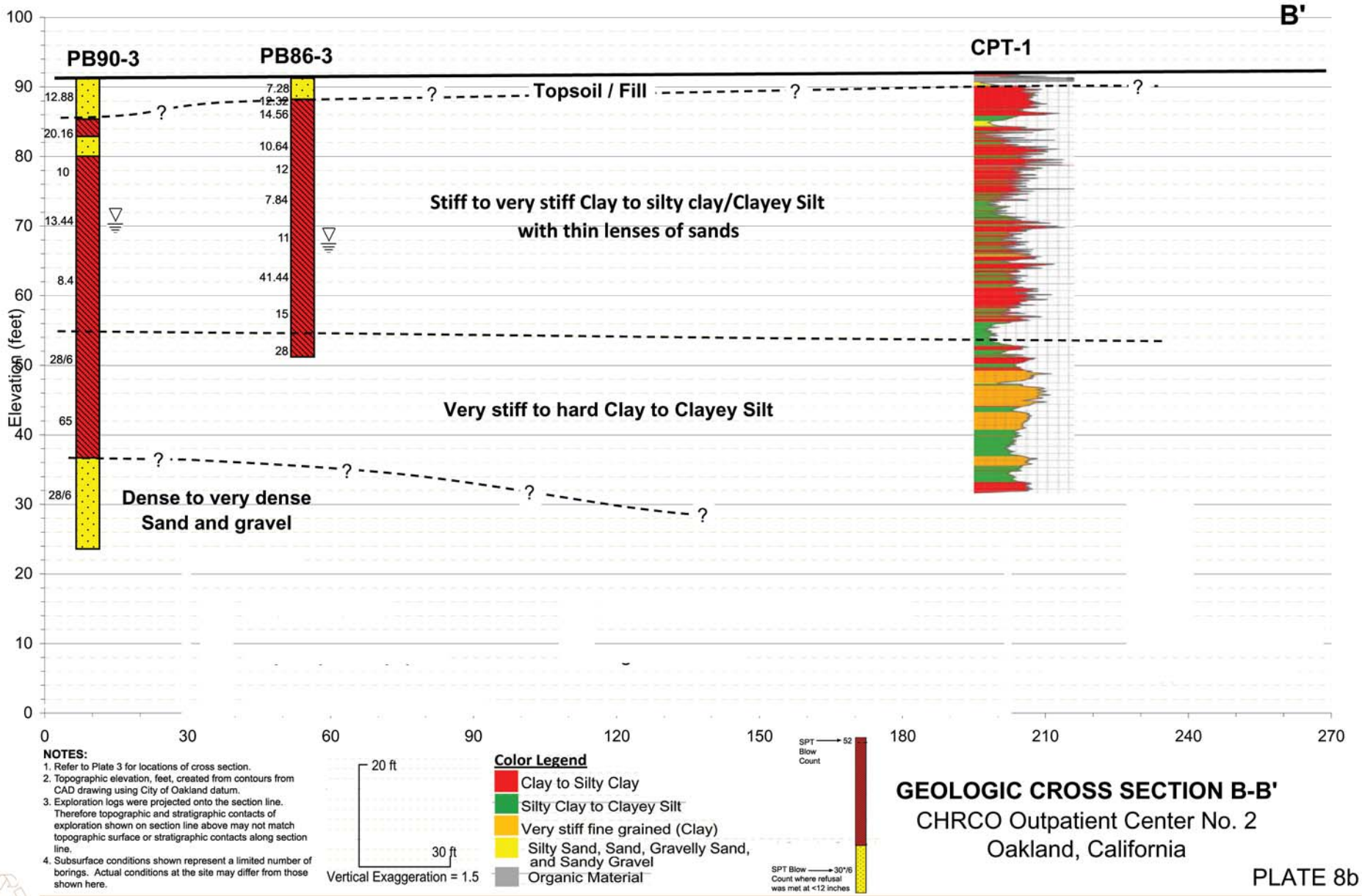
N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate7-Seismicity.mxd, 04/23/08, ksheil



GEOLOGIC CROSS SECTION A-A'
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 8a

Section B



NOTES:
1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing using City of Oakland datum.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.

20 ft
30 ft
Vertical Exaggeration = 1.5

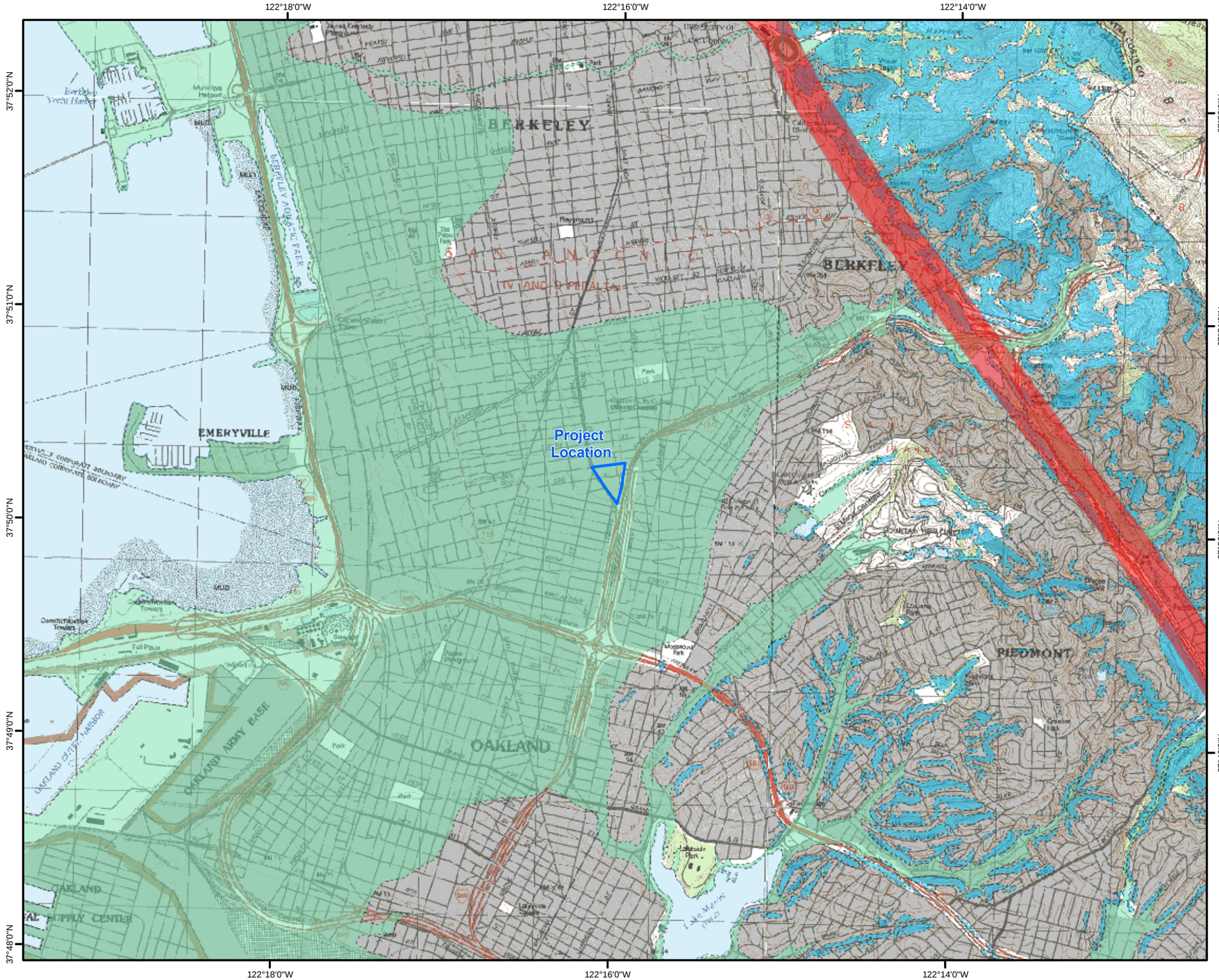
Color Legend
- Clay to Silty Clay
- Silty Clay to Clayey Silt
- Very stiff fine grained (Clay)
- Silty Sand, Sand, Gravelly Sand, and Sandy Gravel
- Organic Material

SPT Blow Count → 52

SPT Blow → 30"/6 Count where refusal was met at <12 inches

GEOLOGIC CROSS SECTION B-B'
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 8b



Legend

Liquefaction Hazard Zone
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacements.

Earthquake-Induced Landslide Hazard Zone
Areas where previous occurrence of landslide movement, or local topographic, and geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements.

Alquist-Priolo Earthquake Fault Zone

Sources:
1) Alquist-Priolo (AP) Special Studies Zone Boundaries: California Geologic Survey (2002)
2) Liquefaction and Landslide Special Studies Zones: California Geologic Survey <http://www.conservation.ca.gov/cgs/shzp/>

N

1:36,000

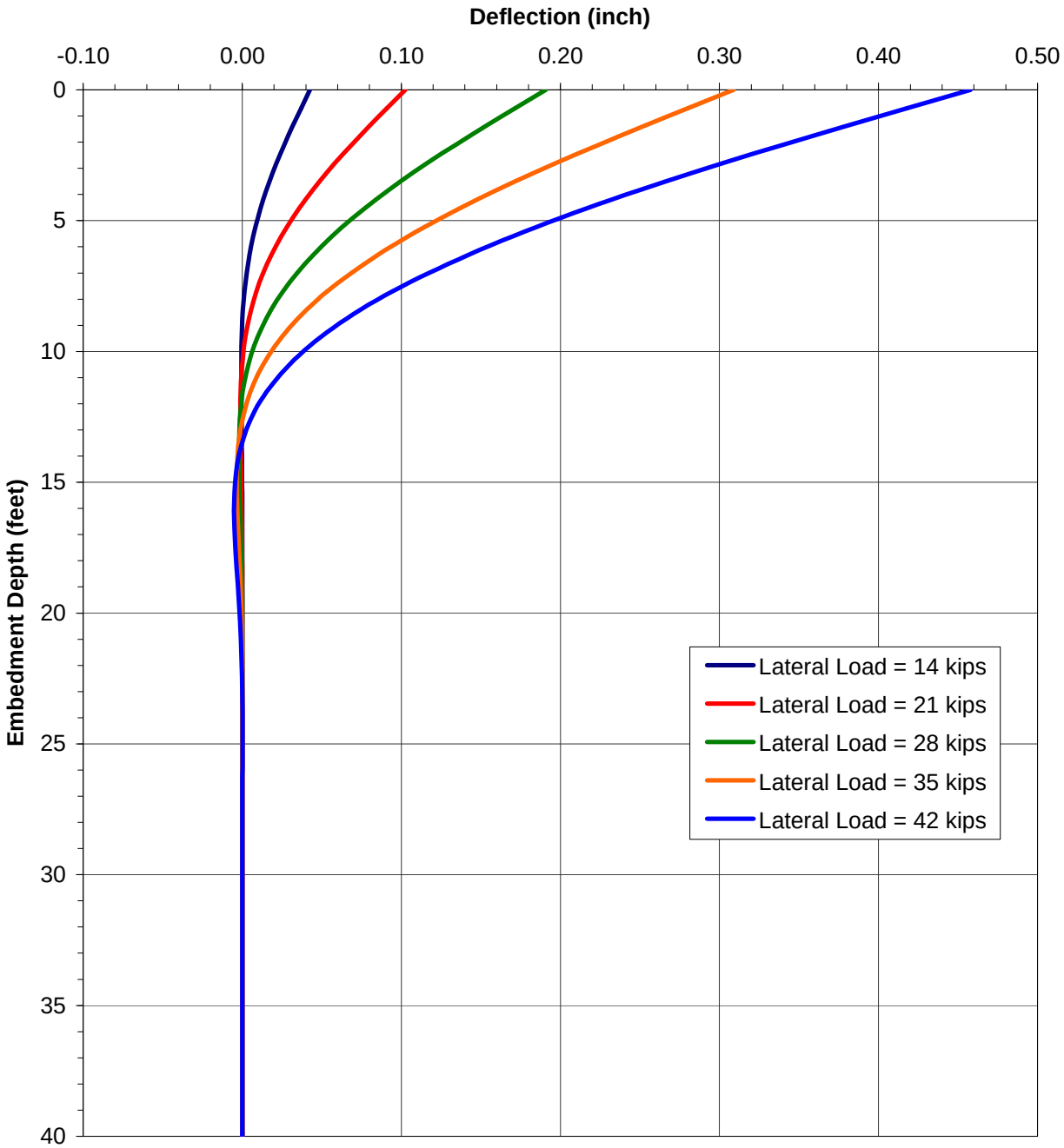
0 1,500 3,000 6,000 Feet

CGS SEISMIC HAZARD ZONE MAP
WITH ALQUIST-PRIOLO EARTHQUAKE ZONES
CHRCO Outpatient Center No. 2
Oakland, CA

PLATE 9

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate9-Seismic_Hazard.mxd, 04/22/08, ksheil

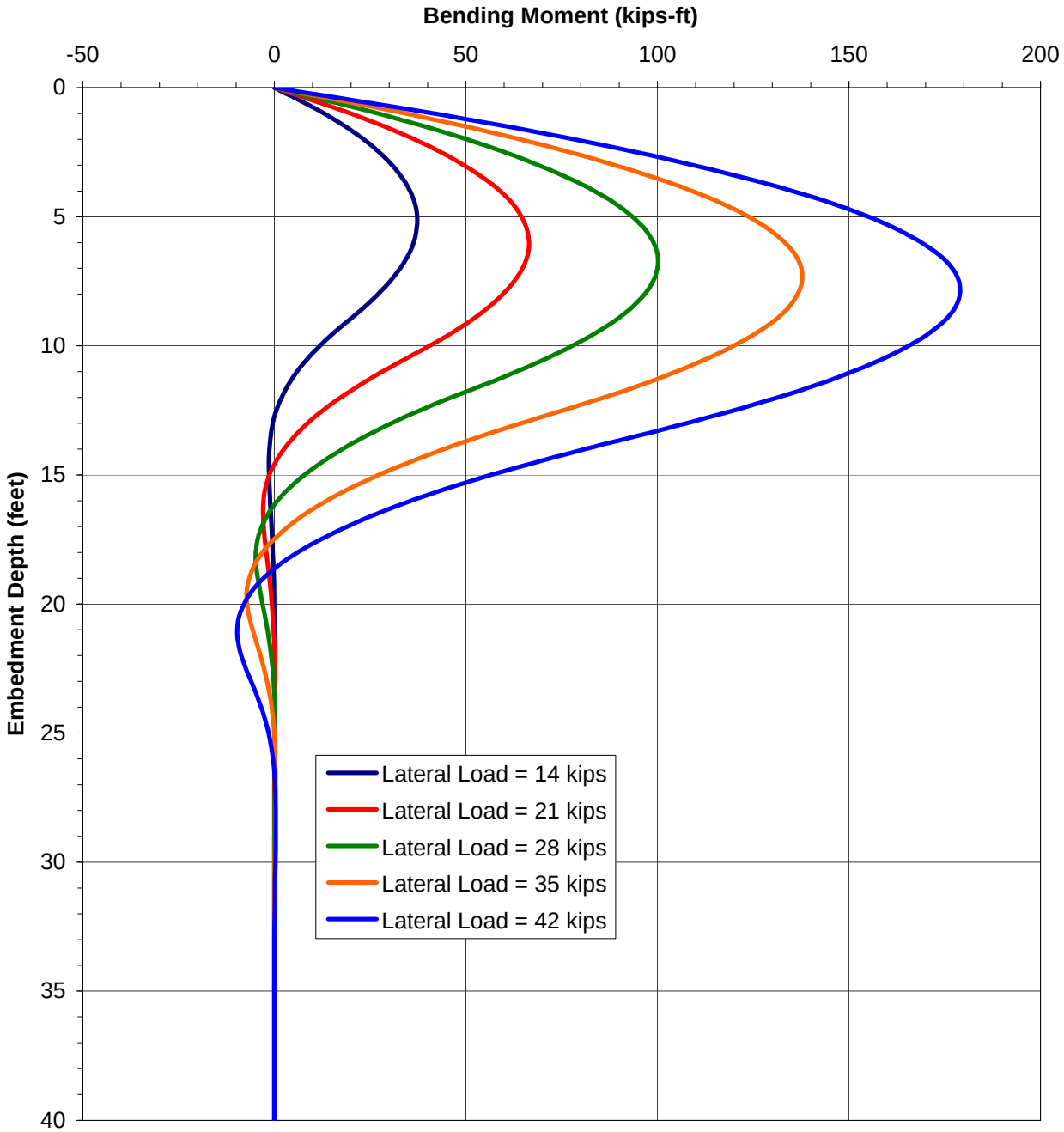
LATERAL DEFLECTION VS. DEPTH
24-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Free Head Condition



DEFLECTION RESPONSE: 24-INCH DIAMETER PIER (FREE HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 10a

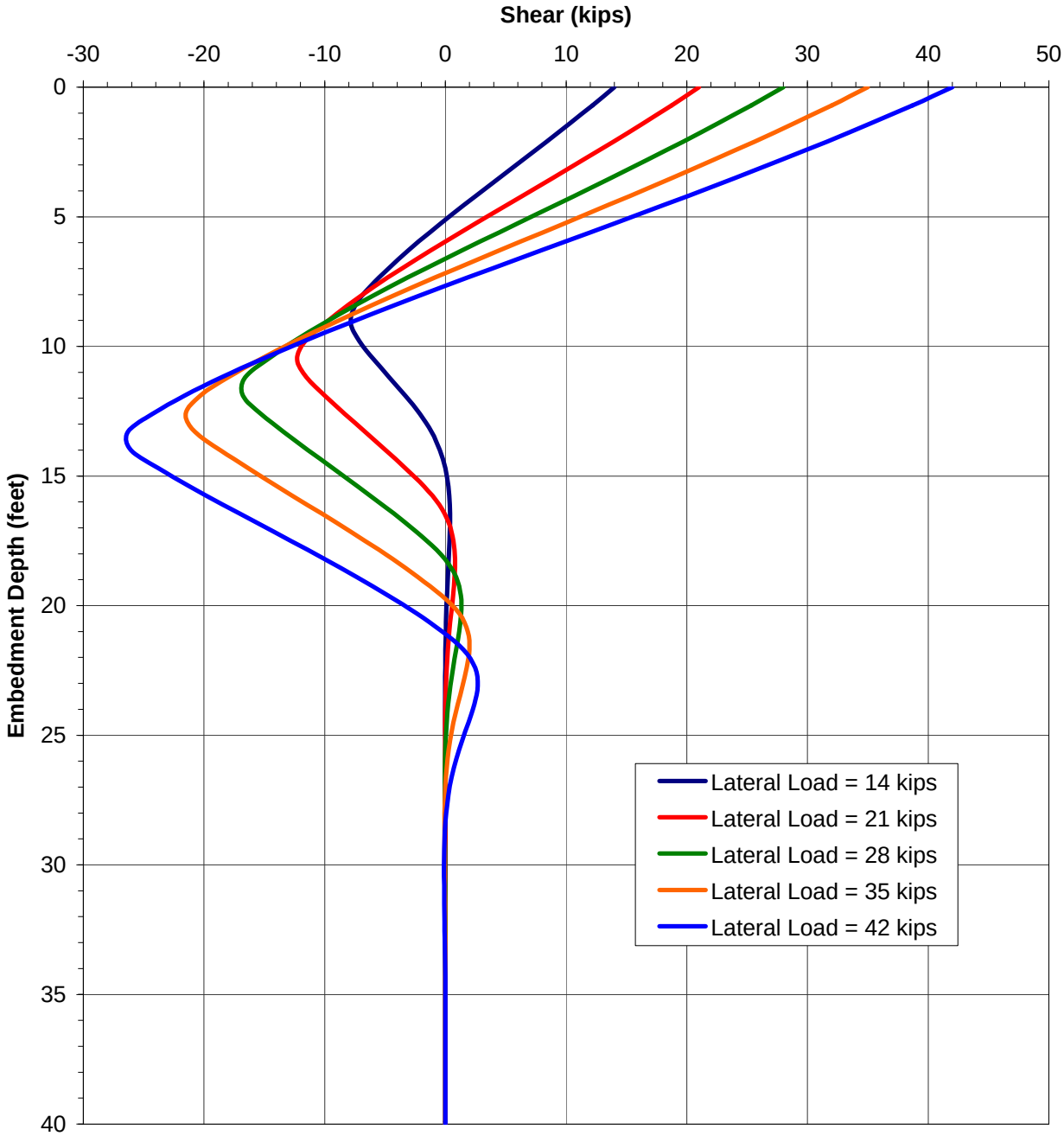
BENDING MOMENT WITH DEPTH BELOW PIER CAP
24-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Free Head Condition



MOMENT RESPONSE: 24-INCH DIAMETER PIER (FREE HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 10b

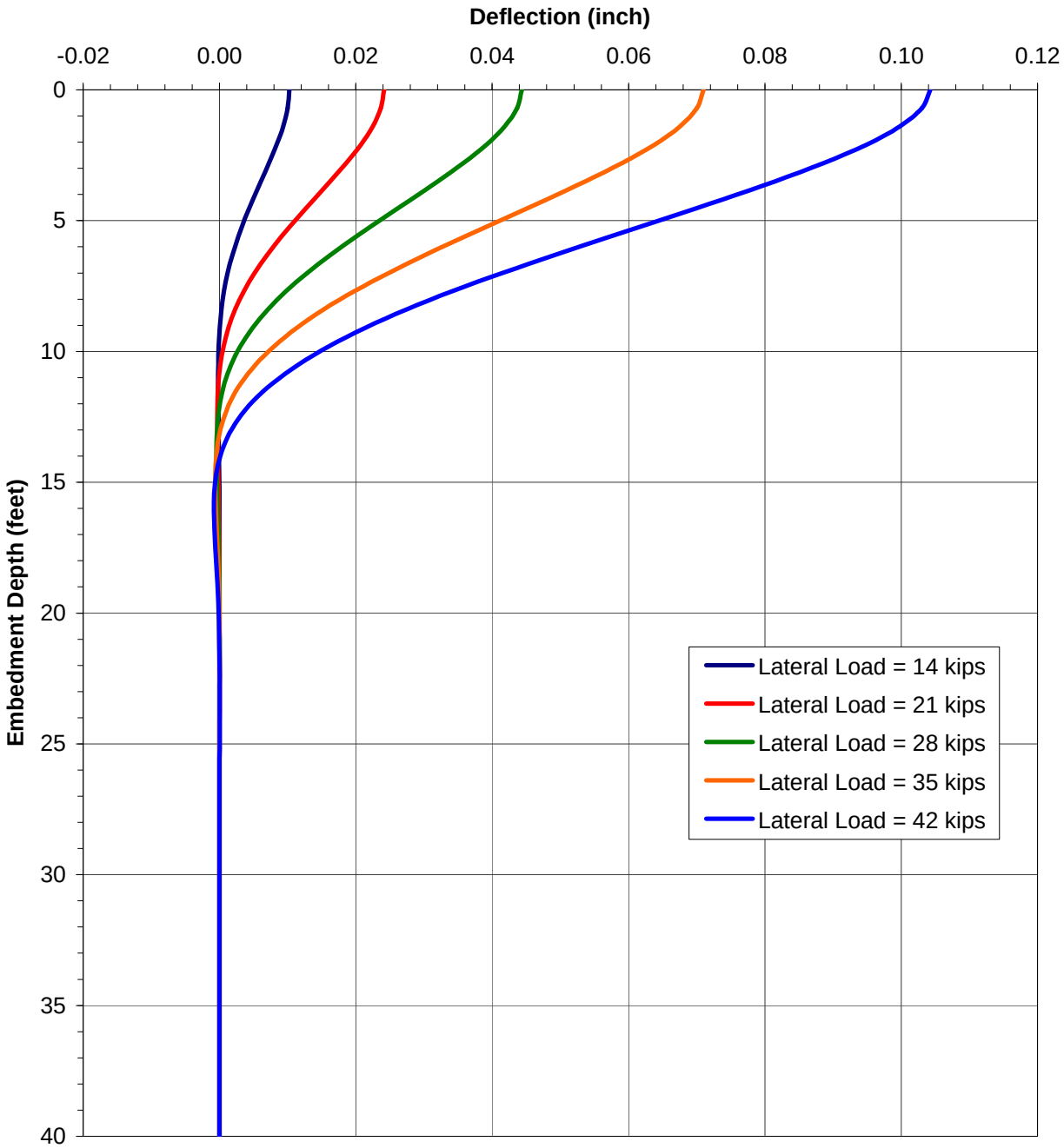
SHEAR WITH DEPTH BELOW PIER CAP
24-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Free Head Condition



SHEAR RESPONSE: 24-INCH DIAMETER PIER (FREE HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 10c

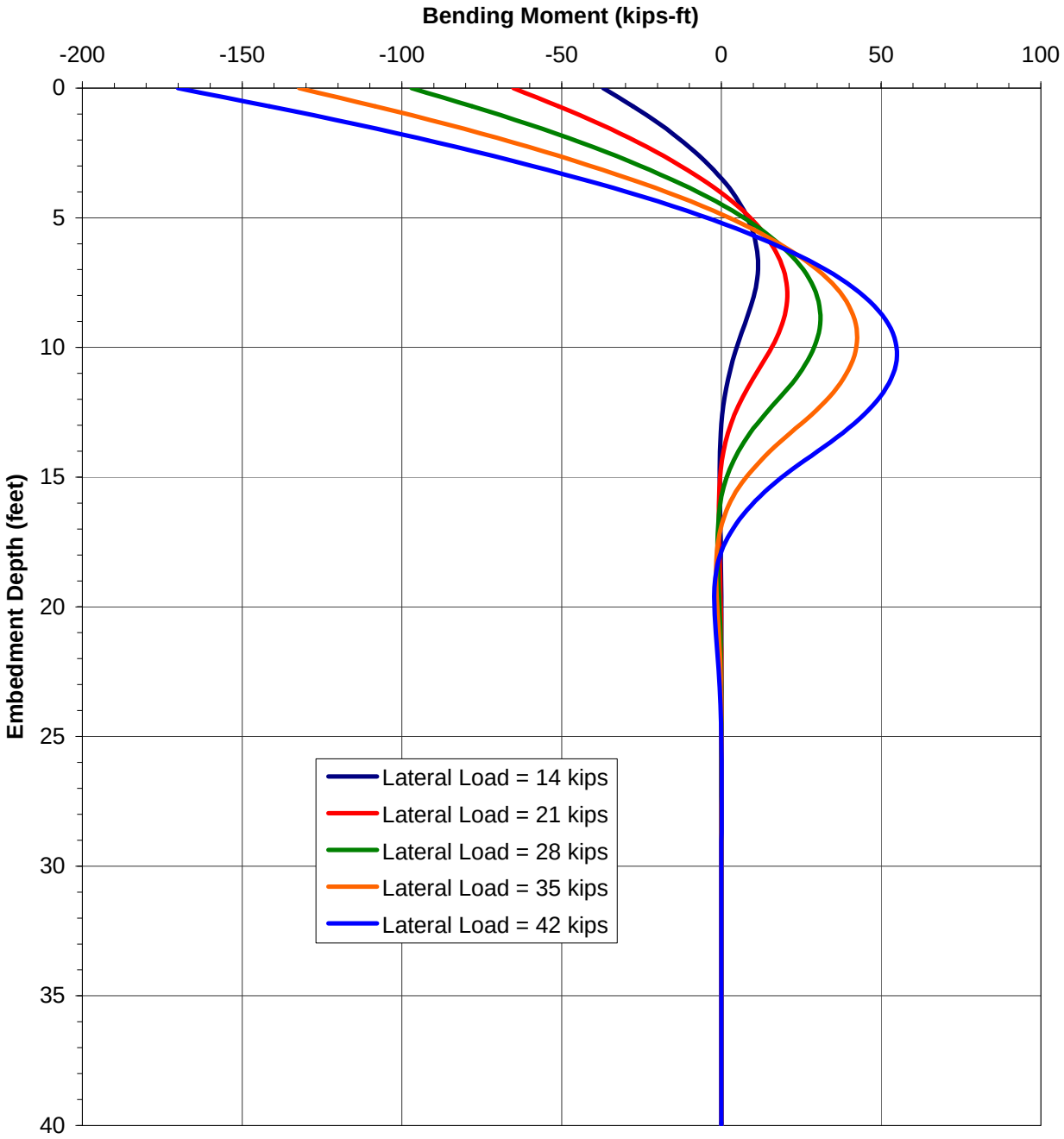
LATERAL DEFLECTION VS. DEPTH
24-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Fixed Head Condition



DEFLECTION RESPONSE: 24-INCH DIAMETER PIER (FIXED HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 11a

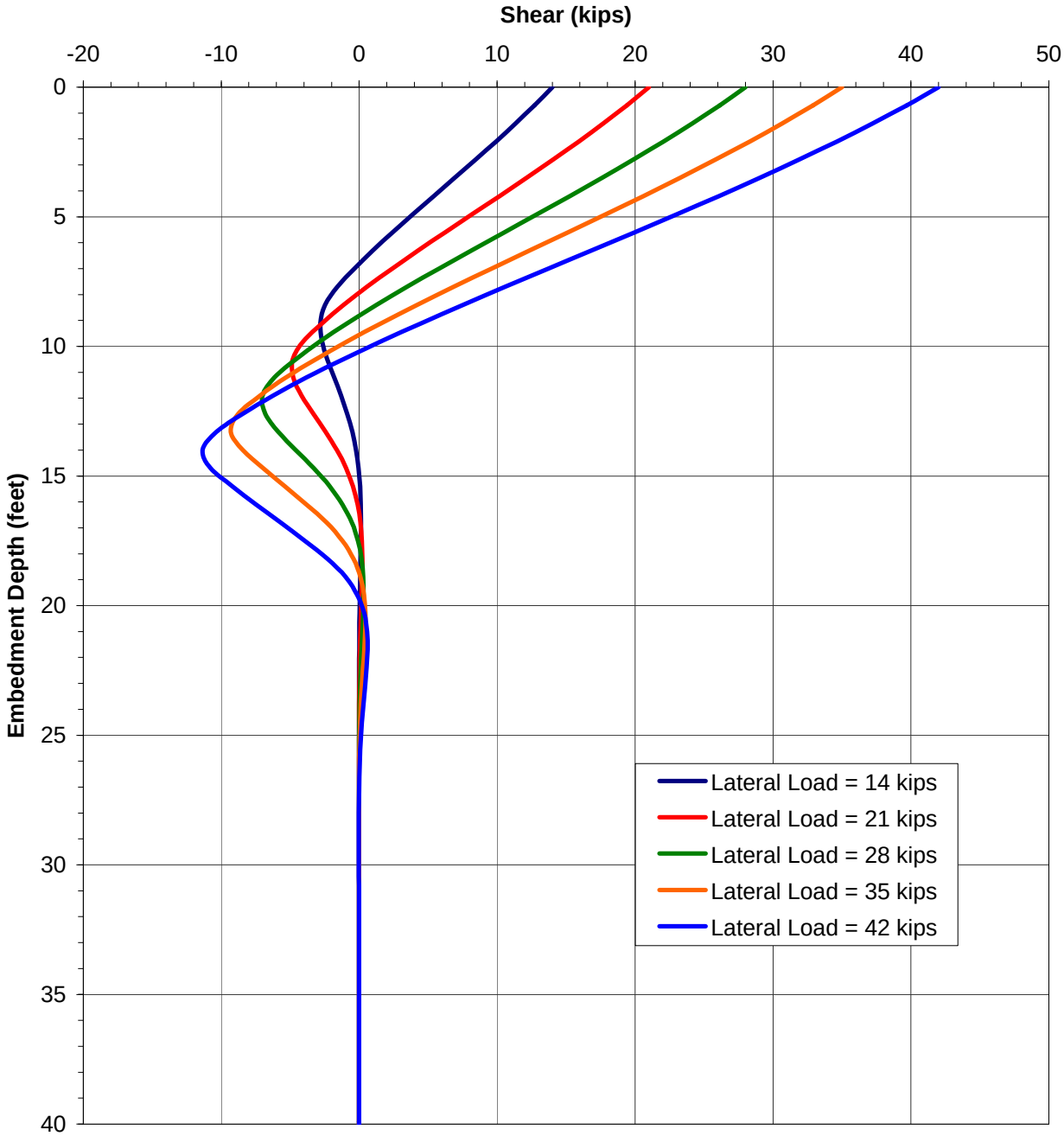
BENDING MOMENT WITH DEPTH BELOW PIER CAP
24-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Fixed Head Condition



MOMENT RESPONSE: 24-INCH DIAMETER PIER (FIXED HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 11b

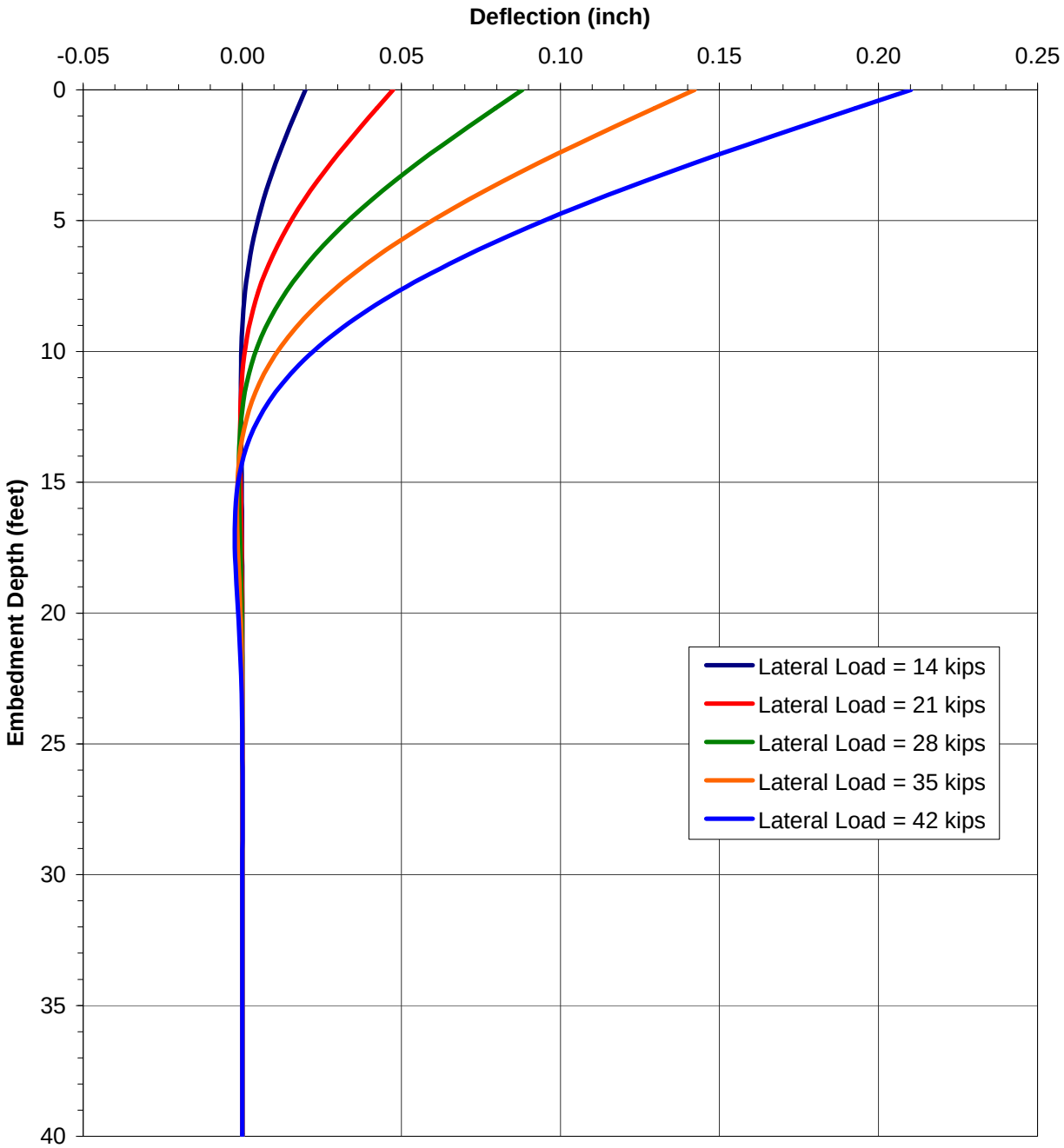
SHEAR WITH DEPTH BELOW PIER CAP
24-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Fixed Head Condition



SHEAR RESPONSE: 24-INCH DIAMETER PIER (FIXED HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 11c

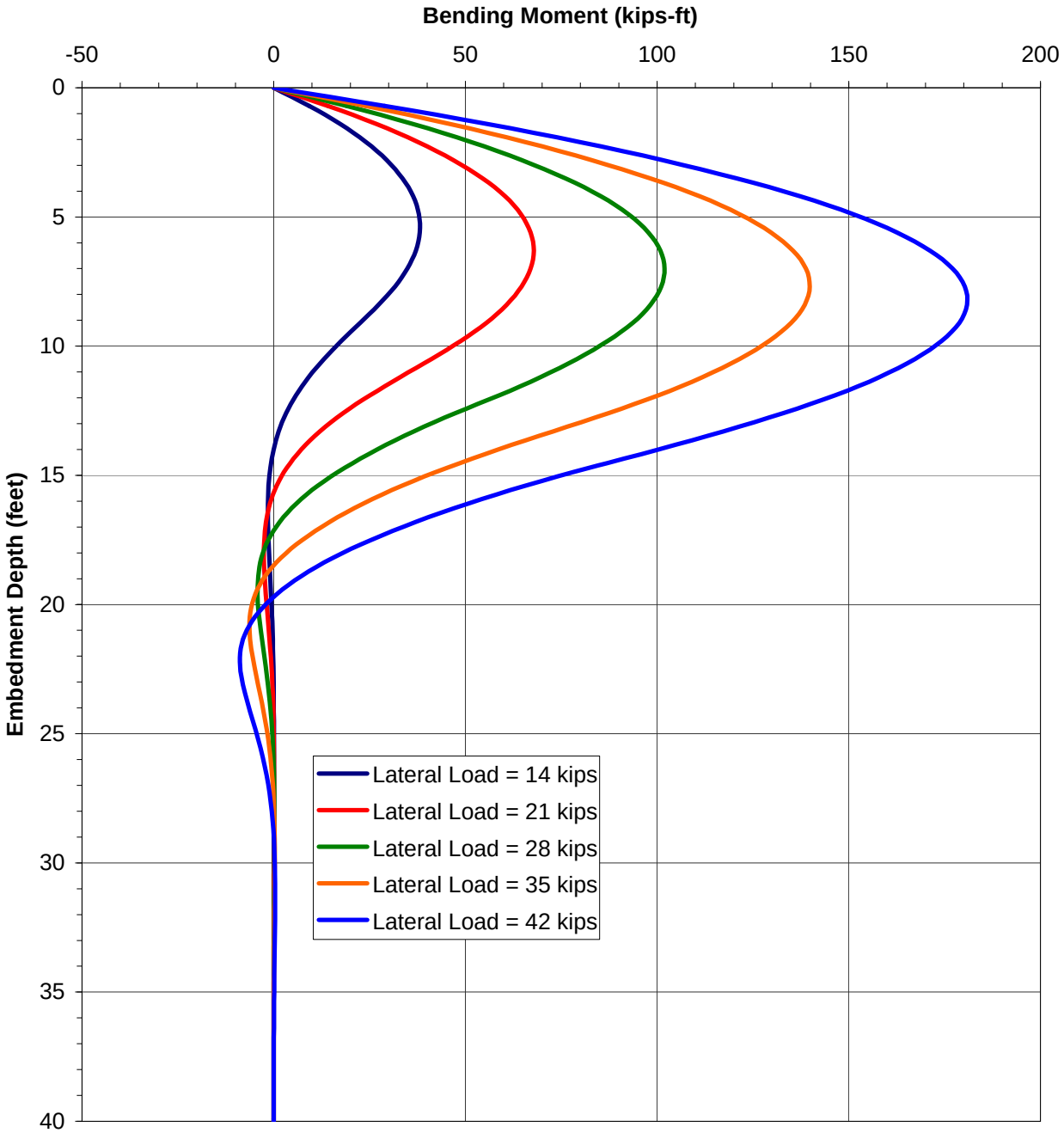
LATERAL DEFLECTION VS. DEPTH
30-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Free Head Condition



DEFLECTION RESPONSE: 30-INCH DIAMETER PIER (FREE HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 12a

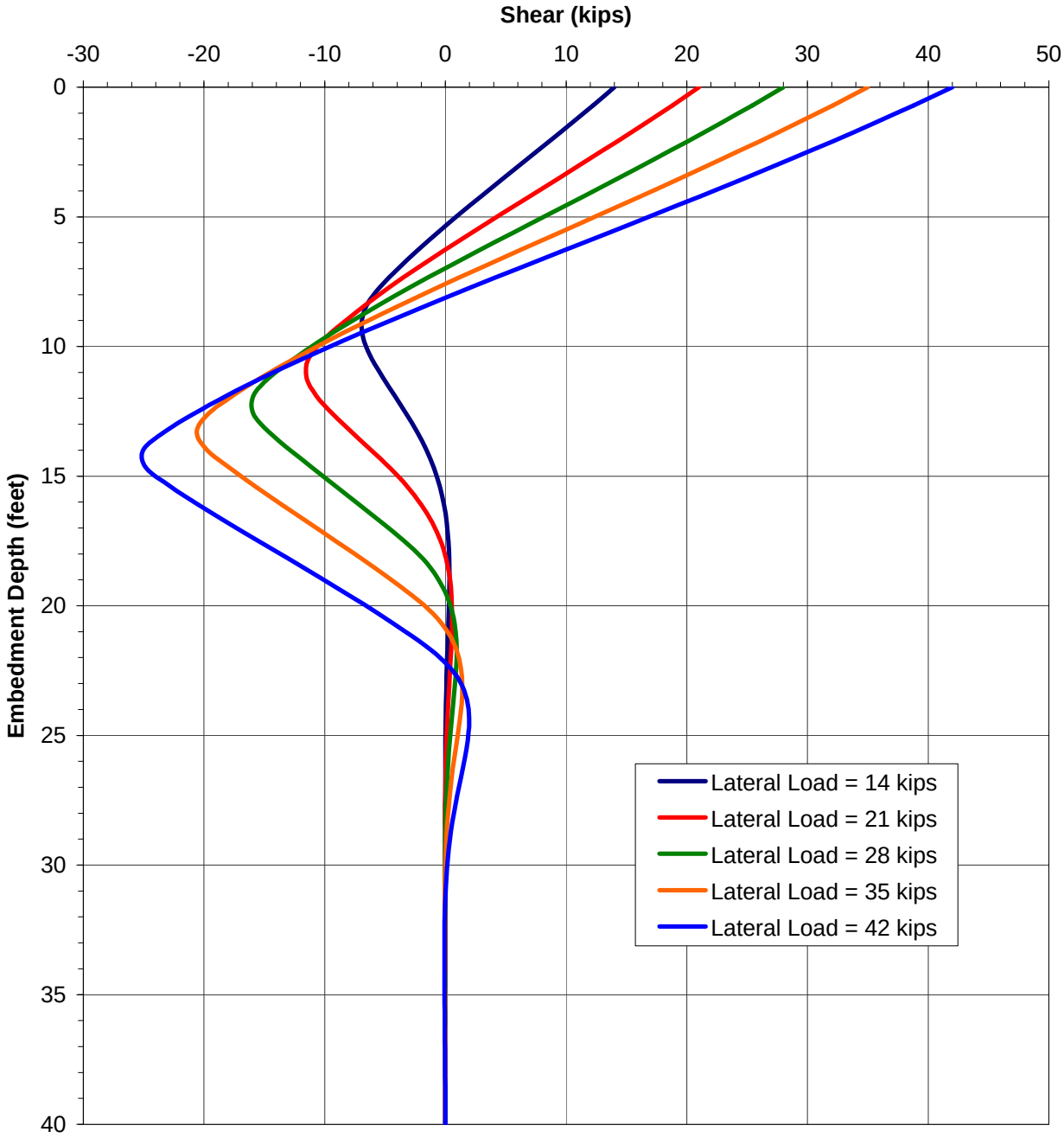
BENDING MOMENT WITH DEPTH BELOW PIER CAP
30-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Free Head Condition



MOMENT RESPONSE: 30-INCH DIAMETER PIER (FREE HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 12b

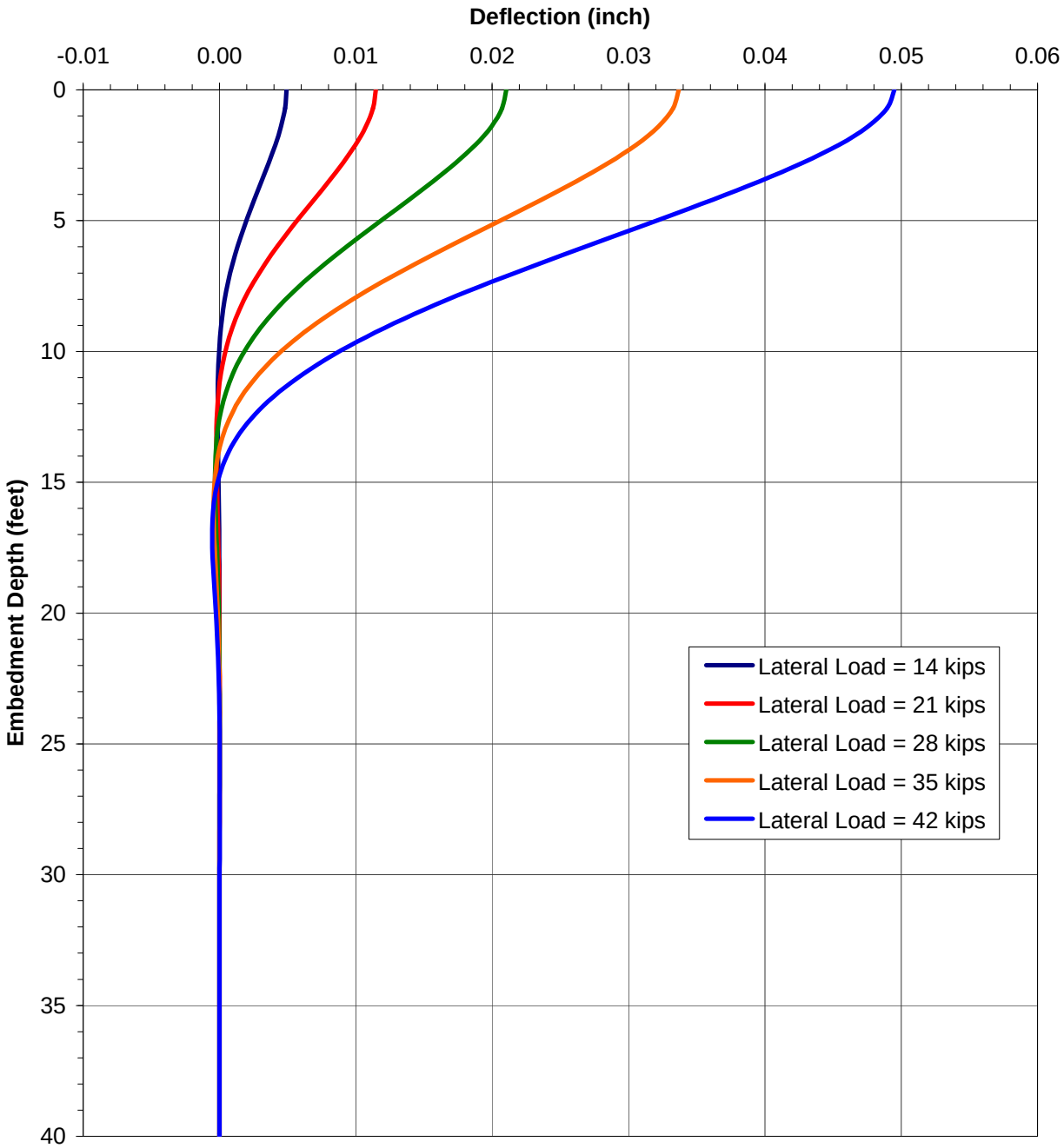
SHEAR WITH DEPTH BELOW PIER CAP
30-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Free Head Condition



SHEAR RESPONSE: 30-INCH DIAMETER PIER (FREE HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 12c

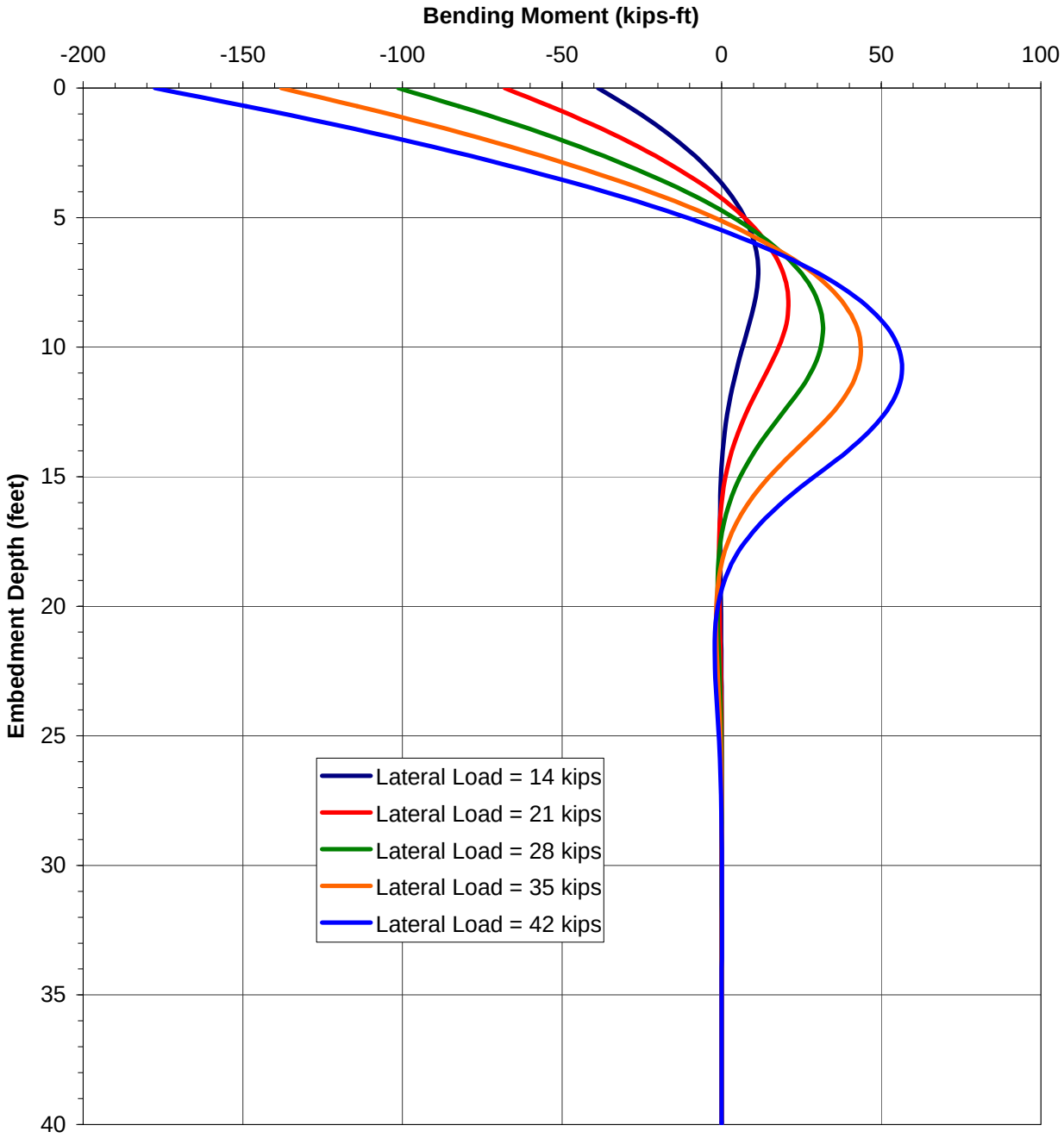
LATERAL DEFLECTION VS. DEPTH
30-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Fixed Head Condition



DEFLECTION RESPONSE: 30-INCH DIAMETER PIER (FIXED HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 13a

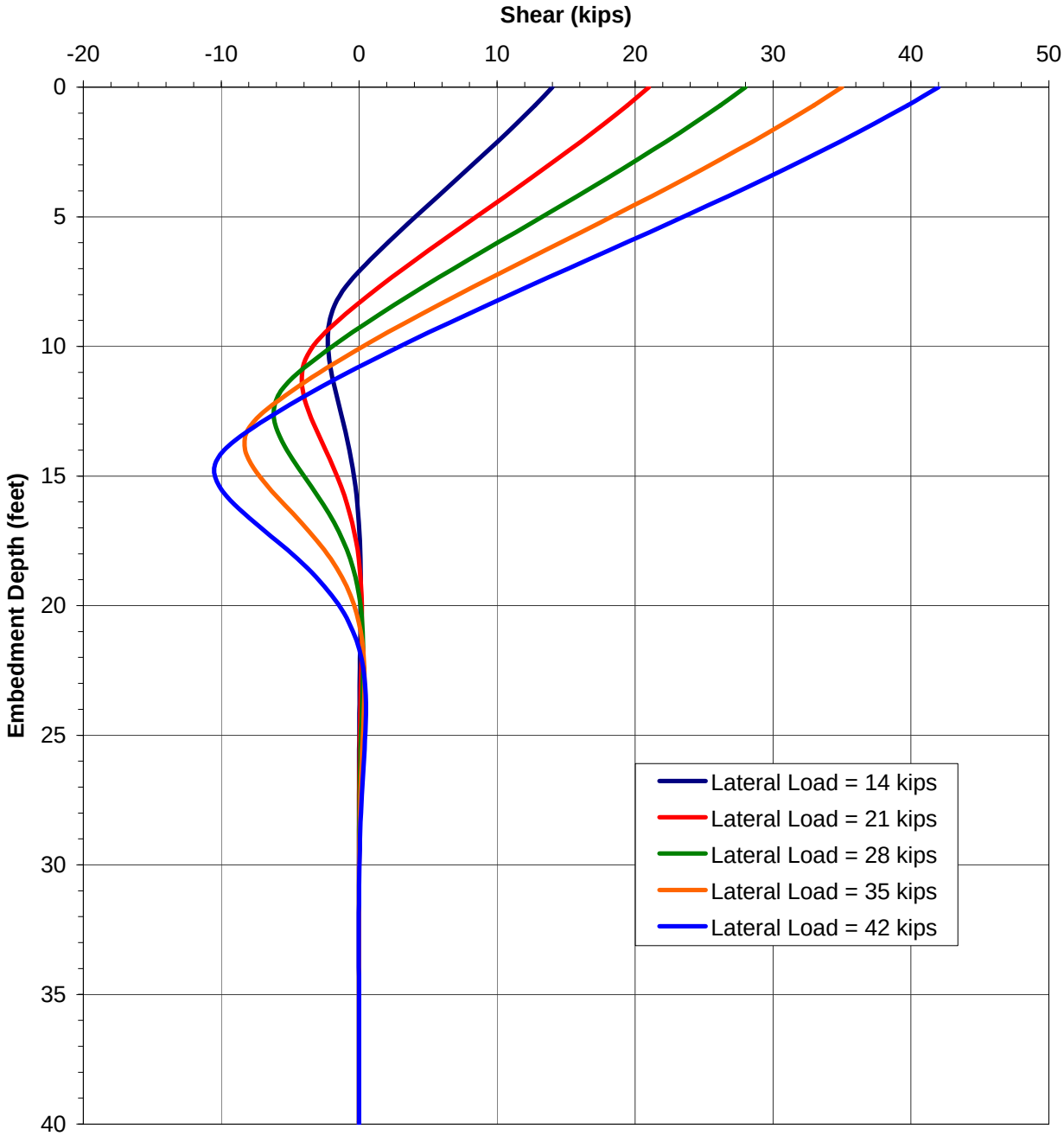
BENDING MOMENT WITH DEPTH BELOW PIER CAP
30-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Fixed Head Condition



MOMENT RESPONSE: 30-INCH DIAMETER PIER (FIXED HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

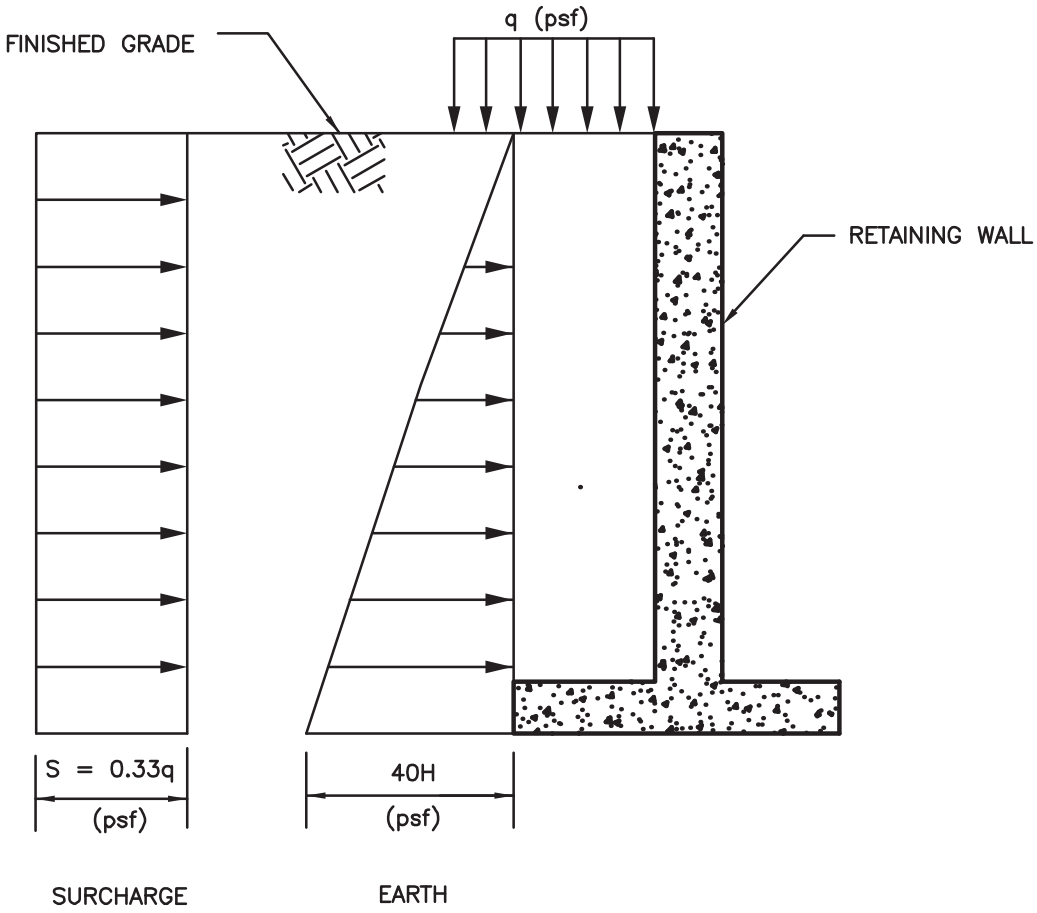
PLATE 13b

SHEAR WITH DEPTH BELOW PIER CAP
30-inch Diameter Drilled Pier
Uncracked Section, f'c=4000 psi
Fixed Head Condition



SHEAR RESPONSE: 30-INCH DIAMETER PIER (FIXED HEAD)
CHRCO Outpatient Center No. 2
Oakland, California

PLATE 13c



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.33q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

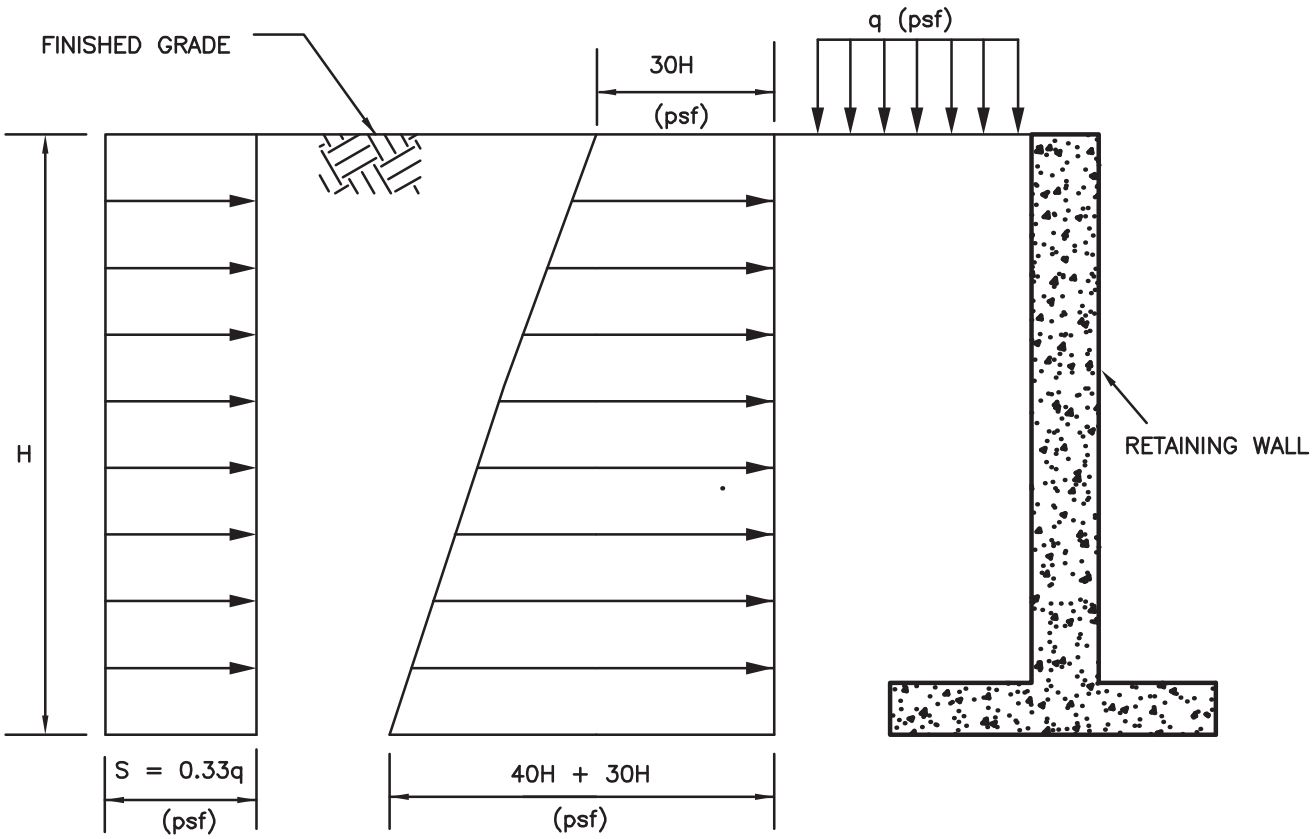
NOTES:

1) UNRESTRAINED WALLS ARE FREE TO MOVE/ROTATE.
2) WALLS ASSUMED TO BE FULLY BACK DRAINED
3) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL
4) ASSUMES LEVEL BACKFILL.

STATIC LATERAL PRESSURES ON UNRESTRAINED WALLS

CHRCO Outpatient Center No. 2
Children's Hospital and Research Center
Oakland, California

PLATE 14a



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.33q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

1) UNRESTRAINED WALLS ARE FREE TO MOVE/ROTATE.
2) WALLS ASSUMED TO BE FULLY BACK DRAINED
3) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL
4) ASSUMES LEVEL BACKFILL.

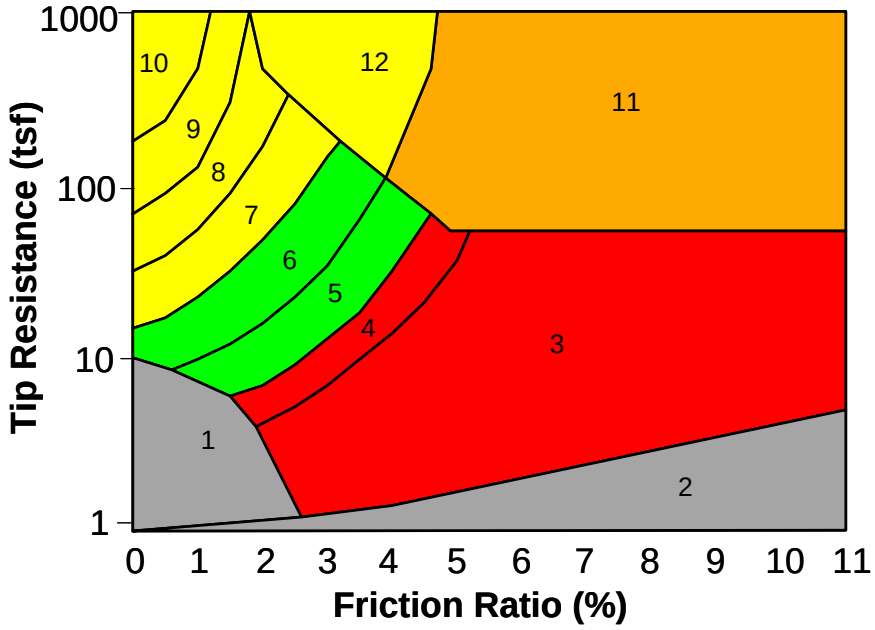
SEISMIC LATERAL PRESSURES ON UNRESTRAINED WALLS

CHRCO Outpatient Center No.2
Children's Hospital and Research Center
Oakland, California

PLATE 14b

APPENDIX A
FIELD EXPLORATIONS

COLOR LEGEND FOR FRICTION RATIO TRACES



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

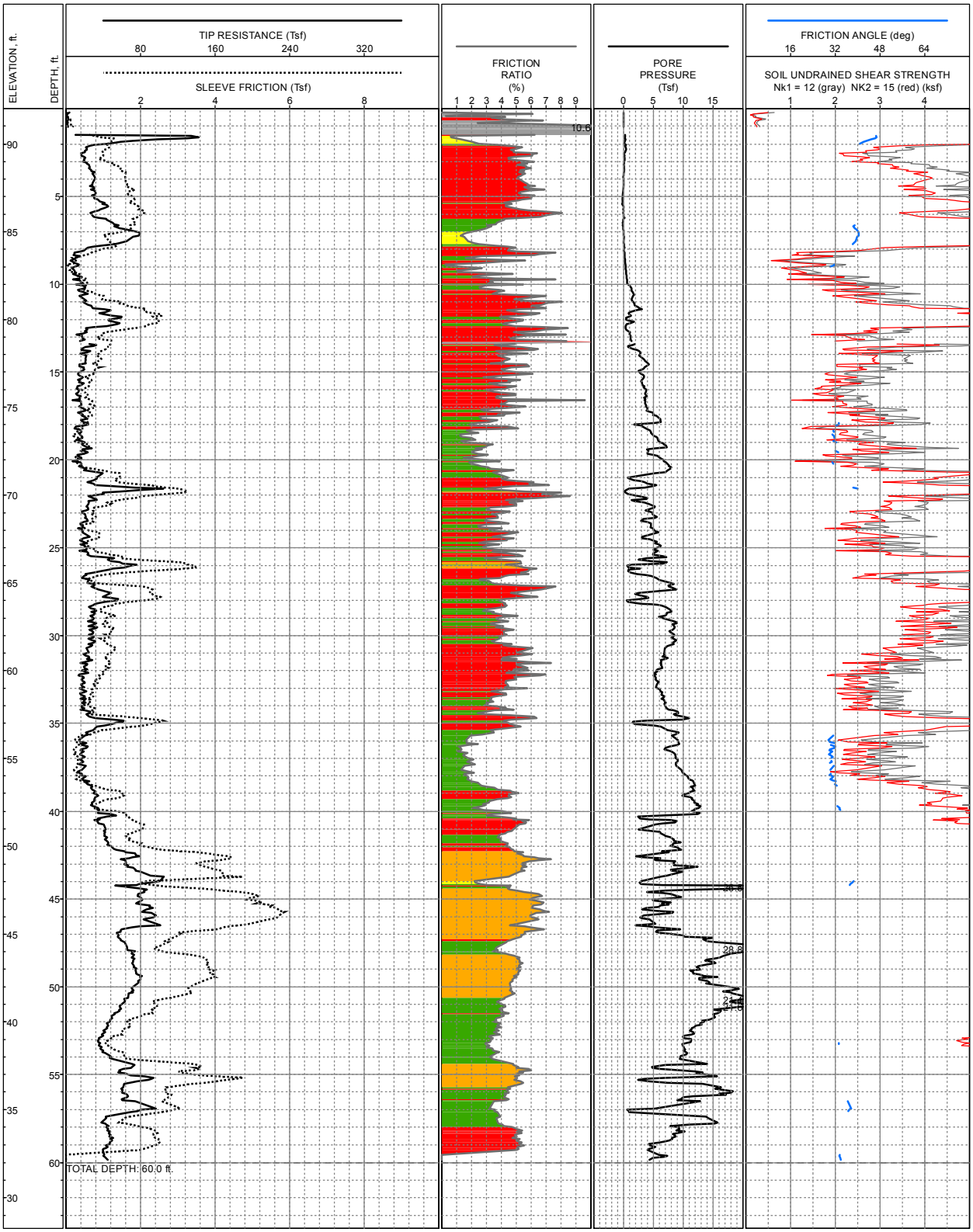
*overconsolidated or cemented

CPT CORRELATION CHART
(Robertson and Campanella, 1988)

KEY TO CPT LOGS

CHRCO Outpatient Center No. 2
Oakland, California

N:\Projects\04_2012\04_7212_0009_CHRCO_Outpatient\Explorations\CPT\2012\Logs\2012_03_01_Logs_SuFr\MXD\KeytoCPT-RC88.mxd, 03/21/12, evonthury



LOCATION: E6,051,016, N 2,132,229, CA State Plane, Zone 3, NAD83, Feet
SURFACE EL: 92.0ft +/- (City of Oakland Datum)
COMPLETION DEPTH: 60.0ft
TESTDATE: 2/16/2012

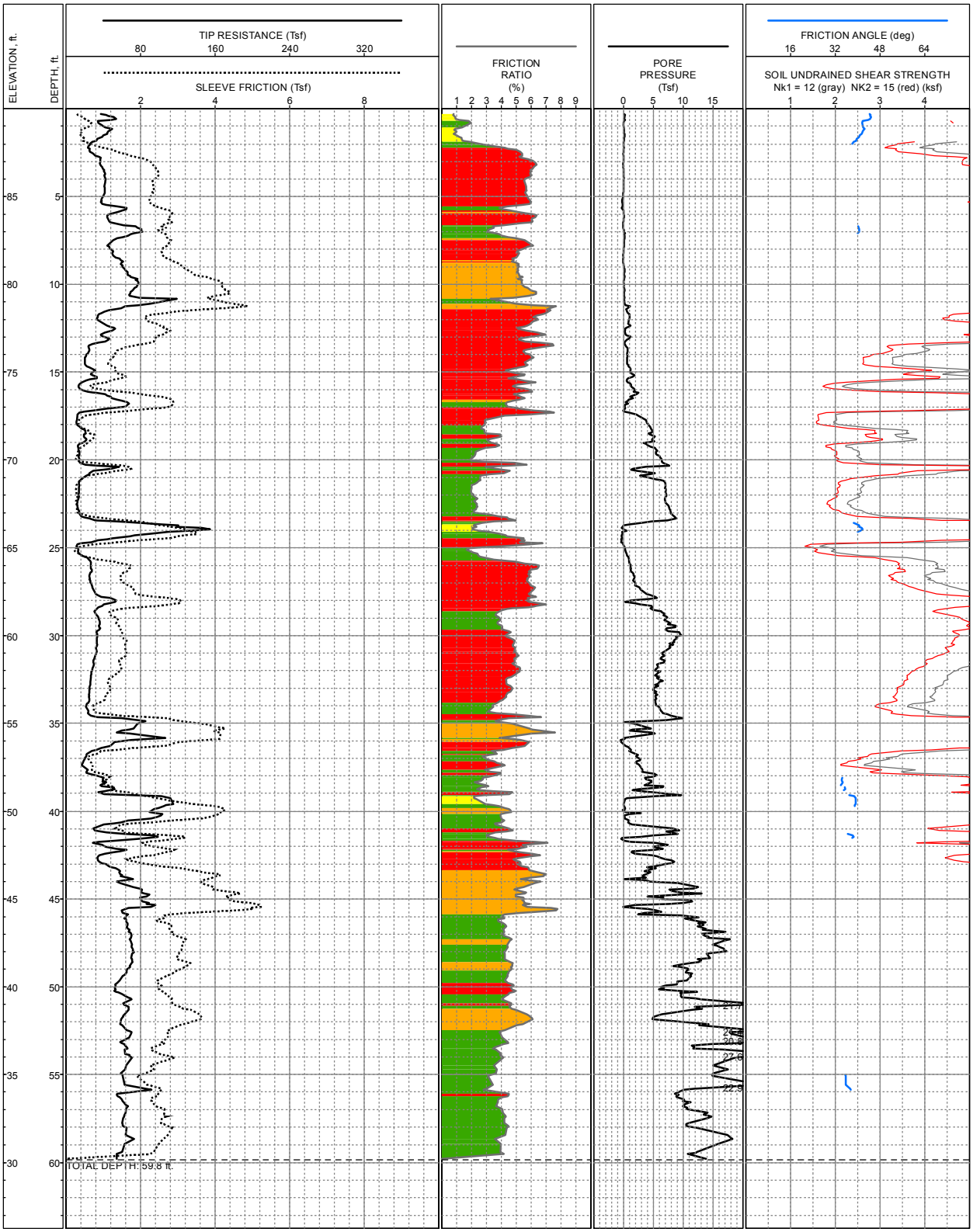
EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: J Kocijan
CONE AREA RATIO: 0.59

LOG OF CPT NO: CPT-1

CHRCO Outpatient Center No. 2
Oakland, California

PLATE A-2

N:\Projects\04_2012\04_7212_0009_CHRCO_Outpatient\Explorations\CPT\2012\Logs\2012_03_01_Logs_SuFr\MXD\Logs_SuFr_VG12C.mxd,07/23/2013,CDean



LOCATION: E6,051,175, N 2,132,244, CA State Plane, Zone 3, NAD83, Feet
SURFACE EL: 90.0ft +/- (City of Oakland Datum)
COMPLETION DEPTH: 59.8ft
TESTDATE: 2/16/2012

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: J Kocijan
CONE AREA RATIO: 0.59

LOG OF CPT NO: CPT-2

CHRCO Outpatient Center No. 2
Oakland, California

PLATE A-3

N:\Projects\04_2012\04_7212_0009_CHRCO_Outpatient\Explorations\CPT\2012\Logs\2012_03_01_Logs_SuFr\MXD\Logs_SuFr_VG12C.mxd,07/23/2013,CDean

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	3/4"	3"	12"	
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

SANDS AND GRAVELS	BLOWS/FOOT†
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH‡	BLOWS/FOOT†
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

†Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

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KEY TO EXPLORATORY BORING LOGS
Unified Soil Classification System (ASTM D-2487)

AMBULATORY SERVICES CENTER
CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	Figure
K938-5-1124	April 1992	A-1

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	92.5 FT.	LOGGED BY	MC
DEPTH TO GROUNDWATER	23.0 Feet	BORING DIAMETER	7-inch	DATE DRILLED	1/2/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
SILT (ML), brown, with clay, some sand (fine- to medium-grained), trace organics	Firm				10*				PP = 0.8 tsf
(clayey, trace sand (fine- to coarse-grained), trace gravel (fine- grained), less organics)	Stiff				14				
CLAY (CL), brown mottled rust, silty, some sand (fine- to coarse- grained), trace gravel (fine- to coarse-grained), trace organics	Stiff		5		28*				PP = 2.0 tsf
SILT (ML), light brown mottled gray, with clay, some sand (fine- to coarse- grained), trace gravel (fine- grained)	Stiff		10		18*	18	109	1.4	PP = 1.8 tsf, Passing #200 Sieve=60%
GRAVEL (GM), light brown and rust, fine-grained, sandy (fine- to coarse-grained), trace silt and clay	Medium dense		15						
SAND (SM), light brown mottled gray and rust, fine- to medium-grained, silty, trace gravel (fine-grained), trace sand (coarse-grained)	Medium dense		20		22*	25	101		PP = 1.5 tsf
(interbedded SILT (ML), brown, with clay, some sand (fine-grained))			25			▽			
CLAY (CL), tan mottled black and rust, some silt, trace sand (fine- grained)	Very stiff		30		19*	23			PP = 4.5 tsf
GRAVEL (GM), light brown mottled rust and gray, speckled black, fine- grained, with sand (fine- to coarse-grained), trace silt and clay	Dense		35						

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-5-1124	April 1992	PB92-1

Plate A-5

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	92.5 FT.	LOGGED BY	MC
DEPTH TO GROUNDWATER	23.0 Feet	BORING DIAMETER	7-inch	DATE DRILLED	1/2/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
GRAVEL (GM), light brown mottled rust and gray, speckled black, fine- grained, with sand (fine- to coarse-grained), trace silt and clay	Dense		40		58*				
			45						
CLAY (CL), rust-brown mottled tan and black, with silt, with sand (fine- to coarse-grained), trace gravel (fine-grained)	Hard		50		34				PP > 4.5 tsf
SAND (SM), mottled light brown, rust, red, gray and black, fine- to coarse-grained, gravelly (fine-grained), trace silt and clay	Very Dense		55						
			60		95*/10"				
Bottom of Boring = 60-1/2 Feet									

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was measured at 23 feet at the time of drilling.
4. The boring was backfilled with cuttings immediately after completion.
5. PP = Pocket Penetrometer, LL = Liquid Limit, PI = Plasticity Index

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-5-1124	April 1992	PB92-1

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	92.5 FT.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	7-inch	DATE DRILLED	1/2/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), dark brown, silty, trace sand (fine- to coarse-grained), trace organics	Stiff				20*	25	94		PP = 2.0 tsf, LL=44, PI=25, Passing #200 Sieve=91%
(same as above)	Very stiff		5		16				
(light brown mottled rust and tan, some sand (fine- to coarse-grained), trace gravel (fine-grained), no organics)	Stiff		10		20*	18	110	2.2	PP = 2.5 tsf, LL=24, PI=8, Passing #200 Sieve=72%
(less gravel, no organics)			15		22*	27	91	2.5	PP = 1.8 tsf
(tan mottled rust and black, with silt, trace sand (fine-grained))			20		24*	26	98	6.0	PP = 2.8 tsf
(interbedded SILT (ML), brown, with clay, some sand (fine-grained))			25						
(same as at 20 feet)					20*				PP = 1.0 tsf in silt interbed PP = 3.2 tsf
			30						
(mottled brown, black, rust and tan)	Firm		35		13*	24	104		PP = 1.2 tsf

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-5-1124	April 1992	PB92-2

Plate A-7

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	92.5 FT.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	7-inch	DATE DRILLED	1/2/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), mottled brown, black, rust and tan, with silt, some sand (fine-grained)	Firm		40						
GRAVEL (GM), light brown mottled rust, gray and tan, fine- to coarse- grained, sandy (fine- to coarse-grained), some silt and clay	Dense		45		87*				
			50						
			55						
(same as above)	Very dense				52				
Bottom of Boring = 59-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not encountered at the time of drilling.
4. The boring was backfilled with cuttings immediately after completion.
5. PP = Pocket Penetrometer, LL = Liquid Limit, PI = Plasticity Index

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-5-1124	April 1992	PB92-2

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION		LOGGED BY	MD/PW
DEPTH TO GROUNDWATER	20.0 Feet	BORING DIAMETER	6-inch	DATE DRILLED	6/7/90

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
Asphalt Concrete (3") **Baserock (6")**									
SAND (SM), fine- to coarse-grained, black, some silt and clay, trace gravel (fine-grained) **(FILL)**	medium dense		5		23*				
CLAY (CL), mottled black, grey and orange, some silt and sand (fine- to coarse-grained	hard				36*	19	104	8.8	
SAND (SM), fine- to coarse-grained, mottled black, orange, red, some silt and clay	medium dense		10		10				
CLAY (CL), tan-grey, some silt and trace sand (fine-grained)	stiff		15						
(grading with silt) (mottled grey and orange)	very stiff				24*	25	99		Su=2.5ksf
			20						
			25						
	firm				15*	20			Poor recovery
			30						
			35						
(grading some sand, fine- to coarse-grained, and trace gravel, fine- to medium-grained)	hard				50*/6	16			
			40						
			45						
(grading sandy, fine- to medium-grained)					65				

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-5-1124	April 1992	PB90-3

Plate A-9

DRILL RIG	**Continuous Flight Auger**	SURFACE ELEVATION		LOGGED BY	**MD/PW**
DEPTH TO GROUNDWATER	**20.0 Feet**	BORING DIAMETER	**6-inch**	DATE DRILLED	**6/7/90**

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), mottled black, grey and orange, sandy (fine- to medium-grained), trace gravel (fine- to medium-grained)	hard								
SAND (SM), fine- to coarse-grained, mottled black, orange, and grey, with silt and clay, trace gravel (fine-grained)	dense		55	☒	50/6*	18	115		-#200=36%
GRAVEL (GM), fine- to coarse-grained, trace silt and clay	dense		60						
			65						
Bottom of Boring = 67 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*), see first page Appendix A. 3. Ground water level was encountered at 20 feet at time of drilling.									

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-5-1124	April 1992	PB90-3

DRILL RIG Hollow Stem Auger	SURFACE ELEVATION --	LOGGED BY JN/KS
DEPTH TO GROUNDWATER 12' (see note 3)	BORING DIAMETER 8 Inches	DATE DRILLED 6/4/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
1" AC over 5" Baserock				1					
CLAY, silty, trace of organics	black	very stiff	CL			18	21		
						35*	20	103	7.1
(grading trace of sand coarse grained and gravel)	brown	stiff		5		28*	22		
(grading with silt, some sand) (interbedded lense of sandy coarse grained gravel)	light brown mottled with grey			10		23*	20	106	4.9
(minor interbedded sandy clay lense)	orange mottled with grey					18*			
Liquid Limit = 36% Plasticity Index = 16% Passing #200 Sieve = 92%				15		300 psi	33 28	90 93	3.4
(grading very silty, sandy)	grey-tan			20					
SILT, some sand (very fine - fine grained), clay (gravelly lense)	orange mottled with grey	stiff	ML-CL			13	31		
				25					
CLAY, silty, trace of sand (fine-coarse grained)	orange mottled with grey	hard	CL			35	21		
				30					
		stiff				25*	21	104	4.6
				35					
SAND (fine-coarse grained), some silt and clay	light brown-grey	medium dense	SM-SC			25	20		
				40					

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-5-1124	April 1992	PB86-1

DRILL RIG Hollow Stem Auger	SURFACE ELEVATION --	LOGGED BY JN/KS
DEPTH TO GROUNDWATER 12' (see note 3)	BORING DIAMETER 8 Inches	DATE DRILLED 6/4/86

DESCRIPTION AND CLASSIFICATION				DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE						
SAND (fine-coarse grained), some clay and silt (continued)	light brown-grey	medium dense	SM-SC	41					
CLAY, very silty, some sand (fine-coarse grained)	light brown mottled with grey & black	hard	CL-ML	45		38	26		
SAND (fine-coarse grained), some clay, trace of gravel (fine-coarse grained)	light brown-orange	very dense	SC	50		54	19		
CLAY, some silty, sand (fine-coarse grained)	brown	hard	CL	55					
				60		35	23		
Bottom of Boring = 60½ Feet				65					
Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3. Groundwater level was measured at 14 feet at time of drilling. Four hours after drilling, the groundwater level was measured at 12 feet.				70					
				75					
				80					

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EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-5-1124	April 1992	PB86-1

Plate A-12

DRILL RIG Continuous Flight Auger	SURFACE ELEVATION --	LOGGED BY JN/KS
DEPTH TO GROUNDWATER 24' (see note 3)	BORING DIAMETER 6 Inches	DATE DRILLED 6/4/86

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
2" AC over 5" Baserock									
SILT, sandy, some gravel (FILL)	brown	firm	ML-SM	1		13*	11		
						22*	23		
CLAY, silty, trace of gravel and sand (coarse grained) Liquid Limit = 40% Plasticity Index = 20% Passing #200 Sieve = 75%	black	stiff	CL	5		26*	19	106	10.1
CLAY, silty, trace of sand (fine-coarse grained) (sand lense, some silt, trace of clay medium dense)	brown	stiff	CL	10		19*	17	106	
(grading very silty) (sand grading fine)	orange mottled with grey			15		12	26		
		firm		20		14*	26		
(grading some silt) (grading sandy fine-medium grained)		stiff	CL-SC	25		11	24		
(grading very silty without sand)		hard	CL	30		74*	18	113	15.8
(grading less silty with trace of sand fine grained)	mottled with black	stiff		35		15	27		
(grading sandy fine-coarse grained, gravelly)		very stiff							
Bottom of Boring = 40½ Feet				40		28	24		

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A.
3. Groundwater level was measured at 24 feet at time of drilling.

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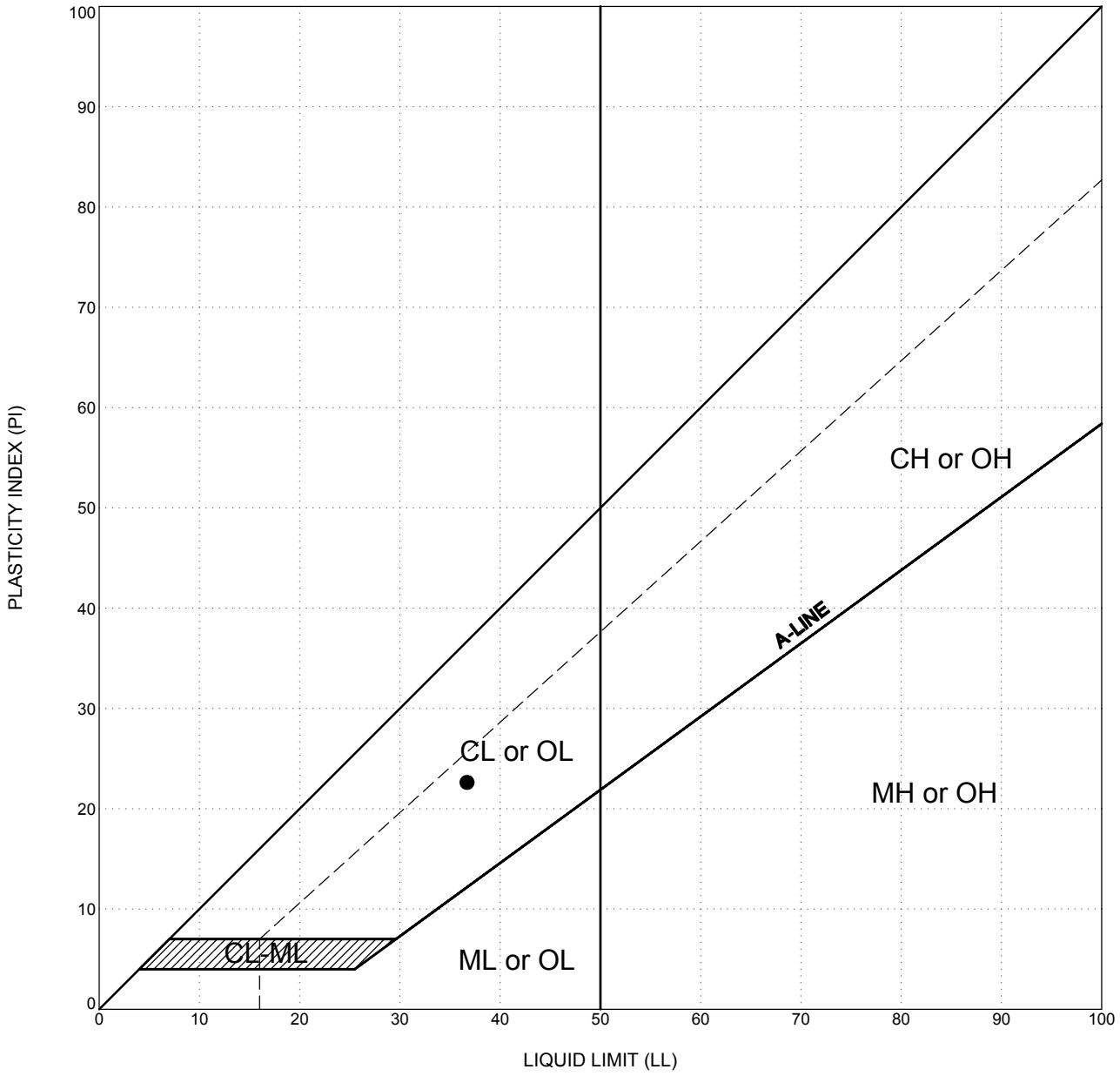
EXPLORATORY BORING LOG

AMBULATORY SERVICES CENTER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-5-1124	April 1992	PB86-3

APPENDIX B
LABORATORY TESTING PROGRAM

Children's Hospital and Research Center Oakland
Project No. 04.72120009

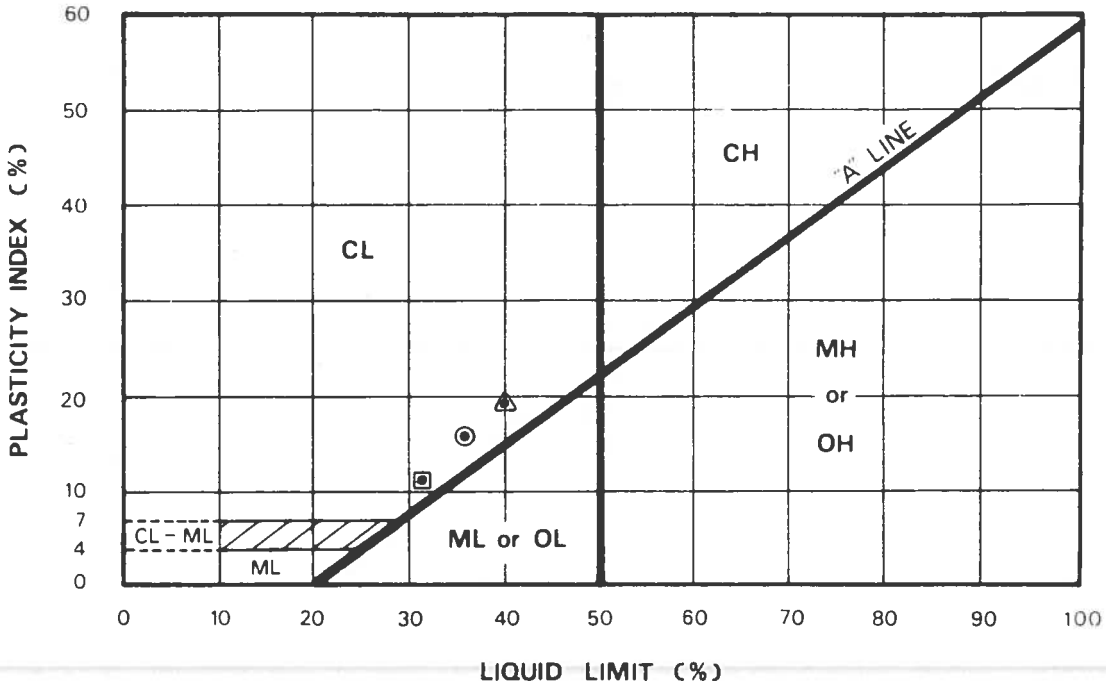


ATTERBERG LIMITS TEST RESULTS

LEGEND location	depth, ft	CLASSIFICATION	LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
● HA-2	0.3	Dark Brown Lean CLAY with SAND (CL)	37	14	23

PLASTICITY CHART
CHCRO Outpatient Center No. 2
Oakland, California

PLATE B-1



KEY SYMBOL	Previous Boring No.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	ATTERBERG LIMITS			PASSING NO. 200 SIEVE %	UNIFIED SOIL CLASSIFICATION SYMBOL
				LIQUID LIMIT %	PLASTICITY INDEX %	LIQUIDITY INDEX %		
◉	1	17.5	29	36	16	0.5	92	CL
◬	3	5	19	40	20	0.1	75	CL
▣	5	10	20	31	11	0.1	85	CL

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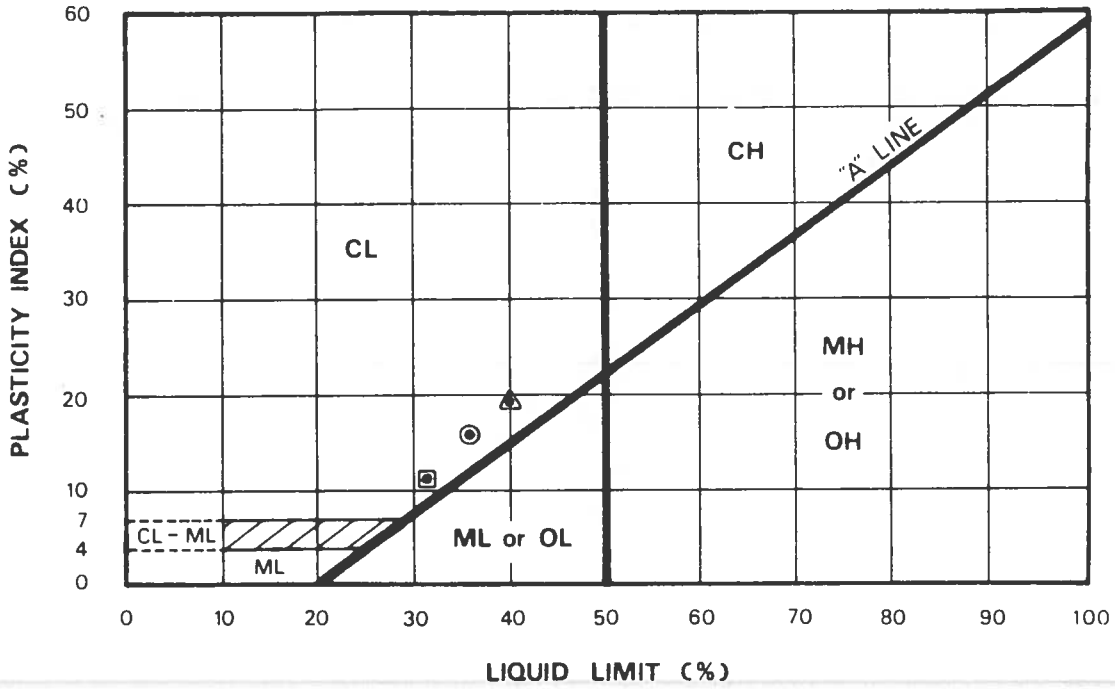
PLASTICITY CHART AND DATA

PROPOSED MEDICAL OFFICE BUILDING
AND PARKING STRUCTURE
Oakland, California

PROJECT NO	DATE	Figure
K339-9	November 1990	B-1

Plate B-2



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	ATTERBERG LIMITS			PASSING NO. 200 SIEVE %	UNIFIED SOIL CLASSIFICATION SYMBOL
				LIQUID LIMIT %	PLASTICITY INDEX %	LIQUIDITY INDEX %		
◉	1	17.5	29	36	16	0.5	92	CL
◬	3	5	19	40	20	0.1	75	CL
▣	5	10	20	31	11	0.1	85	CL

Peter Kaldveer and Associates

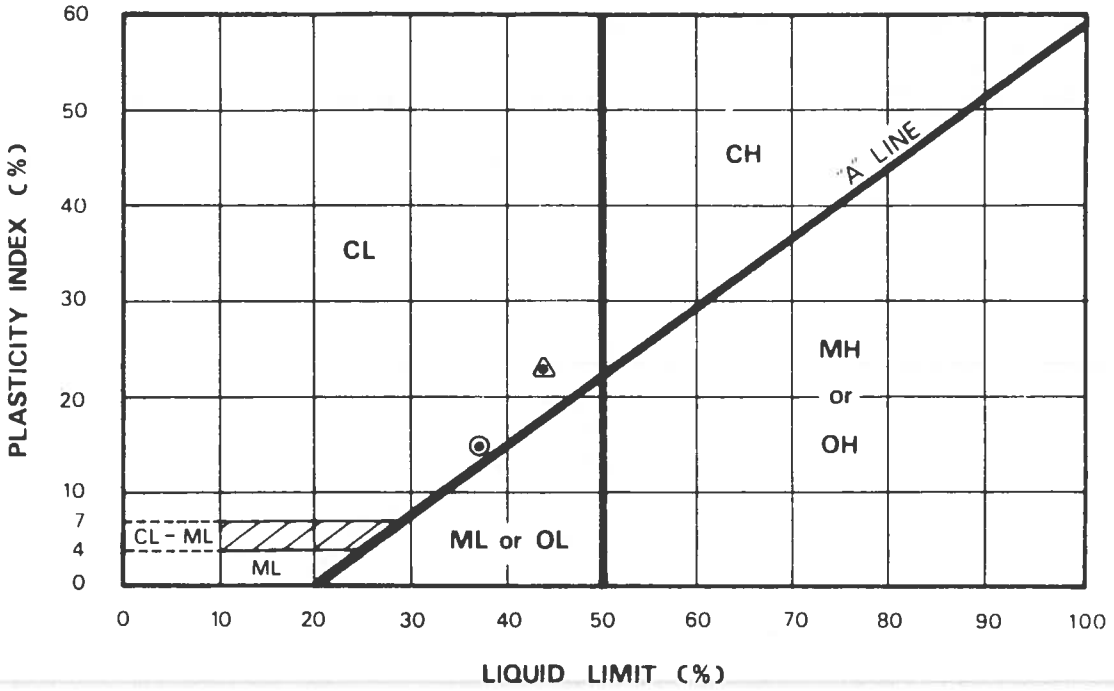
Geotechnical Consultants

PLASTICITY CHART AND DATA

PROPOSED MEDICAL OFFICE BUILDING
AND PARKING STRUCTURE
Oakland, California

PROJECT NO	DATE	Figure
K928-1	June 1986	B-1

Plate 3



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	ATTERBERG LIMITS			PASSING NO. 200 SIEVE %	UNIFIED SOIL CLASSIFICATION SYMBOL
				LIQUID LIMIT %	PLASTICITY INDEX %	LIQUIDITY INDEX %		
⊙	8	2	18	37	15	-0.3	57	CL
◬	S-1	Exp	--	44	23	--	83	CL

Peter Kaldveer and Associates

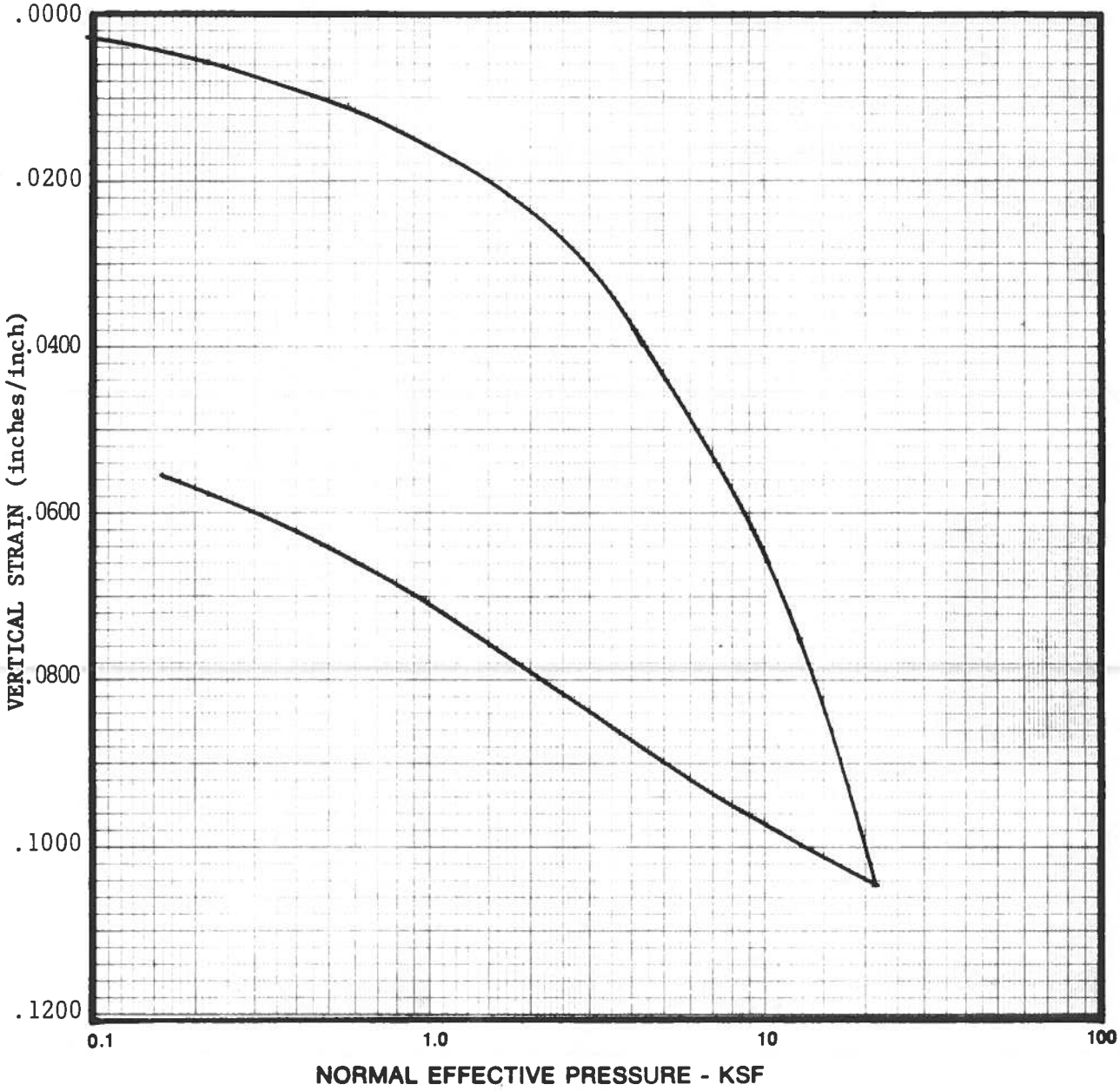
Geotechnical Consultants

PLASTICITY CHART AND DATA

PROPOSED MEDICAL OFFICE BUILDING
AND PARKING STRUCTURE
Oakland, California

PROJECT NO	DATE	
K928-1	June 1986	Figure B-2

Plate 4



SAMPLE DATA

BORING NO.: 1	
DEPTH (ft.): 15-17.5	ELEVATION (ft.): --
DESCRIPTION: Gray and Tan Silty CLAY	
SAMPLE DIAMETER (In.): 2.82	SPECIFIC GRAVITY (Gs): 2.7
LIQUID LIMIT (%): 35.8	PLASTICITY INDEX (%): 16.0

CONSOLIDATION TEST DATA

	INITIAL	FINAL
DRY DENSITY (PCF)	93.3	98.2
MOISTURE CONTENT (%)	28.5	26.5
VOID RATIO	.813	.716
DEGREE OF SATURATION (%)	94.7	100.0
HEIGHT (In.)	.7500	.7100

Peter Kaldveer and Associates

Geotechnical Consultants

CONSOLIDATION TEST DATA

PROPOSED MEDICAL OFFICE BUILDING
AND PARKING STRUCTURE
Oakland, California

PROJECT NO.	DATE	
K928-1	June 1986	Figure B-5

Plate B-5

APPENDIX C
SEISMIC HAZARD ANALYSIS

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

6 March 2012

Job No.1202203
Cust. No.11608

Mr. Coleman Ng
Fugro Consultants
1000 Broadway, Suite 440
Oakland, CA 94607

Subject: Project No.: 04.72120009
Project Name: Oakland, CA, Childrens Hospital
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. Ng:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on February 29, 20112. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a detection limit of 15 mg/kg.

The sulfate ion concentrations ranged from none detected to 32 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 7.4 to 8.2, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 440 to 450-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical

1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

Client: Fugro West, Inc.
Client's Project No.: 04.72120009
Client's Project Name: Oakland, CA, Childrens Hospital
Date Sampled: 28-Feb-12
Date Received: 29-Feb-12
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 6-Mar-2012

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1202203-001	B-1 #1B @ 1.25-2.0	450	7.4	-	5,900	-	N.D.	32
1202203-002	B-2 #2B @ 1.0-1.75	440	8.2	-	4,500	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	2-Mar-2012	2-Mar-2012	-	2-Mar-2012	-	5-Mar-2012	5-Mar-2012

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Chain of Custody

Page 1 of 1

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax 925 462 2775
CERCO analytical

Job No.	CU#		Client Project I.D.
1202203	10169		04.72120009

Schedule
Analyte

Date Sampled | Date Due

Full Name 11608
Coleman Ng.

~~Fax~~ Phone (510) 267 4431

Company email ~~cell~~ cng@fugro.com ☒
Fugro Consultants, Inc.

Sample Source
Oakland CA. Childrens Hospital

ASTM w/Brief Evaluation | **ANALYSIS**

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation							
001	B-1 #1B @ 1.25 2.0	2/28/12	11:05	S	Baggie			1	X	X	X	X	X		X							
002	B-2 #2B @ 1.0 1.75	2/29/12	11:05	S	Baggie			1	X	X	X	X	X		X							
												GSO										

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

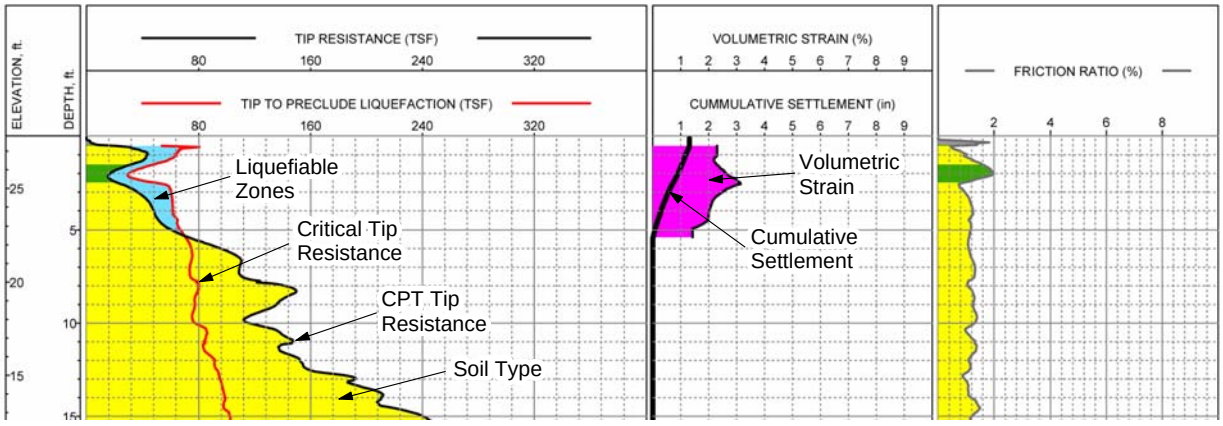
SAMPLE RECEIPT
Total No. of Containers: 2
Rec'd Good Cond/Cold:
Conforms to Record:
Temp. a t Lab -°C:
Sampler:

Relinquished By: Andrew Bajuniemi Date 2/28/12 Time 3:00 pm
Received By: Kim M Dueber Date 2/29/12 Time 10:30
Relinquished By: Date Time
Received By: Date Time
Relinquished By: Date Time
Received By: Date Time

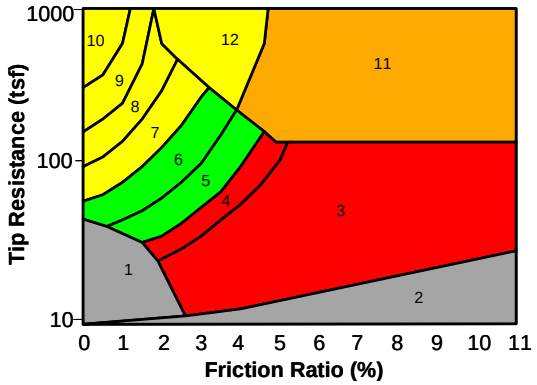
Comments:
THERE IS AN ADDITIONAL CHARGE FOR METAL/POLY TUBES

8/6/2009

APPENDIX D
LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES



COLOR LEGEND FOR FRICTION RATIO TRACES



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

*overconsolidated or cemented

CPT CORRELATION CHART

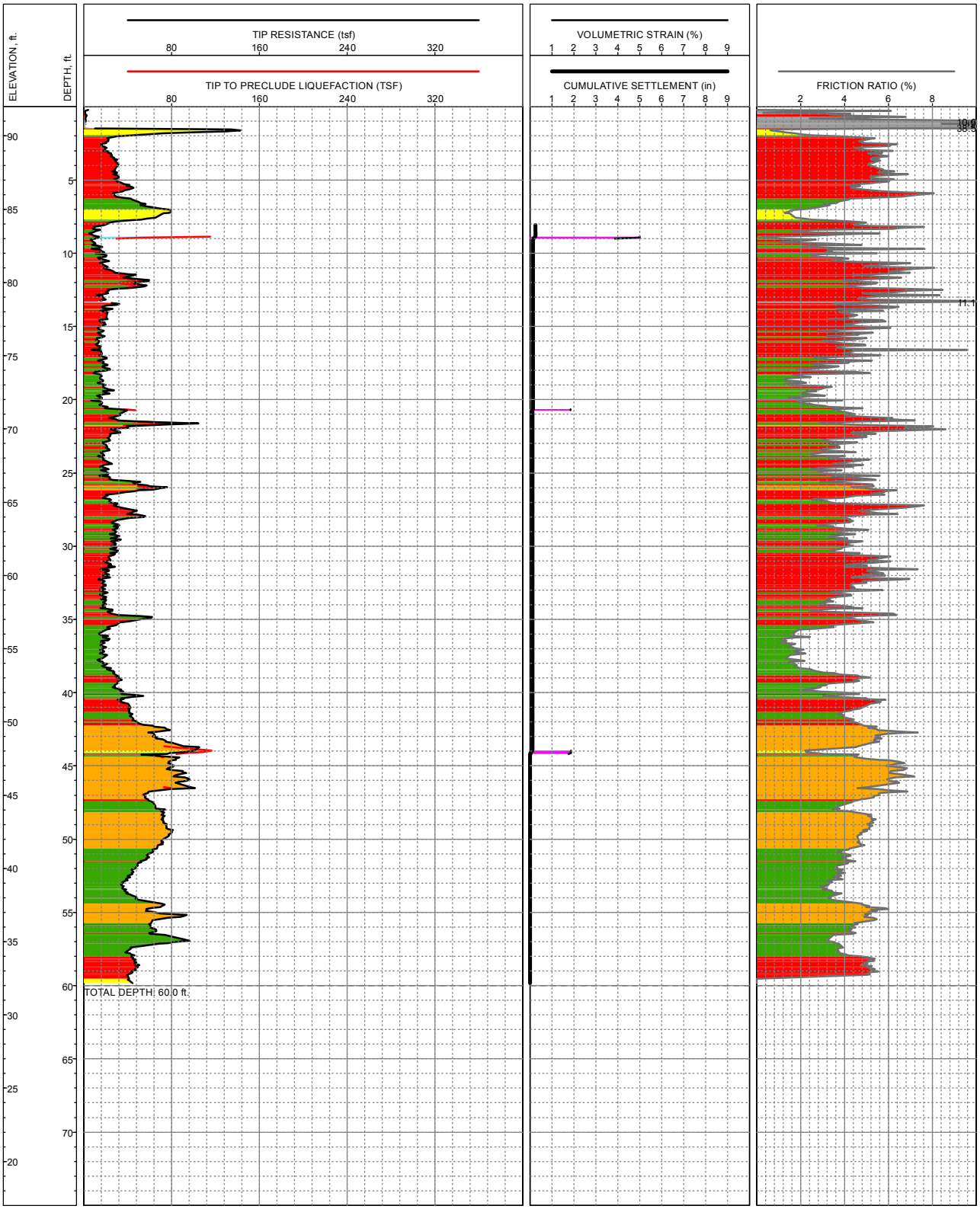
(Robertson and Campanella, 1988)

KEY TO LIQUEFACTION LOGS
NCEER, M=7.3, PGA= 0.73g

CHRCO Outpatient Center No. 2
Oakland, California

PLATE D-1

N:\Projects\04_2012\04_7212_0009_CHRCO_Outpatient\Explorations\CPT\2012\Logs\2012_03_01_Logs_SuFr\MXD\KeytoCPT-Liquefaction.mxd, 03/21/12, evonthury



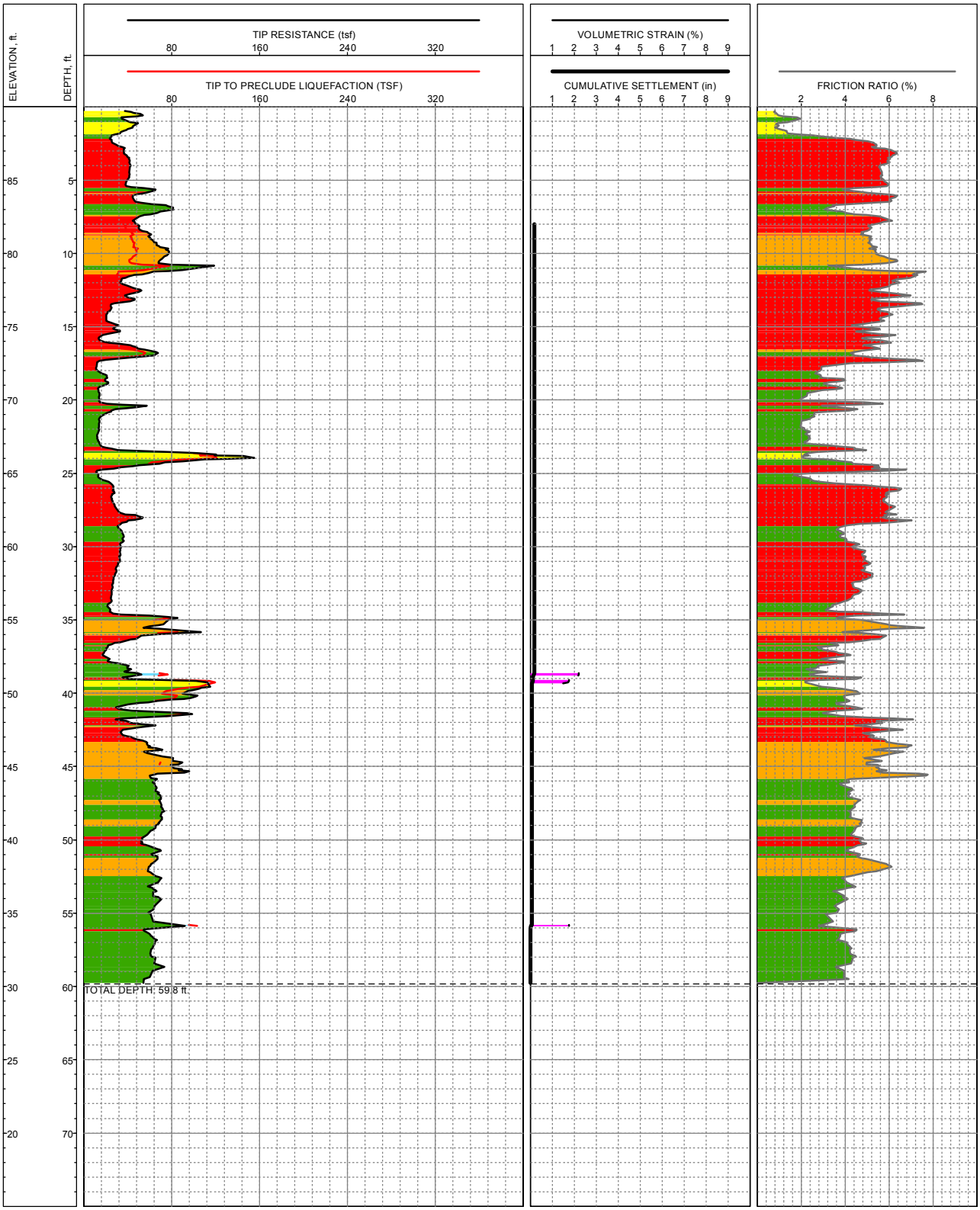
LOCATION: N: 2,132,228.59, E: 6,051,015.63
SURFACE EL: 92.0 +/- City of Oakland Datum
COMPLETION DEPTH: 60.0
TESTDATE: 2/16/2012

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY:
REVIEWED BY: J Kocijan
CONE AREA RATIO: 0.59

LOG OF CPT CPT-1
NCEER, M=7.3, PGA= 0.73g
CHRCO Outpatient Center No. 2
Oakland, California

PLATE D-2

N:\Projects\04_2012\04_7212_0009_CHRCO_Outpatient\Explorations\CPT\2012\Logs\2012_03_01_Logs_NCEER_7_3M_0_73a\MXD\Logs_VG12C_NCEER_7_3M_0_73a_ThinLens.mxd,07/23/2013,CDean



LOCATION: N: 2,132,243.80, E: 6,051,174.77
SURFACE EL: 90.0 +/- City of Oakland Datum
COMPLETION DEPTH: 59.8
TESTDATE: 2/16/2012

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY:
REVIEWED BY: J Kocijan
CONE AREA RATIO: 0.59

LOG OF CPT CPT-2
NCEER, M=7.3, PGA= 0.73g
CHRCO Outpatient Center No. 2
Oakland, California

PLATE D-3

N:\Projects\04_2012\04_7212_0009_CHRCO_Outpatient\Explorations\CPT\2012\Logs\2012_03_01_Logs_NCEER_7_3M_0_73a\MXD\Logs_VG12C_NCEER_7_3M_0_73a_ThinLens.mxd,07/23/2013,CDean

APPENDIX F3

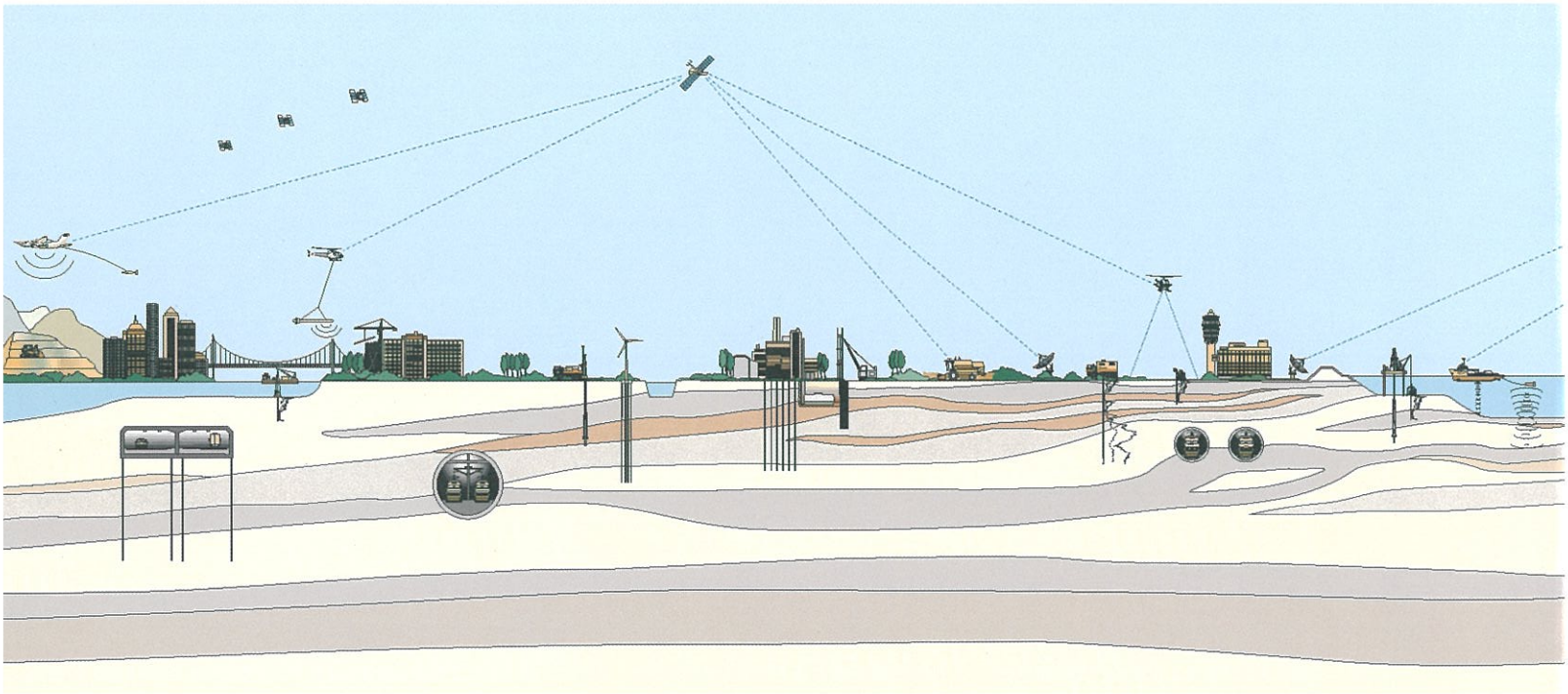
Phase-1 Geotechnical and Geohazard Evaluation, Central Utility Plant

FUGRO WEST, INC.

PHASE-1 GEOTECHNICAL AND GEOHAZARD EVALUATION CENTRAL UTILITY PLANT CHILDREN'S HOSPITAL & RESEARCH CENTER OAKLAND, CALIFORNIA

Prepared for:
CHILDREN'S HOSPITAL & RESEARCH CENTER AT OAKLAND

October 2008
Fugro Project No. 1595.002



FUGRO WEST, INC.

1000 Broadway, Suite 440
Oakland, California 94607
Tel: (510) 268-0461
Fax: (510) 268-0545

October 22, 2008
Project No. 1595.002

Children's Hospital & Research Center at Oakland
Doug Nelson c/o Sharon Knowlton
747 52nd Street
Oakland, California 94609

Attention: Mr. Doug Nelson

Subject: Geotechnical and Geologic Hazards Evaluation
New Central Utility Plant, Oakland, California

Dear Mr. Nelson:

Fugro is pleased to submit this geotechnical and geologic hazards evaluation report presenting the results of our field explorations, laboratory-testing program, and engineering analyses for the New Central Utilities Plant, located in Oakland, California.

We appreciate this opportunity to be of service to you. Please contact us if you have any questions regarding the information presented in this report.

Sincerely,

FUGRO WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
MARTIN J. WALKER
No. 70553
Exp. 12/31/2008
CIVIL
STATE OF CALIFORNIA

Martin Walker, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
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No. 2013
Exp. 6-30-2010
GEOTECHNICAL
STATE OF CALIFORNIA

Corey T. Dare, P.E., G.E.
Principal Engineer

REGISTERED GEOLOGIST
WILLIAM H. GODWIN
No. 2199
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

William H. Godwin, C.E.G.
Principal Geologist

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RONALD L. BAJUNIEMI
No. 112
Exp. 12/31/09
GEOTECHNICAL
STATE OF CALIFORNIA

Ronald L. Bajuniemi, P.E., G.E.
Principal Consultant

SEC/MW/RLB/CTD:rh

Copies Submitted: (2 + PDF) Addressee
(6 + PDF) Tom Onnen, HDR

CONTENTS

Page

1.0 INTRODUCTION 1
1.1 Project Description 1
1.2 OSHPD Compliance 1
1.3 Scope of Services 2
2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING 3
2.1 Review of Existing Data 3
2.2 Aerial Photograph Review 4
2.3 Field Exploration 5
2.4 Laboratory Testing 6
3.0 GEOLOGIC SETTING 6
3.1 Regional Geology 6
3.2 Regional Seismicity 7
3.2.1 Historic Seismicity 9
4.0 SITE CONDITIONS 11
4.1 Surface Conditions 11
4.2 Site Geology 11
4.3 Subsurface Conditions 12
4.3.1 Central Utility Plant (CUP) 12
4.4 Groundwater 13
5.0 GEOLOGIC HAZARDS EVALUATION 14
5.1 Surface Fault Offset Hazard 14
5.2 Shaking Hazards 15
5.2.1 Ground Motions 16
5.2.2 Liquefaction 16
5.2.3 Impacts of Liquefaction on Design and Ground Motion 18
5.2.4 Dynamic Densification 19
5.2.5 Lateral Spreading 19
5.2.6 Tsunami and Seiche 20
5.2.7 Landsliding 20
5.2.8 Other Shaking Hazards 21
5.3 Other Geologic Hazards and Complicating Site Conditions 21
5.3.1 Flooding and Inundation Due to Dam or Embankment Failure 21
5.3.2 Debris Flow 21
5.3.3 Land Subsidence 22
5.3.4 Soil Expansion 22
5.3.5 Corrosion Potential 22
5.3.6 Land Subsidence 24
5.3.7 Naturally Occurring Asbestos (NOA) 24
5.3.8 Radon-222 25
5.4 Conclusions - Geologic Hazards 25

CONTENTS – CONTINUED

Page

5.5 Other Hazards and Complicated Site Conditions Not Addressed In This Report 25

6.0 DISCUSSIONS AND CONCLUSIONS 26
6.1 Expansive Soils 26
6.2 Undocumented Fill 26
6.3 Groundwater 27
6.4 Foundation Support 27
6.5 Construction Considerations 27
6.5.1 Existing Structures and Foundations 27
6.5.2 Excavation Shoring and Dewatering 28
6.5.3 Earthwork 28
6.5.4 Construction Quality Control 28

7.0 RECOMMENDATIONS 28
7.1 Seismic Design Criteria 29
7.1.1 Seismic Parameters 29
7.1.2 Site Class, Seismic Design Parameters, and Seismic Design Category 29
7.2 Site Preparation and Grading 30
7.2.1 Site Preparation 30
7.2.2 Subgrade Preparation 30
7.2.3 Reworking Undocumented Fill 31
7.2.4 Engineered Fill Materials 31
7.2.5 Fill Placement and Compaction 31
7.2.6 Trench Backfill 32
7.2.7 Surface Drainage 32
7.3 Temporary Shoring and Construction Slopes 32
7.3.1 General 32
7.3.2 Temporary Construction Slopes 33
7.3.3 Temporary Shoring 33
7.3.4 Drilled Pier Foundations 34
7.3.5 Adjacent Existing Foundations 36
7.4 Lateral Loads 36
7.4.1 Long-Term Lateral Loads 36
7.4.2 Temporary Seismic Lateral Loads 37
7.4.3 Lateral Loads on Drilled Piers 37
7.5 Waterproofing and Designing for Hydrostatic Pressures 37
7.6 Concrete Slabs-on-Grade 38
7.6.1 Interior Slabs-on-Grade 38
7.6.2 Exterior Slabs-on-Grade 38
7.7 Retaining/Below Grade Walls 38
7.7.1 Restrained Walls 39
7.8 Pavements 39
7.9 Additional Geotechnical Services 41

CONTENTS – CONTINUED

Page

8.0 LIMITATIONS 42

9.0 REFERENCES 43

TABLES

Page

Table 1. Aerial Photographs Examined 4
Table 2. Large Magnitude (M≥6.0) Regional Earthquakes Within 100 Miles of the Site 10
Table 3. Summary of Corrosion Test Results for Buried Concrete 23
Table 4. Summary of Corrosion Test Results for Buried Ferrous Metal 24
Table 5. 2007 CBC Seismic Parameters 29
Table 6. Site-Specific Seismic Design Parameters 30
Table 7. Drilled Pier Design Skin Friction 34
Table 8. Passive Soil Resistance (P) versus Displacement (Δ) Relationship 36
Table 9. Lateral Capacity of Drilled Pier 37
Table 10. Recommended Pavement Design Alternatives 40

PLATES

Plate

Vicinity Map 1
Topographic Site Map 2
Site Plan 3
Creek Map 4
Geologic Map 5
Regional Fault Map 6a
Vicinity Fault Map 6b
Regional Epicenter Map 7
Cross-Section A-A' 8a
Cross-Section B-B' 8b
CGS Seismic Hazard Zone Map 9
Lateral Pile Analysis Results ..... 10a through 10c
Static Lateral Pressures on Restrained Walls 11a
Seismic Lateral Pressures on Restrained Walls 11b

CONTENTS – CONTINUED

APPENDICES

APPENDIX A FIELD EXPLORATIONS

Key to Soil Classification and Test Data ..... Plate A-1
Boring Logs ..... B-113 through B-115
Historic Boring Logs

APPENDIX B LABORATORY TESTING PROGRAM

Summary of Laboratory Test Results ..... Plate B-1
Plasticity Chart and Data ..... Plate B-2a
Plasticity Chart and Historic Data ..... Plate B-2b
Gradation Test Data ..... Plate B-3
Cerco Corrosion test results and Evaluation

APPENDIX C SEISMIC HAZARD ANALYSIS

APPENDIX D LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

1.0 INTRODUCTION

This report presents the results of the geotechnical study, and geohazards evaluation conducted by Fugro West, Inc. (Fugro) for the proposed New Central Utility Plant (CUP) for Children's Hospital & Research Center at Oakland (CHRCO). The new CUP will be located within the hospital campus bounded by Martin Luther King Jr. Way to the west, Highway 24 to the east and 52nd Street to the north in Oakland, Alameda County, California as shown on Plate 1 – Vicinity Map. The site coordinates are:

Latitude: 37.8380 º

Longitude: -122.2665 º

The project site is plotted on a 7 ½-minute United States Geologic Survey (USGS) quadrangle topographic map on Plate 2 – Topographic Site Map.

1.1 PROJECT DESCRIPTION

Based on information presented in the conceptual design drawings provided to us, we understand CHRCO is planning to construct a new 3-story Central Utility Plant (CUP) building along the eastern edge of the campus adjacent to Highway 24 and south of the existing Main Hospital. The footprint of this area is approximately 7,200 square feet. We understand the final finished floor elevation has not yet been determined, and for the purposes of this report have assumed it will be higher than 8 feet below existing grade (i.e., higher than Elevation 93 feet NAVD 88). We assume that appurtenant exterior flatwork and asphalt paved parking areas are included in the project.

Information from the project structural engineer indicates that interior column loads are on the order of 500 kips per column, and that the first floor slab will be a structural element supported by a deep foundation system.

1.2 OSHPD COMPLIANCE

California hospitals are classified as critical structures according to California Code of Regulations, Title 24, of the California Building Code. The office of Statewide Health Planning and Development (OSHPD) reviews proposed new construction of hospital facilities. California Geological Survey (CGS) serves under contract as the advisor to OSHPD for engineering geology and seismology issues, which includes the review of geologic hazard reports.

This report has been prepared to satisfy the guidelines presented in *California Geological Survey – Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.* The most recent revision of Note 48 is October 2007.

One of the CGS requirements is that approximately one subsurface exploration point be completed for every 5,000 square feet of proposed building footprint area. Previously

completed subsurface explorations and laboratory data can be used to partially satisfy this requirement. New borings, which are a part of this study, represent the additional borings necessary to meet the 5,000 square foot criteria. The locations of the borings were chosen based on accessible areas of the site to adequately characterize subsurface conditions at the location of the new CUP.

1.3 SCOPE OF SERVICES

The purpose of our geotechnical field exploration and laboratory-testing program is to obtain information on subsurface conditions to evaluate the geotechnical aspects of the various portions of the site where structures will be located. The scope of our services included:

- Compiling and reviewing available geotechnical and geologic data that is contained in our files and is pertinent to the project vicinity. This includes a series of previous reports prepared for various developments at the site;
- Conducting a field exploration and laboratory-testing program to supplement the available information on subsurface conditions at the proposed development locations;
- Evaluating the geologic hazards at the site (a stability evaluation of the existing Caltrans Highway 24 entrance ramp is beyond this scope of work);
- Performing engineering analyses of the collected data and developing geotechnical recommendations and design criteria for the proposed hospital redevelopment;
- Preparing this geotechnical report presenting the results of our geotechnical field exploration, laboratory testing program, discussion of geotechnical issues and geologic hazards and our geotechnical recommendations.

2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING

The exploration and laboratory-testing program described herein was developed to provide general characterization of the subsurface materials.

2.1 REVIEW OF EXISTING DATA

Prior to conducting our field exploration and laboratory testing, Fugro reviewed relevant information relating to geotechnical, geologic, and seismic issues at the site. Fugro's local predecessor companies have completed several geotechnical investigations at the Children's Hospital site since 1974. Historic boring locations from our previous work at the site are shown on Plate 3 – Site Plan. Data from these borings was used in preparing the cross sections presented in this report, and for our engineering analyses. We reviewed the results of these previous studies performed at the site, including the following reports that contain information directly or indirectly related to the current project:

- Fugro (May 2008) "New Hospital Replacement Project": Fugro performed geotechnical, geological and seismic hazard evaluation for a proposed new hospital replacement project including a 12-story hospital building north of 52nd Street, as well as a parking structure and CUP at the southern end of the CHRCO campus . The CUP location studied by this report has been relocated to the currently proposed project location. Various analyses had been performed as part of the scope satisfying the CGS requirements. This report was used as a primary source of data in our current investigation.
- Harza (1994) "Proposed Patient Services Pavilion, Phase I": Harza completed six borings as part of a geotechnical and seismology study for a proposed patient services pavilion. These borings extended to depths up to 51 feet below grade. Laboratory testing was completed on samples collected from these borings, including index and strength tests. Two borings were converted to monitoring wells to observe groundwater levels. Monitoring well data were used for groundwater information in this report.
- Kaldveer (1987) "Foundation Investigation for Nursing Tower": Kaldveer completed two borings and related laboratory testing for the nursing tower on the south side of the Children's Hospital campus. These borings were drilled near the proposed location of the CUP. This report was used as a primary source of data in our current investigation.
- Kaldveer (1986a) "Foundation Investigation, Family Center": Kaldveer completed four borings as part of a geotechnical study for the Family Center on Dover Street. These borings were located approximately 600 feet north of the proposed CUP site. This report was used as a source of groundwater data in our current investigation.
- Kaldveer (1986b) "Foundation Investigation, Proposed Medical Office Building and Parking Structure": Kaldveer completed seven borings as part of a geotechnical study for the existing medical office building and parking structures on the north portion of the CHRCO campus. These borings were located approximately 650 feet

northwest of the proposed CUP site. This report was used as a source of groundwater data in our current investigation.

A complete list of references used in this study, including all previous studies performed by Fugro at the CHRCO campus and surrounding area, is included in Section 9.0 – References. Pertinent previous boring logs used to interpret subsurface conditions at the proposed structure are included in Appendix A.

2.2 AERIAL PHOTOGRAPH REVIEW

Six sets of vertical angle, black and white aerial photographs taken between 1930 and 1990 were examined both in stereo pairs and as single photos. Stereo views are used to determine relief and shape of landforms, which are commonly controlled by bedrock type and tectonic movement. Single photographs are reviewed for tonal variations and patterns that often go unnoticed in stereo because of the high relief. The historic photos were obtained from the Pacific Aerial Surveys library. A list of the photographs reviewed is provided in Table 1 below.

Table 1. Aerial Photographs Examined

Flight Date	Approximate Scale	Serial Number
6/12/90	1:12,000	AV3845-7-21/22
5/2/69	1:12,000	AV 902-07-17/18
7/25/63	1:36,000	AV 550-08-19/20
5/3/57	1:12,000	AV 253-08-21/22
3/24/47	1:20,000	AV 11-04-11/12
1930	1:9,500	GY-30-38/39/69/70

Each photoset was examined for changes in vegetation and geologic conditions related to soil movements and bedrock conditions. Interpretations are generally based on three factors: tone, texture and trend. Tonal and textural variations, if present, may be directly related to 1) the rocks or soils underlying the site, 2) water content of the underlying material, 3) natural vegetation (commonly controlled by local geology and water content), or 4) human activities (surface modification) including agriculture. Directional trends are observed as linear features.

The examination for this investigation was focused on the identification of landslides and active faulting. Landslides are identifiable through such features as headscarps, concave slopes, lateral drainages, hummocky terrain and lobate toes. Faults are expressed by offset tonal or textural bands, tonal lineaments, isolated lines of springs or trees, abrupt changes in slope or drainage direction, and by streams with anomalously straight or curved courses.

No geologic hazards were observed in the aerial photos that could affect the proposed development. Our photograph review indicates no presence of historic landslides and no features indicative of active faulting at the site. Linear features associated with the Hayward

fault are visible (1963 photo) about 2 miles east of the site. No other features relating to active faulting were observed in the vicinity of the CHRCO campus and project site.

The earliest aerial photographs reviewed for the CHRCO campus and project site were from 1930. At that time, the area surrounding the hospital was developed as a predominately residential neighborhood. The site appears to be developed with an early hospital tower. However, the tower in the photo is not in the configuration of today. To the south of the site, in the vicinity of the intersection of Martin Luther King Way and 47th Street, Temescal Creek is visible. The location of Temescal Creek is shown on Plate 4 – Creek Location Map, and discussed in Section 3.1 below. The close proximity of this creek implies the presence of paleochannels beneath the site, created from a meandering path. No additional features indicative of geologic hazard are visible within or trending towards the subject property. No significant changes were made before the 1947 photos.

The 1957 aerial photograph shows the existing Children's Hospital tower building as the structure appears today. The hospital is surrounded on all sides by residential property. Temescal Creek is visible on the same path as earlier photos. No features indicative of any geologic hazard are visible within or trending towards the subject property.

By 1963 the main u-shaped hospital tower has been constructed. This structure is in the same configuration today, and residential neighborhoods continue to surround the site. Temescal Creek is visible to the south; however, more of the creek path has been diverted into underground culverts.

By 1969 additional hospital towers have been constructed directly northwest of the main tower. Caltrans has constructed Highway 24 in the right-of-way directly east of the hospital. The elevated roadway includes retaining wall and embankment slope with an approximate inclination of 2.5:1 (horizontal to vertical).

The 1990 photograph was the earliest photograph reviewed after the 1989 Loma Prieta Earthquake. No features were readily observed as being related to earthquake-induced damage, such as faulting, liquefaction, or lateral spreading.

2.3 FIELD EXPLORATION

Fugro's field investigation for the project included drilling three new geotechnical borings in the vicinity of the proposed CUP building footprint. The borings were located in the field by our engineer based on existing site features, and to provide adequate coverage to help satisfy CGS requirements for adequate exploration points for the proposed CUP structure.

V&W Drilling, of Lodi, California, drilled the geotechnical borings between August 4 and 5, 2008. The borings were drilled with a truck-mounted drill rig equipped with 8-inch hollow stem augers and mud rotary drilling capabilities. Three borings were drilled for the proposed CUP, designated B-113, B-114 and B-115. The borings extended to depths of 21.5 to 51.5 feet below existing grade. All borings were backfilled with cement grout in accordance with our Alameda County drilling permit.

A Fugro engineer observed the drilling operation and logged the soil encountered. Our engineer recorded SPT N-values and estimated shear strength of the soils using a Pocket Penetrometer. Recovered soil samples were visually examined and classified in accordance with the Unified Soil Classification System (USCS) and ASTM D-2488 (visual method). Soil samples were transported to our office and laboratory for further examination, confirmation of classification and laboratory testing.

Detailed descriptions of the soils encountered in each of the borings are presented on the boring logs in Appendix A. The boring logs depict location specific subsurface conditions. The approximate locations of the borings were estimated by taping from existing landmarks and the locations should be considered accurate only to the degree implied by the method used. Elevations shown on the boring logs are in reference to the North American Vertical Datum of 1988 (NAVD88), and were estimated from contours on the project topographical survey provided by Chaudhary & Associates, Inc. The subsurface conditions encountered in the soil borings are summarized in Section 4.3.

2.4 LABORATORY TESTING

Geotechnical laboratory testing was conducted on the soil samples collected from the borings at Fugro's soil mechanics laboratory in Oakland, California. The geotechnical laboratory test program included: classification tests (gradation, fines content, Atterberg limits, water content, unit weight), and miniature vane strength tests. The results of the laboratory tests are presented on boring logs (Appendix A) at the appropriate sample depths, and in Appendix B - Laboratory Test Results.

Corrosion testing was conducted on one soil sample in the vicinity of the Proposed CUP at CERCO Analytical, Inc., in Pleasanton, California as part of the New Hospital Replacement Project Investigation completed by Fugro in May 2008. Corrosion potential is discussed further in Section 5.3.5 – Corrosion Potential. Results of the corrosion testing are provided in Appendix B.

3.0 GEOLOGIC SETTING

3.1 REGIONAL GEOLOGY

The site is located within the San Francisco Bay alluvial plain. This area, along with the Diablo Mountain Range, and the Santa Cruz Mountains are within the Coast Range Geologic Province, a belt of sedimentary, volcanic, and metamorphic rocks that extend from southern California to Oregon. The structural geology of the Coast Range is complex and dominated by transpressive stress caused by the interaction between the Pacific and North American plates. This stress, concentrated along faults within the San Andreas fault system, results in the series of northwest-trending ridges and valleys that characterize the area. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates, subduction of the Pacific Plate beneath the North American Plate, and subsequent strike-slip faulting along the Proto-San Andreas, and San Andreas fault system.

Strike-slip motion along the plate boundaries replaced subduction several million years ago (Atwater, 1970).

Geologic formations in the San Francisco Bay region range in age from Jurassic (190 to 135 million years ago) to recent Holocene (less than 11 thousand years ago). The oldest rocks are deformed Mesozoic sedimentary and volcanic rocks of the tectonically accreted Mesozoic Franciscan Complex and the contemporaneous Great Valley Sequence. During the last glacial maximum, significant relief of the bedrock was formed as a result of incision of creeks and streams to reach the elevation of the global sea level; approximately 140 feet lower than sea level today. As sea level rose with the onset of the current interglacial, lowland areas in the San Francisco Bay Area were subject to the deposition of a transgressive sequence of alluvial sediments, ranging in age from Pleistocene to the present. Younger alluvial, marine and marsh deposits have accumulated in the valleys in the region as a result of weathering in the surrounding mountains and sea level rise.

The regional geology, as mapped by Graymer (2000) is presented on Plate 5 – Geologic Map. Geologic units that are exposed within a mile of the site include the sandstone and quartz diorite members of the Novato Quarry terrane of the Franciscan Complex (Kfn, Kfgm), and surficial alluvial sediments (Qhaf, Qhl, Qpaf). Holocene aged alluvial fan deposits (Qhaf) underlie the site. Graymer (2000) described this formation as "alluvial fan and fluvial deposits, [a] brown or tan, medium dense to dense, gravelly sand or sandy gravel that generally grades upward to sandy or silty clay." An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site as shown on Plate 5 – Geologic Map. The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. The fault does not appear to impact the Pleistocene aged alluvial deposits mapped in the vicinity. The fault location is poorly constrained, and is not considered active, nor has the fault been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

The regional geomorphology is generally controlled by the active Hayward fault, which is located approximately 2 miles east–northeast of the project. Local geomorphology is controlled by fluvial deposition associated with Temescal Creek.

3.2 REGIONAL SEISMICITY

Geologists and seismologists recognize the San Francisco Bay Area as one of the most active seismic regions in the United States. Dominated by the San Andreas fault system, the Bay Area is comprised of mostly northwest-trending strike-slip faults driven by the interaction of the Pacific and North American Plates. Movement between these two plates is predominantly accommodated on the San Andreas, Hayward-Rodgers Creek, Calaveras, San Gregorio, and Concord-Green Valley faults. The major fault in the system is the San Andreas fault, a major rift in the earth's crust that extends for at least 450 miles.

In 2003, the Working Group on California Earthquake Probabilities (WGCEP), in conjunction with the United States Geological Survey (USGS), published an updated report

evaluating the probabilities of significant earthquakes occurring in the Bay Area over the next three decades (2002-2031). WGCEP found a 62 percent probability that at least one magnitude 6.7 or greater earthquake will occur in the San Francisco Bay region before 2031. This probability is an aggregate value that considers seven principal Bay Area fault systems and unknown faults (background values). The San Francisco Bay region continues to be seismically active. The principal active faults in the Bay Area include the San Andreas, Hayward, Calaveras, and the San Gregorio faults. Earthquakes occurring along these faults are capable of generating strong ground shaking at the project site. The locations of significant Bay Area faults are shown on Plate 6a – Regional Fault Map, and Plate 6b – Vicinity Fault Map.

The closest active fault to the site is the Hayward fault., approximately 2 miles (3.2 km away). The Hayward fault was originally mapped by Lawson (1908), as part of his report to the State Earthquake Investigation Commission regarding the April 18, 1906 San Francisco earthquake. This report included a "comparison with other severe earthquakes in the same region", where he compiles data obtained from newspapers and eyewitness accounts. The fault was named after the city of Haywards (now Hayward), the site of the greatest damage.

The Hayward fault is the southern part of an extensive fault system that includes the Hayward, Rodgers Creek, Healdsburg, and Maacama faults. This system of faults extends over 280 kilometers (km) from San Jose to Mendocino County (Steinbrugge and others, 1987). The Hayward fault extends from the southwestern margin of the East Bay Hills for a distance of 105-km from San Pablo Bay in the north to an area near Mount Misery, east of San Jose, where slip is transferred to the Calaveras fault via a complex system of oblique slip and reverse faulting. The fault is divided into three segments: the 42-km long northern segment which extends from the northwestern margin of San Pablo Bay to the vicinity of San Leandro, the 34-km long southern segment, which extends from San Leandro to the Warm Springs district of Fremont, and the 28-km long southeast Extension segment which extends from the Warm Springs District to the vicinity of Mount Misery (Bryant and Cluett, 2000). The segment closest to the project site is the northern Hayward fault. Based on earthquake focal mechanisms, the Hayward fault is near vertical. However, secondary fault traces are common and exhibit west-vergent structures that may merge at depth with the Hayward fault. Geomorphic evidence for the northern segment of the Hayward fault includes, offset stream channels, sag ponds, and distress to cultural features as a result of fault creep.

A robust recurrence interval has not been developed for this segment, but Lienkaemper and others (1997) concluded there have been at least four surface rupturing earthquakes in the last 2,250 radiocarbon years. The most recent event has been dated as after 1640 AD, but before 1776 AD. The 1836 magnitude 6.8 earthquake was originally believed to have occurred on this segment of the Hayward fault, however, recent work by Toppozada and Borchardt (1998) re-evaluated this earthquake to locate the epicenter further south, east of Monterey Bay.

The latest assessment of the U.S. Geological Survey Working Group on California Earthquake Probabilities (2003) has given the northern segment of the Hayward fault an 11 percent probability of exceedance, and the Hayward-Rodgers Creek fault in its entirety a 27 percent probability of exceedance, in the 30 year period ending in 2031 for a magnitude 6.7 or greater earthquake. Holocene displacement on the Hayward fault is predominantly right lateral,

although varying degrees of vertical slip are indicated by the presence of escarpments along the fault's surface trace. Lienkaemper and Borchardt (1992) evaluated the aseismic slip rate for the Hayward fault. They found the area of the highest aseismic slip, 9 mm/yr (the southern Hayward fault segment in the vicinity of Fremont, California), and assumed that represented the minimum slip rate of the fault at depth. The historic surficial slip rate measured in the Oakland area only accounts for approximately 4 mm/yr of this total. It is assumed, therefore, the Hayward fault in this area is locked at depth and has the potential to rupture in large events.

The approximate distances of the site to the closest known mapped active faults are summarized in Table C-1 in Appendix C. Table C-1 summarizes the seismic sources and their pertinent characteristics. These data were used as part of the site-specific seismic hazard analyses discussed in Appendix C. The distances to faults were estimated using Boore et al. (1997). Fault parameters were adapted from the California Geologic Survey (CGS) fault database (2003).

Earthquakes on these or other active faults (including unmapped faults) could cause strong ground shaking at the site. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site, and other factors.

The project site is not located within an Alquist-Priolo Earthquake Fault Zone (discussed in further detail in Section 5.1 below).

3.2.1 Historic Seismicity

Since the first historical reports from the late 1700s, major earthquakes have been recorded along the San Andreas fault system in the Bay Area. Table 3 presents large magnitude ($M_w \geq 6.0$) earthquakes that have occurred within 100 km of the site between 1800 and 2007. The locations of the closest historic earthquakes to the project are shown on Plate 7 – Regional Epicenter Map. The computer program EQSEARCH, v3.00 (Blake, 2001) was used to compile the seismicity information presented in Table 2.

Table 2. Large Magnitude (M≥6.0) Regional Earthquakes Within 100 Miles of the Site

Epicenter Location	Date	Moment Magnitude	Distance (mi)	Distance (km)	Compass Direction to Epicenter
San Francisco	June 21, 1808	6.0	12.9	20.7	Southwest
Hayward	October 21, 1868	7.0	13.2	21.3	South
San Francisco	April 18, 1906	8.25	15.8	25.4	Southwest
San Francisco Peninsula	June 1, 1838	7.0	17.9	28.8	Southwest
Antioch	May 19, 1889	6.25	23.0	37.0	East
Mare Island	March 31, 1898	6.5	26.0	41.9	Northwest
San Jose	December 26, 1858	6.25	30.8	49.6	Southwest
Vacaville	April 19, 1892	6.5	41.5	66.8	Northeast
South Santa Cruz Mountains	October 8, 1865	6.5	42.2	68.0	South
1984 Morgan Hill	April 24, 1984	6.1	47.5	76.4	Southeast
1911 Morgan Hill	July 1, 1911	6.5	49.5	79.7	Southeast
Winters	April 21, 1892	6.25	49.9	80.4	Northeast
Loma Prieta	October 18, 1989	7.1	59.2	95.3	South
Gilroy	June 20, 1897	6.25	71.6	115.2	Southeast
Pajaro Gap	April 24, 1890	6.25	74.4	119.7	South
Pajaro Gap	October 18, 1800	7	79.4	127.8	South
Pacific Ocean	October 22, 1926	6.1	84.9	136.6	South

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The topography of the area surrounding Children's Hospital is shown on Plate 2 – Topographic Site Map. Note Plate 2 presents a USGS topographic map, with elevations referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). The standard conversion from NGVD29 to North American Vertical Datum, 1988 (NAVD88) is to add 2.38 feet to elevations referencing the NGVD29 datum. The local topography and surface conditions at the proposed structure are discussed below.

The CUP is located in the southern portion of the Children's Hospital campus, and is surrounded by the existing hospital to the north, the Highway 24 entrance ramp directly to the east and the existing helipad (helistop) structure to the west. Temporary office and storage structures, and some asphalt paved parking areas currently occupy the western portion of this site. The exact location of the CUP has not been finalized, pending negotiations with Caltrans for right-of-way encroachment into the Highway 24 embankment. The eastern side of the proposed CUP structure is situated adjacent to the embankment associated with the Highway 24 ramp. The surface in this portion of the site is heavily wooded with brush, grasses, and some trees. A chain link fence bisects the CUP site separating the eastern and western portions of the proposed building footprint.

The site is flat, with a gentle slope increasing from about Elevation 100 feet at the southern tip of the CUP structure, crowning to about Elevation 101 feet near the center of the building moving north, then returning to about Elevation 100 feet at the northern tip of the structure. However, depending on final location, the site could encroach onto the adjacent, approximately 30-foot high, 2.5:1 inclined slope of Highway 24.

4.2 SITE GEOLOGY

The Children's Hospital site is located on a gradually sloping outwash plain located between the Berkeley Hills to the east and San Francisco Bay to the west. The native soils at the site consist of Quaternary age alluvial deposits likely including materials from both the San Antonio and Temescal Formations, according to Radbruch and Case (1967) and Helley, Lajoie and Burke (1972). Graymer (2000) maps the site as Holocene age alluvial and fluvial deposits, as shown on Plate 5 – Geology Map.

Radbruch and Case (1967) map the site as being underlain by the Temescal Formation (Qtc). The Temescal Formation fills old channel meanders (of uncertain location) that were cut in the underlying San Antonio Formation (Qsu), and as such do not uniformly underlie the area. The thickness of the Temescal Formation varies from 3 to 18 feet. The nomenclature "Temescal Formation" was first used by Lawson (1914) to comprise several presumably contemporaneous alluvial units of different origin, lithology, and physical properties. The source material in the vicinity of the project location is assumed to be San Antonio Formation, which consists of "clay, silt, sand, and gravel, some pebbles". Most beds contain flakes or pebbles of

white Claremont chert. Clays of this formation are often montmorillonite, and highly plastic. The maximum thickness of the San Antonio Formation is unknown.

Sowers (1993, rev. 2000) mapped modern day hydrology of Oakland and its vicinity, and where available, historic creek paths were included. Temescal Creek was mapped as being contained in a culvert, passing directly south of the project location (Plate 4 – Creek Location Map). As described in Section 4.3, Subsurface Conditions, paleochannels resulting from the meandering path of the creek underlie portions of the project site.

4.3 SUBSURFACE CONDITIONS

The alluvial stratigraphy encountered across the CHRCO campus site typically consisted of a layer of lean sandy clay fill, over a layer of stiff lean clay, over interlayered clayey sands to sandy clays with varying amounts of gravel over stiff lean clay. The clayey sands encountered within the interbedded layer are interpreted to be the paleochannels associated with a meandering paleo-Temescal Creek. These deposits appear to be at various stages within the rubification process (e.g., the reddening of soils through the release and precipitation of iron as an oxide during weathering). In general, there appears to be two paleochannel deposits that may or may not be laterally continuous (see Plates 8a and 8b). One is shallow and appears to be relatively laterally continuous with a greater percentage of fines, than the deeper, more poorly graded deposit. These deposits may or may not connect at depth. The geologic units are described in order of increasing depth. Detailed descriptions of the deposits encountered in each of the borings are presented on the boring logs in Appendix A. Interpreted subsurface profiles are shown on Plates 8a and 8b.

The attached boring logs and related information depict location specific subsurface conditions, encountered during our field exploration. The passage of time could result in changes in the subsurface conditions.

4.3.1 Central Utility Plant (CUP)

Similar to the hospital campus in general, the area at the southern end of the CHRCO campus is generally characterized by four layers, described below in order of increasing depth: I) undocumented sandy and clayey fill; II) lean clay; III) interlayered clayey sands and sandy clays with gravel; and IV) very stiff to hard clay with varying amounts of sand and gravel.

Beneath the asphalt at the CUP is a layer of fill ranging from stiff sandy lean clay to silty clay. The fill was observed to range from black to brown to reddish-brown and contains varying amounts of gravel. This fill layer is estimated to be about 4 to 8 feet thick beneath the CUP. Previous investigations indicate that the thickness of the fill increases to about 11 to 18-feet thick towards the southern extent of the hospital campus. The thicker fill is likely associated with an existing buried concrete box culvert at the southernmost portion of the hospital campus. One Atterberg limit test for this investigation shows a measured Plasticity Index in the fill of 10 percent. However, historical borings show the Plasticity Index for this fill in the southern end of the site to range typically from about 12 to 21 percent, suggesting a moderate expansion potential.

Beneath the fill at the CUP, a layer of stiff to very stiff sandy lean clay with varying amounts of sand and gravel was encountered. This clay layer was consistently observed to be about 15-feet thick. Water content measured on samples in this layer ranged from about 16 to 23 percent, with an average of about 20 percent. Undrained shear strength (Su) measurements in this layer from miniature vane tests and unconfined compression tests range from about 2 kips per square foot (ksf) to over 4 ksf.

Previous investigations indicate that below the stiff clay layer exists interlayered sandy clays and clayey sands with varying amounts of gravel. The individual clayey sand and sandy clay layers range in thickness from about 5 to 10 feet. Gradual transitions were often observed between the sandy material and the clayey material. Laboratory testing indicates the sandy material typically contains a significant amount of clay (at least 10 percent and often higher than 35 percent) and the clay material contains significant amounts of sand. Both the sand and the clay layers contain a significant amount of gravel. The sandy units were typically medium dense to dense, and the clay units were typically firm to stiff.

Previous investigations also indicate that below the interlayered sands and clays, there exists a very stiff to hard low to medium plasticity (lean) clay layer containing a varying proportion of granular sands and gravels. This layer was typically encountered between 40 and 50 feet below grade at the proposed CUP site. Recorded SPT N-values below 50 feet were very high, typically in excess of 30 blows per foot.

4.4 GROUNDWATER

Groundwater levels were observed to vary significantly across the site during our subsurface investigation. Groundwater was encountered at a depth of 25 feet below surface in B-115, 20 feet below surface in B-113, 25 feet below surface in B-116. In Boring B-112, a boring completed in February 2008, groundwater was observed at a depth of 18 feet below ground surface. Groundwater was typically measured within 30 minutes of completing the boreholes because the borings were backfilled immediately after drilling with cement grout in accordance with our Alameda County drilling permit. The borings may not have been left open for a sufficient period of time to establish equilibrium ground water conditions.

As part of the geotechnical investigation for the Patient Services Pavilion, Harza (1994) installed two monitoring wells at the existing hospital. One Monitoring Well, EB-1 (OW), is located near the southern courtyard and the other Monitoring Well, EB-6 (OW), was located in the physicians parking lot near the entrance gate (approximate locations are shown on Plate 3 – Site Plan). Water levels were recorded in September of 1992, approximately 50 days after drilling, and in March of 2008 as part of this study. Groundwater in EB-1 (OW) was measured at 22 feet (Elevation 79 feet) in 1992, and at 21 feet (Elevation 80 feet) in 2008. For EB-6 (OW), groundwater was measured at 16 feet (Elevation 86.5 feet) in 1992 and 12.5 feet (Elevation 90 feet) in 2008.

Considerable historical site groundwater data are available in previous geotechnical reports completed for projects on the CHRCO campus. Review of these reports indicates that groundwater levels at the Children's Hospital site have varied considerably. Observed levels

range from 6 feet below grade in a boring completed in April 1986 (K876-3, 1986), north of the CUP site and northeast of the present-day CHRCO campus; to 23 feet below grade in (K928-1, June 1986), in a boring located in the existing parking garage area. For the June investigation a boring was converted into a temporary observation well and groundwater was recorded at a depth of 17 feet below grade three weeks after drilling. Near the Ambulatory Services Center, north of 52nd Street and to the west of Dover Street, groundwater was recorded at depths of 12 to 23 feet during investigations in 1986, 1990 and 1991.

The CGS Seismic Hazard Zone Report for the Oakland West Quadrangle (1991) presents a map that shows approximate depths to historic high ground water. Groundwater depth contours presented on this map suggest historic high ground water at the Children's Hospital site to be on the order of about 7.5 feet below grade.

Based on the observed historical depths of groundwater at the Children's Hospital site we have estimated a design ground water depth of 8 feet below ground surface for use in our analyses. This depth corresponds to an approximate elevation of 93 feet at the proposed location of the CUP. Fluctuations in the groundwater level may occur from changes in seasons, variations in rainfall, and other factors.

5.0 GEOLOGIC HAZARDS EVALUATION

Geologic hazards can be grouped into seismic and non-seismic categories. Seismic hazards can be further subdivided into surface fault offset, ground shaking, seismic ground deformation and water movements (tsunamis & seiches). Seismic ground deformation includes liquefaction, dynamic densification and landsliding. Certain types of ground deformation and water movement, however, can also occur under non-seismic conditions, so there is some overlap between seismic and non-seismic hazards. In general, the list of potential geologic hazards can also include such rare occurrences as mine collapse, hydrocompaction and peat oxidation. These hazards are rare and do not warrant discussion in connection with the project site as peat deposits were not encountered in the explorations. In addition, there is no mention of mines underlying the site in the literature we reviewed.

Historical evidence and the results of current seismological research indicate that at least one moderate to severe earthquake will occur sometime during the design life of the proposed development. Geologic hazards in this region typically become more critical during strong earthquakes. Detailed discussions of these hazards with respect to the site are presented below.

5.1 SURFACE FAULT OFFSET HAZARD

The potential for surface fault rupture at the site is low. No known active fault traces are mapped crossing the Children's Hospital Oakland site, based on existing geologic maps and reports including Graymer (2000) and Helley and Graymer (1997). The site is not located within a Fault-Rupture Hazard Zone as designated by the Alquist-Priolo Earthquake Fault Zoning Act (1972), as mapped on the official Earthquake Fault Hazard Zone Maps issued by the State of California (1982). The site is approximately 2 miles west of the closest active fault zone

(Hayward fault) depicted on the Special Studies Zones map (CGS, 1982), as shown on Plate 9 – CGS Seismic Hazard Zone Map. The Fault Activity Map from the CGS digital database (CGS, 2000), as shown on Plates 6a (regional view) and Plate 6b (vicinity view), shows the Hayward fault to the east of the site.

An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site (Plate 5 – Geologic Map). The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. Based on Graymer (2000), this fault does intersect the ground surface within Pleistocene aged alluvial deposits near the site vicinity. The fault location is poorly constrained, and is not considered active, nor has it been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

No indications of surface fault displacement were found in our reconnaissance of the site. While the possibility of new fault breaks developing during future earthquakes cannot be precluded, historic occurrences of surface fault rupture have generally followed pre-existing active fault traces.

5.2 SHAKING HAZARDS

Strong ground shaking at the Children's Hospital site will likely occur during a moderate to severe earthquake occurring on one of the active Bay Area faults. Strong ground shaking can cause the structures to shake and also has the potential to induce other phenomena that can indirectly cause damage to structures. These phenomena include: soil liquefaction, seismically induced waves such as tsunamis and seiches, inundation from dam or embankment failure, landsliding, lateral spreading, differential compaction, and ground cracking. Detailed discussions of these phenomena with respect to the site are presented in the subsequent paragraphs.

Youd and Hoose (1978) compiled data related to damage caused by historic earthquakes in the San Francisco Bay region, including the 1906 San Francisco and 1868 Hayward Earthquakes. To the south, in the vicinity of the Alameda and Oakland border, "the [1906] earthquake did not produce, relatively speaking, much destruction in these [Oakland, Alameda, and Berkeley water systems] works." Damage was limited to minor settlement and pipeline breaks, stream bank failures, and minor lateral displacements on the order of 8 inches. No features were noted in the project area as a result of the 1906 San Francisco Earthquake. In 1868, "portions of the wharves were carried away in some instances, while walls were cracked in almost every house". No features were noted in the Children's Hospital vicinity or surrounding areas as being related to liquefaction.

Liquefaction data are also available for the 1989 Loma Prieta Earthquake. Tinsley and others (1998) mapped liquefaction features nearby in Berkeley and Emeryville. Damage included sand boils, cracks, ground settlement, lateral spreading and pipeline breaks. "Lateral spreading and settlement caused extensive pavement damage to the freeway road surface. Pavement cracking was oriented parallel to the shoreline, with a total lateral movement of 30 to

120 mm. Several cracks were more than 50 m long." No other features indicative of liquefaction or lateral spreading were identified in the vicinity of the Children's Hospital site as a result of the 1989 Loma Prieta Earthquake.

5.2.1 Ground Motions

Site-specific probabilistic seismic hazard analyses were conducted to generate horizontal response spectra. The spectra were modified to incorporate the near source effects, and factors were applied to generate fault normal spectra. Detailed information regarding our site-specific seismic hazard evaluations are presented in Appendix C.

The design response spectrum and Maximum Considered Earthquake (MCE) events were developed per the 2007 CBC requirements and per the analysis procedures outlined in the design guidelines prepared by the American Society of Civil Engineers (ASCE), ASCE 7-05. Site-specific response spectra were developed for this project. The development of site-specific spectra included conducting probabilistic hazard analyses for 2475-year return period, as well as conducting deterministic checks that are required by ASCE 7-05 and the OSHPD draft Code Application Notice (CAN) dated March 24, 2008 and released in August, 2008. Detailed information regarding the development of site-specific spectra is presented in Appendix C. The design peak horizontal ground acceleration (PGA) for the site is 0.76g, based on our site-specific analyses (Table C-2).

The hazard analyses and ground motion evaluations were performed based on our understanding that the CUP structure will be founded on a deep foundation system. If the foundation system is significantly changed, the applicability of the results presented in Appendix C should be re-evaluated with the selected alternate foundation system. The results presented in Appendix C are applicable only at the foundation level, based on the understandings mentioned above.

5.2.2 Liquefaction

The potential for liquefaction occurring at the site is considered low. Settlement can occur as a result of seismic shaking caused by liquefaction of subsurface soils. Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers located close to the ground surface. As very loose to medium dense saturated granular soils are shaken, their tendency to contract and compress may lead to the development of excess pore pressures. If the seismic shaking is strong enough and long enough, the buildup in pore pressure can produce a significant loss of shear strength.

Liquefaction is said to occur when the excess pore pressure equals the initial effective stress in the soil. If the shaking continues after the onset of liquefaction, ground distress may occur (e.g., sand boils, settlement, lurching, and lateral deformation). Liquefaction may also cause a loss of capacity of shallow or deep foundations, loss of lateral capacity or stiffness, and lateral ground spreading.

The susceptibility of a soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in a) distribution of grain size, b) soil density, c) aging, d) cementation, e) fines content, and f) plasticity characteristics of the fines. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface, a depth usually considered to be less than about 50 feet.

The liquefaction susceptibility at the project site is considered to be moderate, based on the Association of Bay Area Governments (ABAG) Liquefaction Susceptibility Map (2000). Witter and others (2006) mapped the quaternary deposits and evaluated the liquefaction susceptibility for the San Francisco Bay Area. The map shows the soils at the site as Holocene aged alluvial fan deposits, and as such, the liquefaction susceptibility is dependent on the depth to groundwater in the vicinity. The liquefaction susceptibility is considered moderate for groundwater depths of 10-30 feet below ground surface and high for groundwater less than 10 feet below ground surface. The site has a moderate to high liquefaction susceptibility as mapped by Witter and others (2006). The site is located in an area designated as a zone of required investigation for liquefaction by California Geological Survey (CGS), as shown on the Seismic Hazard Zones Maps, West Oakland Quadrangle (CGS, 2003), included as Plate 9 – CGS Seismic Hazard Zone Map.

Our liquefaction evaluation was conducted using the procedures recommended by the National Center for Earthquake Research, and summarized by Youd et al (2001). The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.76g. This PGA represents the design response spectrum ground motion at the site, based on our probabilistic seismic hazard analyses discussed in Appendix C. An anticipated earthquake magnitude (maximum moment magnitude) of 7.26, representing the upper bound M_{max} for a full rupture of the Hayward-Rogers Creek fault, was used in conjunction with the PGA value assigned in the liquefaction analyses. The liquefaction-induced settlements were estimated using the procedure of Tokimatsu and Seed (1984).

The calculations of liquefaction potential and corresponding settlement are included in Appendix D. Our liquefaction analyses show that the materials beneath the proposed structures are generally cohesive and/or dense enough to have a low potential for liquefaction. This low potential for liquefaction in the new borings agrees well with previous liquefaction studies around the CHRCO campus that also found low potential for liquefaction. Our previous liquefaction studies included a detailed study for the western expansion of the hospital (OSHPD Permit Number HS-002307-01). This previous study included four soil borings and extensive lab testing to evaluate the liquefaction potential at the site. This detailed study showed the clayey sand layers to be "non-liquefiable" according to the procedure by Youd et al (2001). Additionally, liquefaction analyses of the borings performed as part of the geotechnical investigation for the medical office and parking structure by Kaldveer (1990) indicate the sand layers are medium dense to dense and liquefaction potential is remote.

Boring B-112, drilled during Fugro's previous investigation performed in February 2008 at the then proposed CUP site, and approximately 10 feet from the new proposed CUP location, encountered a potentially liquefiable layer of soil, consisting of about 8 feet of clayey sand with

gravel, beginning at about 14 feet below grade. This layer had an average SPT N-value of about 17 blows per foot. Laboratory sieve test results indicated measured fines content values of about 14 to 15 percent. Atterberg limits run on the fine-grained portion of these samples behave primarily as clay (not silt) with sufficient plasticity (PI of about 17) to reduce the potential for liquefaction in this layer. Nearby borings within about 50 feet of B-112 show the clayey sand layer to be significantly denser than observed in B-112. The liquefaction potential at the CUP is likely an isolated area within what is typically a dense layer. Denser clayey sand was also encountered in Boring B-115 at a depth of 20 feet below grade. In our opinion, the potential for significant liquefaction in this layer is low. Originally, we estimated that these conditions would give rise to up to 1 inch of liquefaction-induced settlement during a design level earthquake.

However, since the issuance of the OSHPD draft Code Application Notice (CAN) dated March 24, 2008, and released in August, 2008, we now conservatively estimate up to 1.5 inches of liquefaction-induced settlement could occur during a design level earthquake. Because nearby borings do not indicate a potential for liquefaction, the settlement should be treated as a differential settlement of 1.5 inches over a distance of 50 feet. The foundation system for the CUP should be chosen and designed to incorporate the possible liquefaction settlement.

Boring B-113, approximately 20 feet west of the proposed CUP footprint, encountered a similar potentially liquefiable layer consisting of about 6 feet of clayey sand beginning at about 20 feet below grade with an average SPT-N value of about 12 blows per foot. It is assumed that this layer is the same layer encountered in Boring B-112, which is approximately 50 feet north of B-113. Liquefaction analysis indicated that the liquefaction-induced settlement at the B-113 location is similar to that calculated for B-112, confirming the conclusions made from the analysis made for Boring B-112.

5.2.3 Impacts of Liquefaction on Design and Ground Motion

The saturated clayey sand materials with the potential for liquefaction identified above are likely part of paleochannels associated with Temescal Creek. The isolated loose zones are within what are typically dense granular layers with low potential for liquefaction. Lower blow count materials were found in two borings indicating a possibly contiguous liquefaction-susceptible layer (B-112 and B-113). However, lower blow count materials were not encountered in the other borings located within 100 feet (B-114, B-115, PB-1 (97), PB-2 (97), PB-1 (87), PB-3 (94)). The potential for significant liquefaction and large liquefaction-induced settlements or other ground failures at the site is considered low, based on the evaluation of the borings and our review of previous liquefaction studies at this site. The associated potential for modification of ground motion by significant strains in softening liquefied layers is low as extensive liquefaction in continuous layers is not considered likely at the site.

Liquefaction effects may be mitigated by supporting the proposed CUP on a deep foundation system deriving its strength below potentially liquefiable layers. Liquefaction-related damage to exterior flatwork may be expected during the design earthquake, but should be repairable and would likely not have a significant effect on CUP operations.

5.2.4 Dynamic Densification

Volumetric compression, or soil densification, is settlement caused by earthquake-induced ground shaking that causes soil particles to compress together. Dry cohesionless soils, such as sands and gravels, are susceptible to this type of settlement. Earthquake induced settlements of dry cohesionless soils depends on three main factors, including the relative density of the soil, the maximum shear strain induced by the design earthquake and the number of shear strain cycles which can be related to earthquake magnitude.

We used the methodology proposed by Tokimatsu and Seed (1987) to estimate the degree of dynamic densification potential in the unsaturated alluvial clay and clay fill above groundwater at the Children's Hospital site. This methodology is outlined below:

1. Estimate the shear stress in the soil induced by the earthquake (peak horizontal ground acceleration value);
2. Estimate the volumetric strain of the soil based on this shear stress value using the chart developed by Tokimatsu and Seed (1987);
3. Adjust the unidirectional volumetric strain to account for the multidirectional shaking effects of an earthquake;
4. If the moment magnitude earthquake being considered is different than 7.5, adjust the volumetric strain by a ratio value based on the earthquake magnitude; and
5. Based on the thicknesses of the dry cohesionless soil layers and the adjusted volumetric strain values, compute the total predicted settlement.

The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.76g. This PGA represents the design response spectrum ground motion at the site, based on our probabilistic seismic hazard analyses, as discussed in Appendix C. An anticipated earthquake magnitude (maximum moment magnitude) of 7.26, representing the upper bound M_{max} for a full rupture of the Hayward-Rogers Creek fault, was used in conjunction with the PGA value assigned in the dry dynamic densification analyses.

The calculations for the soil dry densification evaluation are included in Appendix D. The soils above groundwater at our site were primarily clay. No dynamic settlement was identified in any of our three borings, based on our evaluation. The potential for dynamic densification of unsaturated soils above groundwater at this site is considered low.

5.2.5 Lateral Spreading

Lateral Spreading occurs where the contact between a layer of liquefiable material and the material below is sloped. Saturated sands lose their strength during an earthquake and become fluid-like and mobile. As a result, the ground may undergo large permanent displacements that can damage underground utilities and well-built surface structures. Lateral spreading involves displacement of large blocks of ground down gentle (greater than 0.1 percent) slopes or towards the open face of slopes such as a stream channel. In areas where there is no open face, buckling of the overburden is often observed.

Historic earthquakes in the Bay Area have produced lateral spreading features. However, large-scale lateral spreading is considered unlikely at this site because the site is essentially level and the probability for liquefaction at the site is considered low, except in localized areas (discussed in Section 5.2.2). In addition, the transitions between cohesive layers and granular layers were observed to be gradual and occur at varying depths. Liquefaction-induced lateral spreading along the contact between cohesive materials and granular materials is unlikely. Liquefaction induced lateral spreading into buried paleochannels is unlikely because the paleochannels are typically dense and have a low potential for liquefaction, as discussed above.

5.2.6 Tsunami and Seiche

During a major earthquake, strong waves such as tsunamis or seiches may be generated in large bodies of water and may cause damage to structures affected by them. A tsunami (or seismic sea wave) is an open ocean phenomenon caused by faulting, volcanism or other abrupt movements on the ocean floor. A seiche is a wave that occurs in an enclosed basin as a result of fault displacement in the basin bottom, large landslides into the basin, or from periodic oscillation or sloshing of the water in the basin in response to ground shaking.

The potential for tsunami and seiche hazards at this site is considered remote. The site is located at an elevation of about 100 feet according to the project survey, and about 1.8 miles east of San Francisco Bay. There are no large enclosed bodies of water in the vicinity of the site. Damaging tsunamis are not common on the California coast. Most California tsunamis are associated with distant earthquakes (often those in Alaska or South America), not with local earthquakes. Devastating tsunamis have not occurred in historic times in the Bay Area. Because of the lack of reliable information about the kind of tsunami run-ups that have occurred in the prehistoric past, there is considerable uncertainty over the extent of tsunami run-up that could occur at the site. However, the Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region (Ritter & Dupre, 1972) does not indicate that the site is in an area that may be inundated by a tsunami with a run-up of 20 feet at the Golden Gate Bridge.

5.2.7 Landsliding

A landslide is a movement of a mass of soil down a steep slope when the soil cannot support the weight of overlying soil or rocks. Landslides vary in size and rate of movement. Slides can occur slowly over time or suddenly. Areas susceptible to landslides are those where masses of soils are weakly supported because of natural erosion, changes in ground water or surface water patterns, or human activities such as undercutting. Heavy rains or seismic shaking can also trigger landslides.

A large earthquake in the San Francisco Bay Region may cause movement of active slides and could trigger new slides. However, no significant slopes are present at the site, and no evidence of previous or active slides were observed during our site investigation. The site is not within a mapped area of existing or potential slope instability. The site is not located within a State of California designated seismically induced landslide hazard zone, as shown on Plate 9 –

CGS Hazard Zone Map. The site is not located within a City of Oakland landslide Hazard Zone (Miles and others, 2001). The potential for landsliding at the site is considered low.

The nearby Highway 24 entrance ramp to the east of the site includes retaining walls and embankment slopes with an approximate inclination of 2.5:1 (horizontal to vertical). This roadway is about 25 to 30 feet higher than the surrounding relatively flat terrain. We have assumed for purposes of this study that the Caltrans structures were engineered and constructed such that a low potential for sliding exists at the slope. An evaluation of the stability of the Caltrans Highway 24 entrance ramp embankment is beyond the scope of work for this study.

5.2.8 Other Shaking Hazards

Ground shaking during an earthquake could cause objects within buildings that are not rigidly attached to the building structure (such as desks and bookshelves) to undergo some differential movements with respect to the structure. Building construction should include designs that reduce such potential differential movements and the adverse effects of such movements where they cannot be prevented.

We have also considered the possibility of the occurrence of other seismic hazards. Differential compaction is considered unlikely because of the abundance of very stiff clayey soils and bedrock and low potential for liquefaction. Ground cracking and lurching can be caused by any of the phenomena discussed above.

5.3 OTHER GEOLOGIC HAZARDS AND COMPLICATING SITE CONDITIONS

5.3.1 Flooding and Inundation Due to Dam or Embankment Failure

The site is located at an elevation of approximately 100 feet (relative to NAVD88) and about 1.5 miles west of San Francisco Bay. The potential for flooding from San Francisco Bay is remote. The site is not within the flood plain of any nearby water body. The National Flood Insurance Program designates flood prone areas. The site is designated as Zone C, an area of minimal flooding (FEMA, 1982). Flooding resulting from ponding of locally heavy rainfall was not included in our evaluation.

Dams and reservoirs, which hold large volumes of water, represent a potential hazard from failure caused by ground shaking. The site is in the path of inundation from failure of Lake Temescal dam (owned by the East Bay Regional Park District). The state's Division of Safety of Dams last inspected this dam in July 2002. At the time, no issues necessitating corrective action were presented and the dam was "judged satisfactory for continued operation" (City of Oakland General Plan, 2004).

5.3.2 Debris Flow

The project area is about 2 mile from the nearest zone designated as a debris-flow source area (Oakland Hills) in the Principal Debris-Flow Sources Areas in Alameda County map (USGS, 1997), and is therefore not likely to be impacted by debris flows.

5.3.3 Land Subsidence

Land subsidence is a gradual settling or sudden sinking of the Earth's surface caused by subsurface movement of earth materials. The principal causes of subsidence are aquifer-system compaction, drainage of organic soils, underground mining, hydrocompaction, sinkholes, and thawing permafrost. The conditions needed for these hazards are not known to exist within the project area.

5.3.4 Soil Expansion

The near surface clay fill soils observed at the proposed CUP location have Plasticity Indices (PI) ranging from about 17 to 26 percent, based on laboratory testing from current and previous borings at the site. These PI values indicate a moderate to high expansion potential. In addition, an expansion index test was performed as part of the previous study for the adjacent office building and parking structure (Kaldveer, 1990). This test showed an Expansion Index of 115 in the fill material, indicating a high expansion potential. These results suggest that the clayey soils can be classified as "expansive" as defined in Section 1802A.3.2 of the 2007 California Building Code.

The moderate to high expansion potential of the clayey surface soils encountered at the site is a primary consideration for foundation and floor slab design. These materials could be subjected to volume changes during seasonal fluctuations in moisture content. However, it is anticipated that the CUP first floor slab will be completely supported by the deep foundation system, and imported non-expansive fill is not necessary since reinforcement will provided in the slab to minimize the impact of expansion pressures.

Expansive materials surrounding the structures may be subjected to volume changes during seasonal fluctuations in moisture content, which could affect minor slabs-on-grade and landscaping hardscape such as sidewalks and exterior slabs.

5.3.5 Corrosion Potential

Corrosion testing in the vicinity of the Proposed CUP was conducted as part of the New Hospital Replacement Project Investigation completed by Fugro in May 2008 on a sample of the surficial material from B-112 at a depth of 4 feet. The corrosion testing was conducted at CERCO Analytical Laboratory in Pleasanton, California. The corrosivity testing included resistivity analysis, chloride ion concentration, sulfate ion concentration, pH testing, and redox potentials. The corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. The results of the laboratory tests and details of the results are presented in the letter reports provided by CERCO Analytical, Inc., attached in Appendix B.

The results of our corrosion tests generally show slightly more corrosive soils than identified in previous corrosion studies performed at the nearby hospital office and parking structure site by Kaldveer (1990). The Kaldveer study included two resistivity traverses and one sulfate content test. The historic resistivity tests indicated resistivity values ranging from 3,066 to 11,975 ohm-cm; indicating slightly to moderately corrosive conditions. The sulfate test indicated a sulfate content of 0.005 percent; suggesting negligible sulfate attack potential.

Tests for Buried Concrete

Soluble sulfate concentration, chloride ion concentration, and pH tests were performed on the samples from B-112 completed in February 2008. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures. The table below presents the depth at which the samples were collected and the laboratory test results.

Table 3. Summary of Corrosion Test Results for Buried Concrete

Test No.	Boring	Depth	Sulfate (mg/kg)	Chloride (mg/kg)	pH
1	B-112	4 ft	33	N.D.	5.6

N.D. = none detected

CERCO's evaluation detected a sulfate ion concentration in the tested sample of 33 mg/kg. The sulfate concentrations are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel. Section 1904.3 of the 2007 California Building Code requires that concrete exposed to sulfate containing soils comply with the provisions of ACI 318, Section 4.3. According to ACI 318 sulfate exposure may be considered negligible if water-soluble sulfate in soil is less than 0.1 percent by dry weight. The water-soluble sulfate was measured to be about 33 mg/kg (ppm) or less (0.00033 percent by dry weight). Therefore, the test results suggest Type II Portland cement is suitable for onsite below-grade concrete structures.

CERCO's evaluation detected the chloride ion concentration in the tested sample ranging to be none detected (N.D.). The chloride concentrations are determined to be insufficient to attack steel embedded in a concrete mortar coating. Section 1904.4 of the 2007 California Building Code requires that reinforcement in concrete be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.4. Table 4.4.1 of ACI 318 suggests commencing mitigative measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06 percent by dry weight. The chloride content measured in the soil sample was negligible, which does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures.

CERCO's evaluation indicates the pH of the soil tested was 5.6, which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Soils with a pH less than 6.0 are considered corrosive, and corrosion prevention measures need to be considered for structures placed in this acidic soil.

For specific long-term corrosion control design recommendations, we recommend that a corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures and steel pipe coated with cement-mortar.

Tests for Buried Ferrous Metals

In addition to the pH tests described above, the resistivity and redox tests were performed on the soil samples discussed above. These tests may be evaluated together to provide an indication of the corrosion potential of the soil environment on buried ferrous metals such as steel or cast-iron pipes. The results of these tests are presented in the table below.

Table 4. Summary of Corrosion Test Results for Buried Ferrous Metal

Test No.	Boring	Depth	Resistivity (ohm-cm)	Redox Potential (mV)	pH
1	B-112	4 ft	1,800	410	5.6

ND = none detected

CERCO classified the soil samples as "corrosive" based on the resistivity measurements. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron pipeline should be protected against corrosion as well.

The measured soil reduction-oxidation (redox) potentials were measured to be 410 to mV and is considered indicative of aerobic soil conditions.

Appendix B presents the complete results of corrosion tests and recommendations provided by CERCO. For specific long-term corrosion control design recommendations, we recommend that a corrosion engineer evaluate the corrosion potential of the soil environment on buried metal.

5.3.6 Land Subsidence

Land subsidence is a gradual settling or sudden sinking of the Earth's surface owing to subsurface movement of earth materials. The principal causes are aquifer-system compaction, drainage of organic soils, underground mining, hydrocompaction, sinkholes, and thawing permafrost. The conditions needed for these hazards are not known to exist within the project area.

5.3.7 Naturally Occurring Asbestos (NOA)

Inhalation of asbestos fibers may cause cancer. Most commonly, asbestos occurrences are associated with serpentinite and partially serpentinized ultramafic rocks. Asbestos occurs naturally in certain geologic settings in California. Exposure and disturbance of rock and soil that contains asbestos can result in the release of fibers to the air and consequent exposure to the public. Asbestos most commonly occurs in ultramafic rock that has undergone partial or complete alteration to serpentine rock (proper rock name serpentinite) and often contains chrysotile asbestos. In addition, tremolite, another form of asbestos, can be found associated

with ultramafic rock, particularly near faults. Sources of asbestos emissions include: unpaved roads or driveways surfaced with ultramafic rock, construction activities in ultramafic rock deposits, or rock quarrying activities where ultramafic rock is present.

The bedrock underlying the site has the potential of containing asbestos fibers; however, bedrock is anticipated to be at a depth of about 300 feet below grade and construction activities should not penetrate through the Quaternary alluvium overburden. The potential for NOA hazards at the site is low.

5.3.8 Radon-222

Radon gas is a naturally occurring radioactive gas that is invisible and odorless. Radon forms from the radioactive decay of small amounts of uranium and thorium naturally present in rocks and soils. Some rock types, such as black shales and certain igneous rocks, can have uranium and thorium in amounts higher than is typical for the earth's crust. Increased amounts of radon will be generated in the subsurface at locations where these rocks are found. Because radon is a gas, it can easily move through soil and cracks in building slabs or basement walls and concentrate in a building's indoor air. Areas with higher amounts of radon in the underlying rocks and soil are likely to have higher percentages of buildings with indoor radon levels in excess of U.S. Environmental Protection Agency guidelines, and incidences of very high indoor radon levels are more likely in these areas. Long term exposures to elevated levels of radon can increase the risk of lung cancer.

In California, the Department of Public Health (CDPH), Radon Program, collects radon test data for buildings throughout the state. The data are maintained in a digital database. The database is used to help CDPH determine areas with excessive indoor radon levels, determine areas that may need testing, and inform the public of the results. This database is organized by zip code, and is **not** to be considered a site specific investigation. The United States Environmental Protection Agency (EPA) recommends that individuals avoid long-term exposures to radon concentrations above 4 picocuries per liter (pCi/L). The Oakland zip code 94118 has had three tests conducted, none returning with levels exceeding 4 pCi/L.

5.4 CONCLUSIONS - GEOLOGIC HAZARDS

The geologic hazards at the Children's Hospital site appear to include the moderate to high expansion potential of the clay soils, a corrosive soil environment, the potential for moderate to strong ground shaking and localized areas of potential liquefaction based on the results of our review and evaluation. Details regarding specific geologic hazards are presented above. Practically all structures within the San Francisco Bay Area will experience similar shaking effects during a moderate to strong earthquake.

5.5 OTHER HAZARDS AND COMPLICATED SITE CONDITIONS NOT ADDRESSED IN THIS REPORT

Under the "Exceptional Geologic Hazards and Complicated Site Conditions" of CGS Note 48 are a number of exceptional items not typically applicable statewide including:

- Phase I & II Environmental Site Assessment;
- Hazardous Materials;
- California Environmental Quality Act;
- Groundwater Quality;
- Onsite Septic Systems; and
- Non Tectonic Faulting and Hydrocollapse of Alluvial Fan Soils;

These exceptional items have not been evaluated in detail as part of this report, and are considered outside of the current scope of Fugro's work.

6.0 DISCUSSIONS AND CONCLUSIONS

We believe that redevelopment of the Children's Hospital is feasible from a geotechnical and geological standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections:

6.1 EXPANSIVE SOILS

As discussed in Section 5.3.4, the moderate to high expansion potential of the clayey surface soils encountered onsite is a primary consideration for foundation design. These materials could be subjected to volume changes during seasonal fluctuations in moisture content. With the understanding that the CUP's first floor will be below grade, will be a structural slab, and will be supported entirely on a deep foundation system, the potential impact of expansive soils on the interior slab is somewhat lessened. Note that special design considerations will apply for the design of exterior slabs, as discussed in Section 7.5.2 - Exterior Slabs on Grade.

Additional requirements for interior slabs-on-grade and moisture protection are discussed in Section 7.7.1 of this report.

6.2 UNDOCUMENTED FILL

Our subsurface investigation indicates that about 4 to 8 feet of clay fill with varying amounts of sand and gravel exists at the proposed development location. This information agrees well with the historic borings drilled at this site that indicate a fill layer. Because this is an undocumented fill, the consistency and density of the material cannot be relied upon for full foundation support. Variations in consistency and density could result in differential settlement of structures founded on this material. Therefore, reduced lateral and axial capacity values will apply for deep foundations making use of these soils. Additional recommendations for construction of exterior flatwork and slabs on-grade are presented below.

6.3 GROUNDWATER

Based on our recent borings and available historical groundwater data, groundwater was encountered at depths as shallow as 7.5 feet below grade. We anticipate that perched or free groundwater may be encountered in the excavation for the CUP, and during the installation of a deep foundation system. Installation of deep foundations will likely require dewatering via sump, or hole casing or stabilization methods to prevent caving. Construction during the dry season would reduce the potential for encountering groundwater in excavations for the CUP floor slab, or for surface water to pond in the building pad excavation. For design calculations at the CUP location, we recommend that 8 feet below grade be considered the design depth to water.

Additional care should be taken to avoid the buildup of water behind retaining walls at the CUP site. Seeping surface water, or sheeting water down the slope of the Highway 24 berm should be considered in the design of retaining wall drainage where CUP walls will retain the berm.

6.4 FOUNDATION SUPPORT

Suitable foundation options for the proposed CUP are discussed below. Our design recommendations for support of the CUP are discussed in Section 7.

We recommend that the new CUP be supported on a deep foundation system because of the potentially liquefiable soils observed at the proposed CUP location. In addition, the deep foundation should support structural column loads and the first floor slab to avoid the stresses resulting from total and differential settlement caused by liquefaction. Based on the restrictions of the noise and vibration on the operations of the existing hospital, a driven pile foundation is not considered to be a feasible alternative. Therefore, the CUP should be supported on a drilled pier foundation according to the recommendations presented below.

6.5 CONSTRUCTION CONSIDERATIONS

6.5.1 Existing Structures and Foundations

Several existing structures, associated foundation systems and buried utilities exist at the proposed development site and will likely require demolition as part of the construction. The contractor should be aware that buried obstructions are likely to be encountered during construction of the proposed CUP. The proposed CUP site is partially located in the footprint of several temporary buildings placed on top of the existing asphalt parking surface. It is not anticipated that these structures feature any below ground foundations.

The eastern wall of the proposed CUP structure may also be immediately adjacent to the embankment that exists for the on-ramp for Highway 24. It is possible that this eastern wall will serve as a retaining wall that supports the current embankment. Since this embankment exists in the Caltrans right-of-way, it must be protected during construction with appropriate shoring measures to allow for continued freeway operation. Construction of the proposed CUP structure into the Caltrans embankment must be proposed and carried out with Caltrans

permission and regulations. The eastern wall should be considered as a restrained wall and should be designed in accordance with Section 7.8.1 of this report.

6.5.2 Excavation Shoring and Dewatering

Excavations will be required for below grade portions of the structures, to install utilities, and to remove locally weak or unsuitable soils, if encountered. Based on the currently proposed location of the CUP, excavations will likely extend below the grade of the adjacent Highway 24 ramp; this excavation will require shoring rigid enough to avoid damage to the onramp. Underpinning of neighboring structures may not be needed; however, shoring will likely needed to limit the size of the excavation because of the depths of excavation required and spatial constraints. The shoring system must be designed to resist loads from foundations of neighboring structures or active pavement or parking areas. All excavations that will be deeper than 5 feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards.

For the purposed of shoring designs, site soils should be considered Type C soils in accordance with the OSHA Technical Manual.

Depending on the time of year of construction, dewatering may be required to remove rainwater during construction. The performance of the shoring and dewatering systems are highly dependent on the construction methods and procedures employed. The design of the necessary shoring and dewatering systems, as well as the protection of existing facilities, site improvements and utilities should be the responsibility of the contractor. The project geotechnical and structural engineers should review the proposed shoring system prior to installation.

6.5.3 Earthwork

If earthwork is performed during the dry season, moisture conditioning will be required to raise the in-situ moisture contents to near optimum moisture content (per ASTM D1557). If earthwork is performed during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

6.5.4 Construction Quality Control

Careful quality control should be maintained during construction so that the recommendations presented in this report are achieved. Construction observation is necessary to check that the quality control measures are being properly implemented.

7.0 RECOMMENDATIONS

The recommendations presented in this report should be incorporated in the design and construction of the project to help minimize soil or foundation related problems. Detailed

earthwork and foundation recommendations for use in design and construction of the project are presented below. We recommend that our firm review the final design and specifications to check that the earthwork and foundation recommendations presented in this report have been properly interpreted and implemented. Fugro can assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications.

7.1 SEISMIC DESIGN CRITERIA

7.1.1 Seismic Parameters

The proposed structure should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practice. This section presents seismic design parameters for use with the 2007 California Building Code (CBC). The CBC is based on the 2006 International Building Code (IBC). The CBC refers to the design code by American Society of Civil Engineers (ASCE 7-05) for the development of site-specific response spectra. Because of the critical nature of the structures, site-specific response spectra were developed in accordance with guidelines mentioned in Section 21 of ASCE 7-05. The values of S_{DS} and S_{D1} from of our site-specific analyses are included in Table 5, below.

Table 5. 2007 CBC Seismic Parameters

Parameter/Coefficient Description	2007 CBC Reference	Parameter/ Coefficient	Value
MCE for 0.2 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(3)	S_s	1.73
MCE for 1.0 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(4)	S_1	0.64
Long-Period Transition Period	ASCE 7-05 Figure 22-15	T_L	8
Soil Profile Type	Table 1613A.5.2	Site Class	D
Site Coefficient	Table 1613A.5.3(1)	F_a	1.0
Site Coefficient	Table 1613A.5.3(2)	F_v	1.5
Code Based Spectral Acceleration Parameters NOT FOR CUP STRUCTURAL DESIGN	Equation 16-39	S_{DS}	1.16
	Equation 16-40	S_{D1}	0.64

**Site Located at: Latitude: 37.8380, Longitude: -122.2665

7.1.2 Site Class, Seismic Design Parameters, and Seismic Design Category

The above seismic parameters for the simplified method are presented indicating a Soil Profile Type, Site Class D based on an average site shear wave velocity (v_{s_30}) of 270 meters per second (discussed in detail in Appendix C). Site Class has been determined based on the referenced 2007 CBC Table 1613A.5.2. We note that the discussion of liquefaction potential above indicates the probable effects of liquefaction are confined to an isolated area of the site. Additionally, we recommend that the proposed CUP structure be constructed on a deep foundation system bearing in non-liquefiable soils. Based on the isolated area of potential

liquefaction, its minimal impact on the support of the proposed structure, and the minimal impact of liquefaction on propagating ground motions affecting the structure, we have selected Site Class D to calculate the seismic parameters presented above. We note that the code-based simplified method presented in ASCE 7-05 Chapter 11 provides a lower-bound check for site-specific analysis conducted according to ASCE 7-05 Chapter 21.

Table 6. Site-Specific Seismic Design Parameters

Site-Specific Design Spectral Acceleration Parameters	Section 21, ASCE 7-05	**S_{DS}**	**1.178**
	Section 21, ASCE 7-05	**S_{D1}**	**1.140**

The new CUP is assumed to be an Occupancy Category IV Structure according to the 2007 CBC, and should be assigned to Seismic Design Category "D", based on an S_1 value presented above of 0.64 and according to Section 1613A.5.6 of the 2007 CBC.

7.2 SITE PREPARATION AND GRADING

7.2.1 Site Preparation

The site should be cleared of all obstructions, including concrete, asphalt pavement, slabs, fences, utility lines, trees and associated root systems, and debris. Removed concrete, asphalt concrete, and baserock may be reused as fill, provided the material is broken up to meet the requirements in Section 7.2.4 - Engineered Fill Materials. Holes resulting from the removal of root systems of larger trees could extend to depths of 3 feet, and laterally to the drip line of each tree. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with suitable material compacted to the requirements in Section 7.2.5 - Fill Placement and Compaction. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

After clearing, the portions of the site containing surface vegetation or organic laden topsoil should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by the Geotechnical Engineer at the time of construction. Stripped materials should be removed from the site, or stockpiled for later use in landscaping, if approved by the owner.

7.2.2 Subgrade Preparation

Following excavation to the required grades, soil subgrades in areas to receive engineered fill (as defined in Section 7.2.4) or slabs-on-grade should be scarified to a depth of at least 6 inches, moisture conditioned to at least slightly above optimum moisture content, and compacted to at least 95 percent of the soil's maximum dry density. The top 6 inches of subgrade in areas to receive pavements should be moisture conditioned and compacted to at least 95 percent of the soil's maximum dry density. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades

should be kept moist during construction. If the subgrade is allowed to become dry, it should be moisture conditioned to eliminate shrinkage cracks.

In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry, or that scarification and aeration be performed in any soils that are too wet. There was no indication of weak or unstable soils in our borings; however, after the removal of existing buildings and pavements, the exposed subgrade materials may be above their optimum moisture content, and may be locally unstable. If areas of unstable soils are encountered at the time of construction, the geotechnical engineer should review conditions in the field and provide recommendations for stabilization procedures.

7.2.3 Reworking Undocumented Fill

As previously discussed, about 4 to 8 feet of undocumented fill was observed in our borings at the site. Prior to constructing a building pad, the undocumented fill material should be scarified to a depth of 12 inches, moisture conditioned to slightly above optimum moisture content, and compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM D1557 (modified proctor test.) Excavated material may be used as engineered fill elsewhere on the site, provided the material is placed in accordance with the requirements of Section 7.2.5 – Fill Placement and Compaction.

7.2.4 Engineered Fill Materials

All fill placed at the site should consist of engineered fill meeting the requirements presented in this report, except for landscaping materials placed on level ground. On-site soil below the stripped layer and having an organic content of less than 3 percent by volume can be used as fill except where "non-expansive" import is required beneath slabs-on-grade. All engineered fill placed at the site, including on-site soils, should not contain rocks or lumps larger than 4 inches in greatest dimension and contain no more than 15 percent larger than 2.5 inches.

"Non-expansive" fill should be predominantly granular, have an organic content of less than 3 percent by volume, should have a liquid limit less than 40 percent, have a plasticity index not exceeding 15, and should contain no environmental contaminants or debris. All imported fill should consist of "non-expansive" fill.

7.2.5 Fill Placement and Compaction

All engineered fill should be compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM Designation D1557 (latest edition). The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent of the soil's maximum dry density. Fill material should be spread and compacted in lifts not exceeding 8 inches in pre-compacted thickness. The moisture content of the natural on-site expansive clayey soils reused as fill should be 2 to 3 percent above the optimum moisture content for the soil at the time of compaction. In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may

require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet.

7.2.6 Trench Backfill

Pipeline trenches should be backfilled with materials satisfying the criteria described above for fill, placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts may be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Onsite soil used for trench backfill should be compacted to at least 90 percent of the soil's maximum dry density by mechanical means only (jetting should not be permitted). Sand can be used for trench backfill if the sand is compacted to at least 95 percent of the soil's maximum dry density and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill below slab and pavements should be compacted to at least 95 percent of the soil's maximum dry density.

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts can be used, provided the method of compaction is approved by the Geotechnical Engineer, and the required minimum degree of compaction is achieved.

Where utility trenches backfilled with sand enter building areas, they should be backfilled by an impermeable soil plug that is at least 2-feet thick. The plug should be located at the perimeter of the foundation. The plug should ideally be keyed at least 12 inches into the sides and bottom of the trench and should extend to within 12 inches of the finished grade. This should help to minimize moisture change in the moderately to highly expansive clays beneath the slabs. Where sand backfilled utility trenches cross planter areas and pass below pavements or concrete sidewalks, they should be plugged as described above to minimize soil volume change below asphalt and concrete areas.

7.2.7 Surface Drainage

Positive surface gradients should be provided adjacent to the CUP to direct surface water away from foundations and slabs toward suitable discharge facilities. Water from roof downspouts should be carried away from the buildings in solid pipes and discharged into suitable drainage facilities. Surface water should not be allowed to pond adjacent to the structures or on pavements.

7.3 TEMPORARY SHORING AND CONSTRUCTION SLOPES

7.3.1 General

Construction may require either temporary slopes or shoring for excavations as needed. If site constraints are not present, temporary construction slopes may be used. However, where site constraints are present, or at the contractor's option, temporary shoring should be used. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary construction slopes and shoring system. In selecting the temporary slope, the

contractor should evaluate the potential of shallow soil movement, which may potentially jeopardize the existing nearby facilities.

7.3.2 Temporary Construction Slopes

If temporary slopes are used, the contractor should be aware that in no case should slope height, inclination, and excavation depths exceed those specified in local, state, or federal safety regulations. Specifically, one needs to be aware of the current OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P. In areas next to adjacent existing foundations, we recommend that open excavations not be performed within 1.5:1 (horizontal: vertical) slope extended downward from the bottom edge of adjacent foundations.

Fugro recommends that the contractor, or his specialty subcontractor, design temporary construction slopes to conform to the OSHA "Guidelines for Excavations and Temporary Sloping." The contractor, or responsible subcontractor, should determine temporary slope inclinations based on the subsurface conditions exposed at the time of construction.

For planning purposes, the onsite soils should be categorized as OSHA Type C with temporary slope inclinations of no steeper than 1.5:1 (horizontal to vertical) for excavations less than 20 feet deep.

If temporary slopes are left open for extended periods of time, exposure to weathering and rain could have detrimental effects such as sloughing and erosion. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes and that the temporary slopes be protected from excessive drying or saturation during construction.

7.3.3 Temporary Shoring

If open-excavation is not possible because of site constraint and proximity of existing or new buildings or streets, we recommend that a temporary shoring system be used during any needed excavations. Shoring will be required adjacent to the Highway 24 onramp. Such shoring should be sufficiently rigid to minimize damage to the adjacent facilities (streets, utilities, neighboring buildings, etc.) caused by deflection of the sides of the excavation. The temporary shoring system must be designed to include loads from foundations of neighboring structures.

Fugro recommends that the contractor, or his specialty subcontractor, design and install the temporary shoring. We recommend that the geotechnical and structural engineers review the final temporary shoring plan to ensure compliance with the anticipated soil conditions encountered at the site. In addition, we recommend that a geotechnical engineer's representative observe the installation of the temporary shoring systems. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary shoring system.

7.3.4 Drilled Pier Foundations

Drilled pier foundations should be used to support the CUP. The drilled piers should be cast-in-place, straight piers that develop their load carrying capacity through skin friction in the stiff clay soils and interlayered clayey sands and sandy clays. Piers located in a group should have a minimum center-to-center spacing of three times the shaft diameter. The minimum pier diameter should be 18 inches. Pier reinforcing should be based on structural requirements. For piers to be used at the CUP location, they must include downdrag loads, as discussed below.

The design depth of the piers should be determined using an ultimate skin friction of 1,400 pounds per square foot (psf) in the native, upper stiff clay and the interlayered clayey sand and sandy clay to a depth of 50 feet. An ultimate skin friction value of 2,000 psf may be used in the very stiff to hard clay below a depth of 50 feet. We recommend a minimum factor of safety of two be used for design of friction piers. Reductions in capacity in the near-surface undocumented fill, and additional loads caused by liquefaction-induced settlement (downdrag) should be considered in determining the pier length. The elevations below have been calculated assuming a site elevation of approximately +103 feet.

Table 7. Drilled Pier Design Skin Friction

Depth Below Ground Surface (feet NAVD88)	Soil Type	Ultimate Skin Friction (psf)	Allowable Skin Friction (psf)
+95 to +103	Undocumented Fill	0	0
+83 to +95	Stiff Clay and Interlayered Clayey Sand and Sandy Clay	-700 Downdrag	--
+53 to +83	Stiff Clay and Interlayered Clayey Sand and Sandy Clay	1,400	700
Below Elevation +53	Very Stiff to Hard Clay and Dense Granular Lenses	2,000	1,000

Our recommended allowable skin friction values may be increased by one-third for all load combinations that include wind or seismic loads. Uplift loads may be resisted using 80 percent of the allowable skin friction values. Downdrag friction should not be used to counter uplift forces.

Static total and differential settlements of the CUP supported on drilled piers should not be significant (i.e., less than ½ inch) and within tolerable limits for the proposed structure.

Drilled Pier Downdrag

Downdrag occurs where soils settle around a relatively rigid deep foundation system and pull the pier downward. The piers must be designed to include the downdrag load that could be imposed on the piers by post liquefaction settlement of the surrounding soil during a major seismic event. As the surrounding soils settle with respect to the piers, the side friction force in the upper zone acts downward instead of upward and becomes a load instead of a resistance.

Borings B-112 and B-113 at the CUP location indicate the top of a liquefiable layer to be between Elevation 89 and Elevation 83. For design purposes, we recommend applying an ultimate negative skin friction of 700 psf to the portion of the pier above Elevation 83. The liquefiable sand layer is not expected to contribute significantly to the downdrag load, and the potential effect has been considered in developing our recommended average skin friction design value between Elevations 53 and 83.

Negative skin friction should be applied to the pier based on final design grade, not existing site grade. We recommend negative skin friction be subtracted from the ultimate capacity of the pier, not the allowable capacity, as shown below:

$$\frac{Q_{ULTIMATE} - NegativeSkinFriction}{Factor\ of\ \ Safety = 2} = Q_{ALLOWABLE\ (dead+live)}$$

Drilled Pier Construction

Pier excavations extending below a depth of about 8 feet could encounter groundwater and may require special considerations during construction. Excavations may need to be cased, or a polymer drill slurry used to protect the holes from caving. All pier excavations adjacent to existing structures should be cased to prevent caving of the hole caused by nearby footing loads. In addition, we recommend that concrete be placed in the bottom of the drilled holes by the tremie method. Concrete should be designed with a high slump, equal to or greater than 6 inches, to facilitate construction and reduce the potential for free water to be trapped in the pier excavation. Concrete should be placed in all piers the same day that the excavations are completed. Additionally, in locations where multiple piers will be required connected by pier caps, alternating piers should be drilled and poured separately to reduce the potential for caving during excavation. For example, in a four pier cap, two diagonal piers should be excavated and poured prior to the excavation of the remaining two piers.

We recommend that the excavation of all piers be performed under the direct observation of the geotechnical engineer to confirm that the pier foundation is constructed in general accordance with the requirements presented above.

Pier Caps and Grade Beams

Pier caps and grade beams should be used to tie together the foundation piers. Pier caps and grade beams within 3 feet of the ground surface should be designed to resist an uplift pressure of 2,500 pounds per square foot (from highly expansive soils), with uplift resistance provided by the drilled piers. Pier caps and grade beams beneath 3 feet below adjacent soil grade can be considered in a zone of constant moisture the proposed basement do not need to be designed for expansion pressures, provided the basement floor elevation is at least 5 feet below surrounding site grades (a zone of constant moisture and reduced potential for expansion). Grade beam reinforcement should be designed to meet structural requirements.

7.3.5 Adjacent Existing Foundations

The proposed CUP will be constructed within close proximity of the existing helistop structure. The pier foundations should be designed to maintain a minimum distance of three pier diameters from adjacent existing piers. The CUP piers are expected to bear significantly deeper than the existing piers due to additional seismic concerns related to Occupancy Category IV structures. Thus the proposed foundations are expected to have minimal impact on support or settlement of the existing helistop provided adequate horizontal spacing is maintained.

7.4 LATERAL LOADS

7.4.1 Long-Term Lateral Loads

Long-term resistance to lateral loads may be provided by pier caps, and grade beams. A long-term passive pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for lateral load resistance against the face of drilled pier caps, or grade beams perpendicular to the direction of loading where the foundation is poured neat against engineered fill or undisturbed native material. However, a reduced passive pressure of 150 pcf should be used on any portion of the foundation within the undocumented fill layer (i.e., the upper 8 feet below existing grade).

For additional lateral capacity of retaining structures (for instance, the east wall of the CUP), a shear key may be added below the wall, constructed as a grade beam between drilled pier caps. Lateral resistance capacity of the grade beam may be calculated using the side area of the beam in contact with soil (e.g., not supported by drilled piers) and an allowable equivalent fluid pressure of 300 pcf.

Note that lateral deflections corresponding to a Δ/H of 0.04 (where Δ equals horizontal deflection and H equals the height of the footing or structural slab) are required to mobilize full passive resistance. Compatible deflection-resistance relationships must be considered when evaluating the relative contributions of passive resistance from foundation elements. The table below presents the recommended load-deflection curves to mobilize the passive resistances for different lateral deflections.

Table 8. Passive Soil Resistance (P) versus Displacement (Δ) Relationship

Deflection Ratio (Δ/H)	Passive Soil Resistance Ratio (P/P_{max})
0.00	0.0
0.01	0.5
0.04	1.0
1.00	1.0

7.4.2 Temporary Seismic Lateral Loads

For temporary dynamic loads, a maximum passive resistance of 1,500 psf can be used instead of the passive pressures given above. Passive resistance in the upper 1 foot should be ignored unless the soil is confined by a pavement or slab.

For lateral resistance to seismic loads, friction along the sides of the grade beam, cap, or below grade retaining wall parallel to the direction of the imposed lateral force may be combined with passive resistance. An allowable uniform frictional resistance of 300 psf, corresponding to a lateral displacement of about ½ inch, may be used to resist lateral seismic loads. *However, a reduced frictional resistance of 150 psf should be used on any portion of the grade beams retaining walls, or pier caps within the undocumented fill layer (upper 8 feet).* Frictional resistance should only be considered applicable to the sides of the pier cap and grade beam parallel to the direction of the imposed lateral seismic force.

7.4.3 Lateral Loads on Drilled Piers

We have calculated the lateral load resistance for drilled piers using the liquefied soil cases. Our calculations were based on an assumed maximum lateral displacement of 0.5 inches at the pier head, modeled at the ground surface. Displacement, Moment, and Shear diagrams for the piers are presented on Plates 10a, 10b, and 10c, respectively. Table 8 presents the maximum shear (lateral capacity) and moment values for the piers, assuming 500 kips axial compression load.

Table 9. Lateral Capacity of Drilled Piers

Load Case	Maximum Shear Capacity	Maximum Moment	Depth to Fixity
Free-Head, 0.5" Displacement	12.5 kips	955 kip-inches	15 feet
Fixed-Head, 0.5" Displacement	28.0 kips	2,570 kip-inches	24 feet

7.5 WATERPROOFING AND DESIGNING FOR HYDROSTATIC PRESSURES

The below-grade walls and slab at the proposed CUP structure should be waterproofed. If the structure extends more than 8 feet below existing grade, the structural slab should be designed to resist hydrostatic uplift pressures. Appropriate slab reinforcement should be provided in accordance with the anticipated loading of the slab, including hydrostatic uplift pressures. We recommend that hydrostatic pressure be calculated assuming a design groundwater depth of 8 feet below existing grade. Hydrostatic uplift pressure can be resisted by the dead load of the structure or by drilled piers. Some efflorescence should be expected through below grade concrete walls or floors that are left unprotected from ambient moisture.

7.6 CONCRETE SLABS-ON-GRADE

7.6.1 Interior Slabs-on-Grade

We recommend that the interior structural slab be totally supported on the drilled piers, caps and grade beams. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support during construction.

The structural slab, if located above 8 feet below site grade, should include at least one pipe drain to remove water trapped beneath the slab over the lifetime of the structure. If the CUP structural slab is located below 8 feet below grade, it should be designed for hydrostatic uplift pressure.

7.6.2 Exterior Slabs-on-Grade

As previously discussed, the onsite moderate to highly expansive surface soils could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs, sidewalks, and pavements should be anticipated. This movement could result in damage to the slabs, sidewalks, and pavements that might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

Exterior slabs such as sidewalks could be reinforced with steel reinforcing bars in lieu of wire mesh to minimize the impact of expansion pressures.

Walkways and pavement curbs and gutters should be supported directly on properly prepared native soils. Curbs should extend to the bottom of the pavement and baserock layer. One to two days prior to placing concrete, subgrade soils should be soaked to increase their moisture content to 3 to 5 percent above laboratory optimum moisture (ASTM D1557). The water content of subgrade soils should be verified by field-testing by the Geotechnical Engineer prior to placing concrete.

To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and/or a "drip" irrigation watering system be used. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the tree from slabs or pavements to reduce the effects of tree roots on these improvements.

7.7 RETAINING/BELOW GRADE WALLS

Below grade walls must be designed to resist lateral earth pressures and any additional lateral loads caused by surcharging or seismic pressure. If the walls are designed to resist lateral loads through passive pressure, we recommend they be designed for the equivalent fluid pressures discussed in Section 7.4 – *Lateral Loads*. If below grade walls are not designed to resist passive lateral loads, they should be designed as discussed below.

The following recommendations were developed based on the clayey soils encountered at the proposed CUP location. Retaining wall backfill less than 5 feet deep should be compacted using light compaction equipment to at least 90 percent of the soil's maximum dry density. Backfill greater than 5 feet deep should be entirely compacted to at least 95 percent of the soil's maximum dry density. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

7.7.1 Restrained Walls

Restrained walls (walls that are prevented from moving/rotating) should be designed for at-rest earth pressure. We recommend restrained walls be designed to resist an equivalent fluid pressure of 60 pounds per cubic foot (pcf). These equivalent fluid pressures are based on:

- Walls backfilled with onsite, reworked lean clay fill compacted to the requirements in this report;
- The walls are prevented from moving/rotating such that at-rest pressure conditions develop; and
- Level backfill. Walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.
- Appropriate subdrainage is placed to avoid the buildup of hydrostatic pressure.

For seismic loading conditions only, we recommend that soil pressures be reduced to the active (unrestrained) condition plus the seismic increment. For the seismic case, restrained walls should be designed to resist an equivalent fluid pressure of 40 pcf for soil loading, plus a uniform seismic pressure of 38*H psf, where H is the height of backfill above the top of the wall footing, in feet.

Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.50 times the anticipated surcharge load. The design surcharge should include the anticipated surcharge caused by vehicular traffic and construction equipment. Surcharge loads from adjacent structures need to be considered if the proposed walls extend below the zone of influence of adjacent foundations. The zone of influence of adjacent foundations can be defined as the area below an imaginary 1.5:1 (horizontal to vertical) line extending downward from the bottom of footings near the wall.

Recommended static design lateral pressures for restrained walls are presented on Plate 11a. Recommended seismic design lateral pressures for restrained walls are presented on Plate 11b.

7.8 PAVEMENTS

We have developed the following pavement design recommendations based on an R-value of 5, based on the moderate to high expansion potential of the clayey soils at the site

and R-value tests performed on bulk samples collected during Fugro's previous investigation in May 2008. We developed the following alternative preliminary pavement sections based on Chapter 630 of the State of California Department of Transportation Highway Design Manual and assumed traffic indices. Pavement designs for pavement life of 11 to 20 years are presented below.

Table 10. Recommended Pavement Design Alternatives

Location	Anticipated Pavement Life (years)	Pavement Components, R-value = 5		Total Thickness (inches)
		Asphalt Concrete (inches)	*Caltrans Class 2 Aggregate Base (inches)*	
Automobile Parking (T.I. = 4.5)	20	2.5	9.0	11.5
Automobile Access (T.I. = 5.5)	20	3.0	12.0	15.0
Heavy Truck Access (T.I. = 6.5)	20	3.5	15.0	18.5

The traffic indices used in our design were established assuming a typical mix of automobile and "delivery or garbage" truck type of use in the proposed development once construction has been completed. As an alternative to asphalt pavements in areas where heavy trucks may make tight turns (such as trash enclosures or delivery aprons), a Portland cement concrete apron can be constructed with a minimum of 6 inches of concrete on 8 inches of aggregate base. The Portland cement concrete apron should be reinforced with a minimum of #4 rebar at 18 inches on center, both ways.

If the pavements are planned to be placed prior to, or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. Therefore, if the pavement sections will be used for construction access, our firm should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use. If requested, we could provide recommendations for a phased placement of the asphalt concrete to minimize the potential for mechanical scars caused by construction traffic on the finished grade.

The traffic indices should provide the indicated pavement lives with only a normal amount of pavement maintenance. Selection of the design traffic parameters, however, was based on engineering judgment, and not on an equivalent wheel load analysis developed from a traffic study or furnished to us.

In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils and trench backfill should be protected against saturation. Planned concrete sidewalks, driveways, and curb and gutters should be supported directly on the properly compacted native soils. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas

and the pavement section. In addition, a compacted impermeable soil plug should be constructed within any lateral or other trench backfill that passes beneath the curb and gutter and under the adjoining pavement. In addition, water should never be allowed to pond behind the curb and gutter during or after the completion of construction.

The Aggregate Base for use in flexible pavements should conform to Caltrans Standard Specification Section 26-1.02A for Class 2 Aggregate Base. The Aggregate Base used in the pavement sections should be compacted to 95 percent of the soil's maximum dry density (ASTM D1557) and should be firm and unyielding.

7.9 ADDITIONAL GEOTECHNICAL SERVICES

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations. The analyses, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from the subsurface explorations conducted for the project, and upon the conditions existing when services were conducted. Variations of subsurface conditions from those analyzed or characterized in the report are possible, as may become evident during construction. In that event, it may be advisable to revisit certain analyses or assumptions.

We recommend that Fugro be retained to provide geotechnical services during site grading and foundation installation to observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered. During construction, our field engineer should observe and/or test the following:

- Soil conditions exposed by site grading and foundation excavations, to check that they are consistent with those encountered during the field exploration,
- Installation of temporary shoring,
- Pier foundation excavations,
- Pavement subgrade preparation, and
- Fill placement and compaction, including backfill of utilities and compaction of aggregate base.

8.0 LIMITATIONS

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The analyses and recommendations contained in this report are based on the data obtained from the subsurface explorations conducted for this study and relevant previous explorations at the site. These explorations indicate subsurface conditions only at specific locations and times, and only to the depths explored. Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our analyses of the observed conditions. If conditions other than those described in this report are encountered, we should be notified so that we can provide additional recommendations, if warranted.

This report has been prepared for the exclusive use of Children's Hospital and Research Center Oakland and their consultants for specific application to the Children's Hospital development as described herein. In the event that there are any changes in the ownership, nature, design, or location of the proposed project, or if any future additions are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) the project changes are reviewed by Fugro, and 2) conclusions and recommendations presented in this report are modified or verified in writing. Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for non-consulted use of segregated portions of this report.

9.0 REFERENCES

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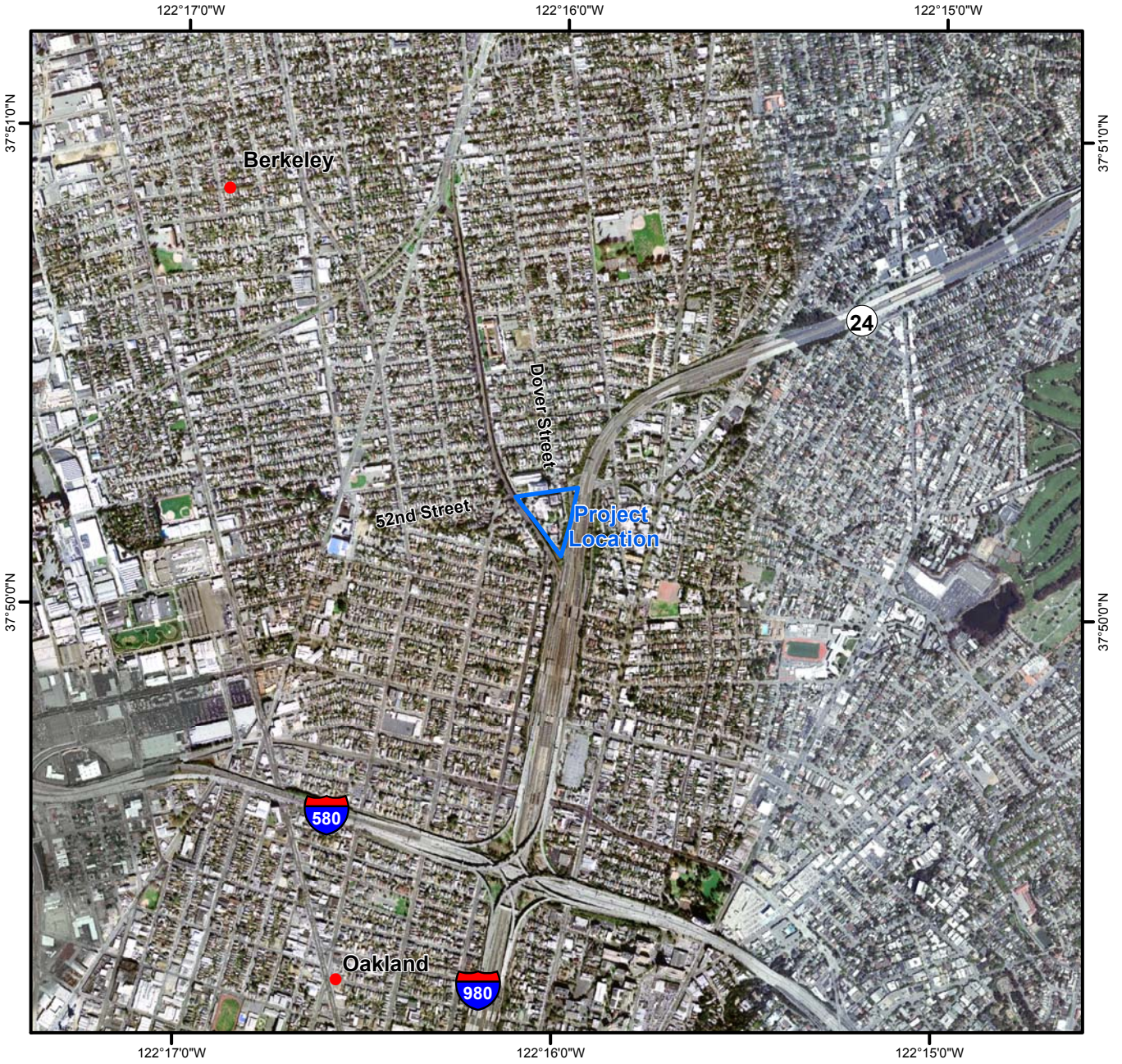
PLATES



Source: Aerial photograph obtained from Google Earth

Site Location:
Latitude: 37.8380°
Longitude: -122.2665°

N

1:24,000

0 1,000 2,000 4,000 Feet

VICINITY MAP
New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

PLATE 1

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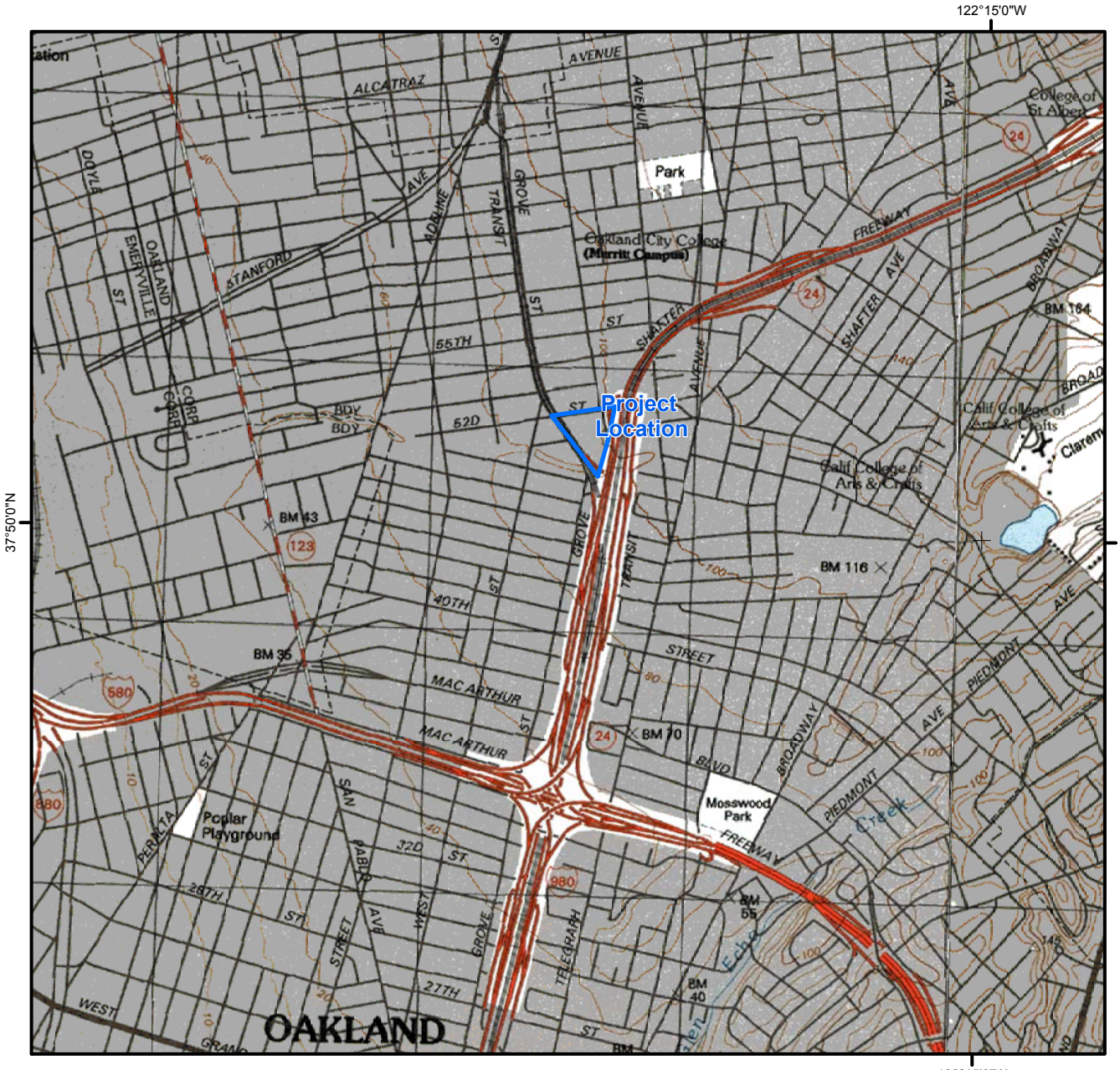

Site Location:
Latitude: 37.8380°
Longitude: -122.2665°

N

1:24,000

0 1,000 2,000 4,000 Feet

TOPOGRAPHIC SITE MAP
New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

PLATE 2

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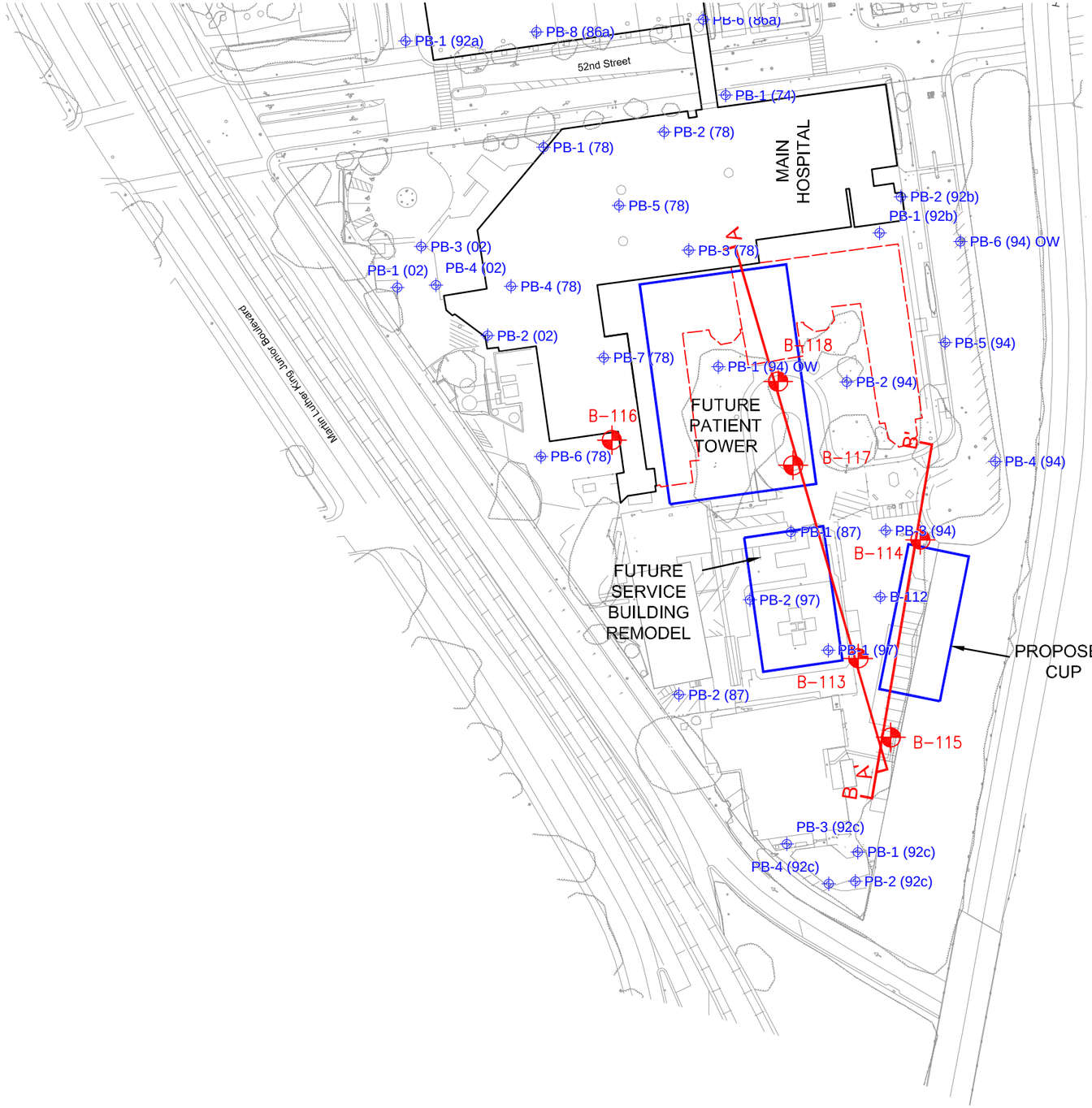

LEGEND

B-118 — APPROXIMATE LOCATION OF BORING (2008)

PB-6 (78) — APPROXIMATE LOCATION OF PREVIOUS BORING

APPROXIMATE LOCATION OF PROPOSED BUILDING

DEMOLISHED STRUCTURE

APPROXIMATE LOCATION OF CROSS SECTION

NOTES:

(74) Harza Kaldveer "Patient Services Pavilion - Phase I" Project No. K938-8-1421

(78) Kaldveer "Addition to Children's Hopsital" Project No. K480-J

(87) Kaldveer "Nursing Tower Investigation" Project No. K339-7

(92b) Kaldveer Harza "Proposed Cath Laboratory" Project No. K938-7-1313

(92c) Kaldveer "Chori Laboratory Addition" Project No. K938-6-1125

(94) Harza "Geologic Hazards and Seismic Response Study" Project No. K938-8-1421

(97) Harza "Helistop Facility" Project No. L998-G

(2) Fugro "Western Expansion" Project No. 21110-GI

NORTH

0 100 200

FEET

SITE PLAN

New Hospital Replacement Project
Children's Hospital and Research Center - Central Utility Plant
Oakland, California

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Children's Hospital and Research Center Oakland
Project No. 1595.002

122°18'0"W 122°17'0"W 122°16'0"W 122°15'0"W 122°14'0"W

37°51'0"N 37°50'0"N 37°49'0"N

Project Location

Derby Creek
Temescal Creek
Glen Echo Creek
Glen Echo Creek (Cemetery Creek)
Bushy Dell Creek
Wildwood Creek
Indian Gulch
Broadway Branch
Rockridge Branch
Pleasant Valley Creek

SAN ANTONIO
EMERYVILLE
OAKLAND
BERKELEY
OAKLAND ARMY BASE

Oakland City College
Golden Gate Jr High Sch
Hawley Sch
St Columbia Sch
Washington School
Peralta School
Claremont Jr High Sch
Lincoln School
Yates Sch
Santa Fe School
Technical High Sch
Hillcrest Sch
Redwood Park
Piedmont Hills Park
Rockridge Sch
Hillcrest Sch
Tompkins High School
Durant School
Westlake Jr High Sch
Lakeview School
Lowell Jr High Sch
Redwood Park
Municipal Rose Garden
Piedmont High School
Piedmont Jr High Sch
Mountain View Cemetery
Golf Course
Aquatic Park
Piles
Pile
Mud
Helipad
OAB

13 14 15 16 17 21

Legend

- Creeks
- Former creeks, buried or drained - dashed where channel was ephemeral
- Underground culverts & storm drains
- Engineered channels
- Flood control channels >100 feet wide
- Water spreads over the ground
- Water seeps out of the ground
- Present watersheds
- Artificial bodies of water
- Bay or slough
- Tidal marsh, circa 1850
  - Now water
  - Now filled or drained land
- Tidal marsh formed after 1850
- Fresh water marsh, modern red=artificial, blue=natural
- Willow groves, circa 1850

N

1:24,000

0 1,000 2,000 4,000 Feet

CREEK LOCATION MAP
New Hospital Replacement Project
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

PLATE 4

Source: Creek and Watershed Map of Oakland and Berkeley; Sowers; 2000.

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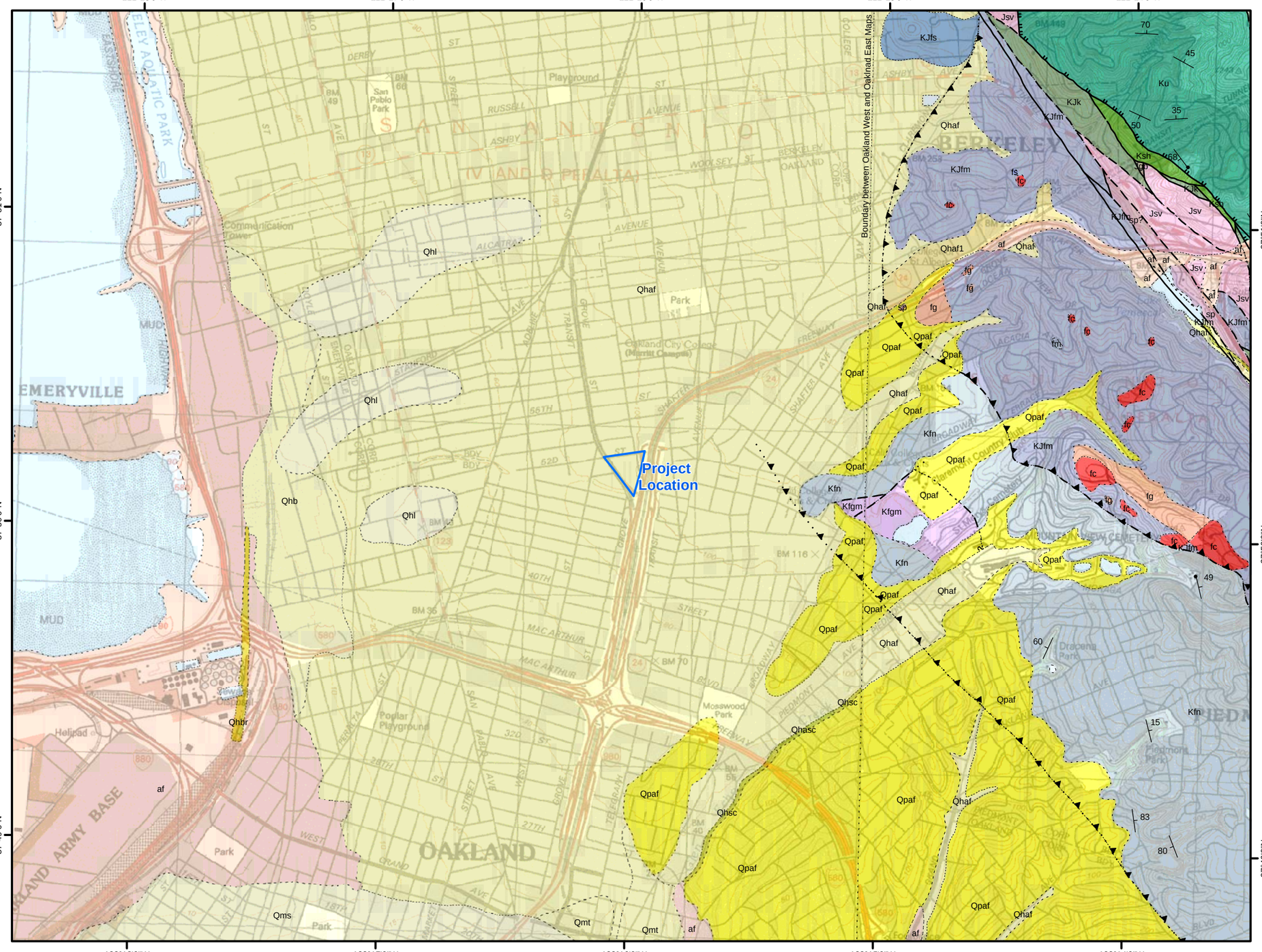

Legend

Contact

Faults (dashed where approx. located, small dashes where inferred, dotted where concealed, queried where location is uncertain)

Fault

Reverse or Thrust Fault

Normal Fault

70 — Strike and Dip of Bedding

49 — Strike and Dip of Bedding, top indicator observed

Geology

water

Surficial Deposits

af	Artificial Fill (Historic)	Qhl	Natural Levee Deposits (Holocene)
Qhasc	Artificial Stream Channels (Historic)	Qhsc	Stream Channel Deposits (Holocene)
Qhaf1	Younger Alluvial Fan Deposits (Holocene)	Qms	Merrit Sand (Holocene and Pleistocene)
Qhaf	Alluvial Fan and Fluvial Deposits (Holocene)	Qmt	Marine Terrace Deposits (Pleistocene)
Qhb	Basin Deposits (Holocene)	Qpaf	Alluvial Fan and Fluvial Deposits (Pleistocene)
Qhbr	Beach Ridge Deposits (Holocene)		

Great Valley Complex

Ku	Undivided Great Valley Complex Rocks (Cretaceous)
Ksh	Siltstone and Shale
KJk	Knoxville Formation (Early Cretaceous and Late Jurassic)
Jsv	Keratophyre and Quartz Keratophyre (Late Jurassic)

Coast Range Ophiolite

Jgb	Gabbro

Franciscan Complex

Kfn	Sandstone of the Novato Quarry Terrane of Blake and Others (1984) (Late Cretaceous)	fc	Serpentinite Blocks
Kfgm	Fine-grained Quartz Diorite (Late Cretaceous (?))	sp	Chert Blocks
KJfs	Franciscan Complex Sandstone, Undivided (Late Cretaceous to Late Jurassic)	fg	Greenstone Blocks
KJfm	Franciscan Complex, Melange (Cretaceous Late Jussaric)	fm	Schist Blocks
fs	Graywacke and Meta-Graywacke Blocks		

N

1:24,000

0 1,000 2,000 4,000 Feet

GEOLOGIC MAP

New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

PLATE 5

Source: USGS Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California; Graymer; 2000.

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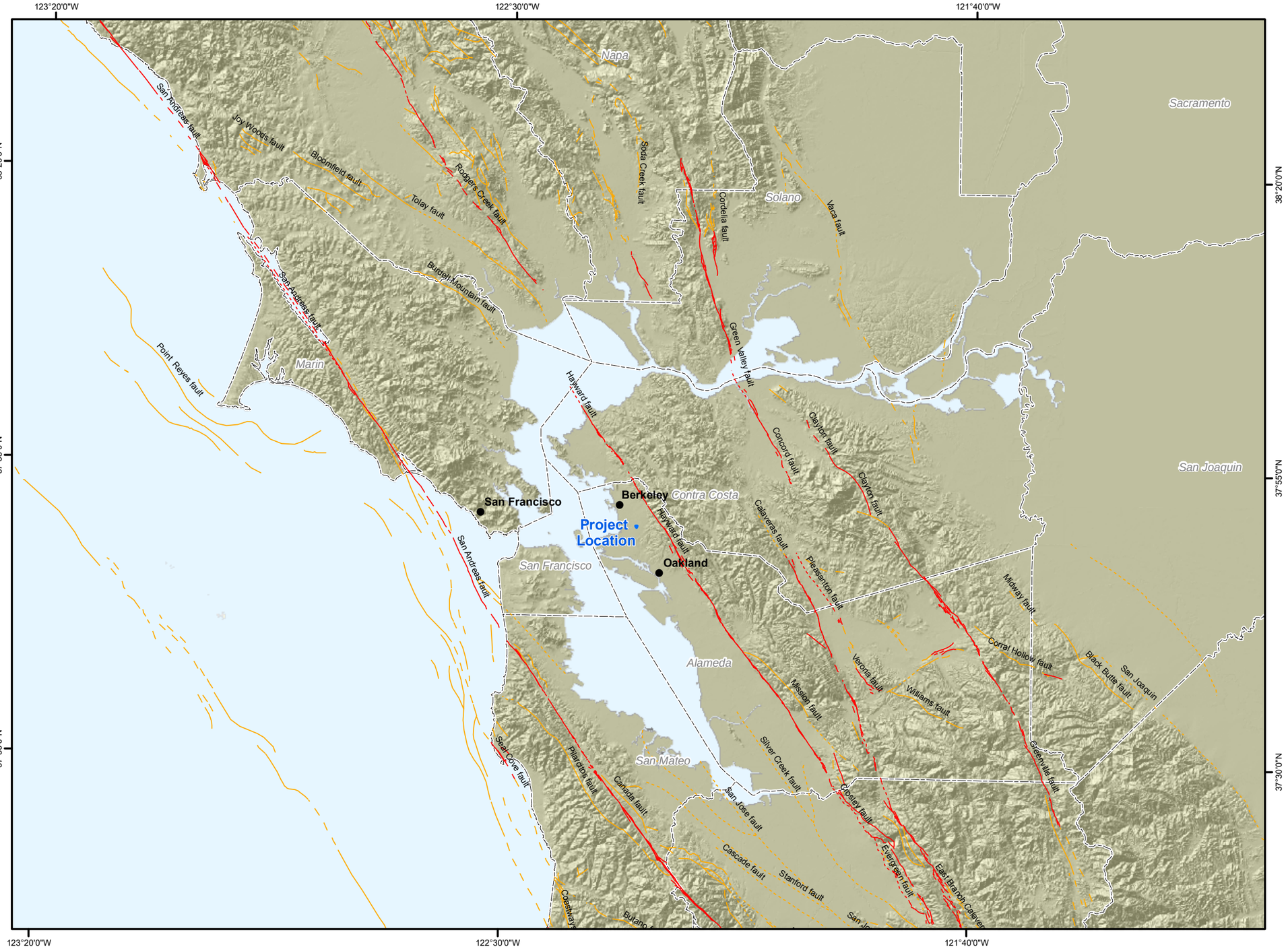

Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Sources:

a) Bryant, 2005

b) Jennings, 1994

N

1:633,600

0 5 10 20 Miles

REGIONAL FAULT MAP
New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland, California
PLATE 6a

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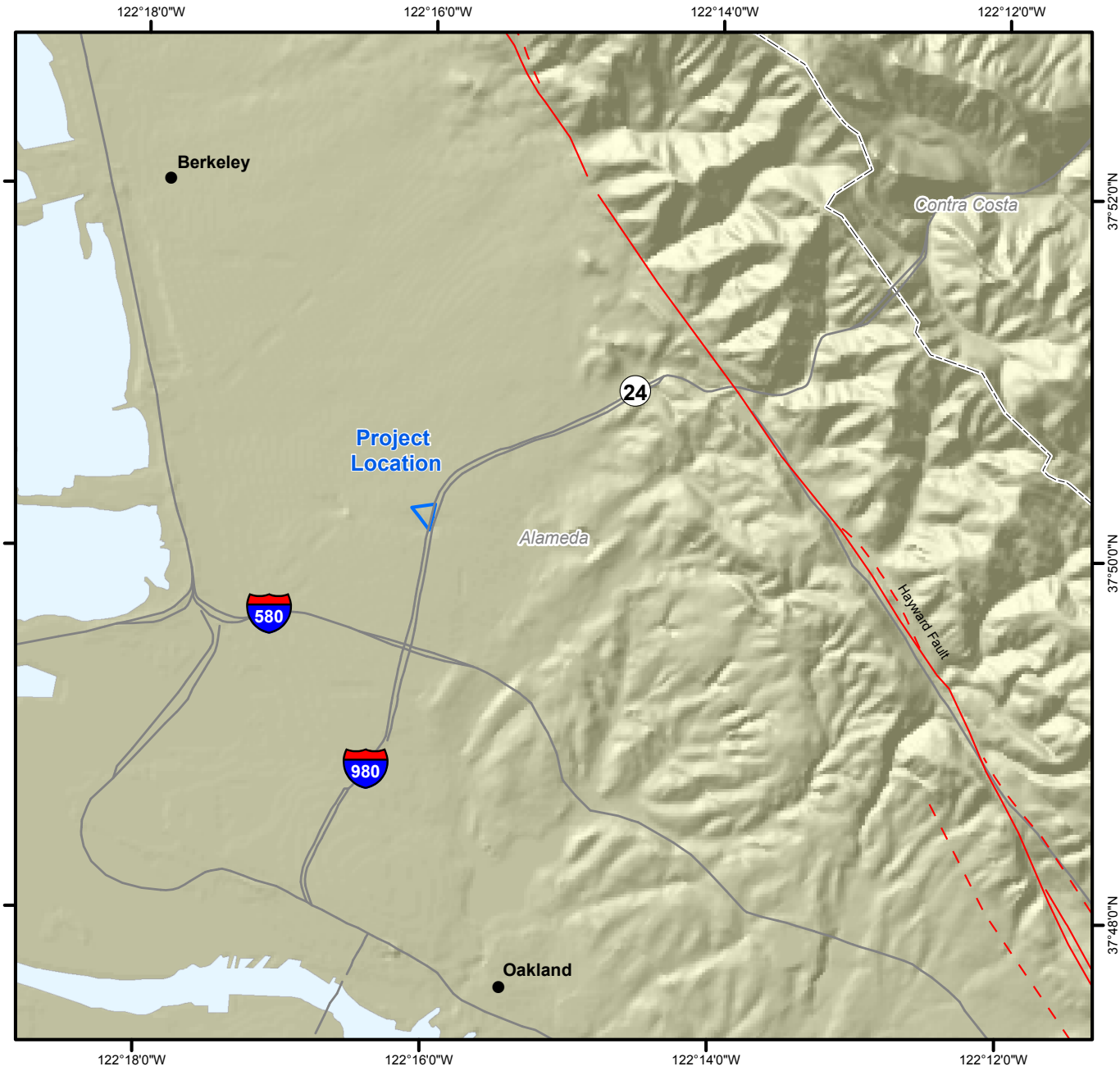

Source: Bryant, 2005; Jennings, 1994; Fisher et al., 2005

Legend

- Major Roads
- County Limits

Faults

- Active Fault, dashed where inferred
- Potentially Active Fault, dashed where inferred

N

1:63,360

0 0.5 1 2 Miles

VICINITY FAULT MAP

New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

PLATE 6b

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123°20'0"W 122°30'0"W 121°40'0"W

38°20'0"N 37°55'0"N 37°30'0"N

5.6 (1969)
5.7 (1969)
5.2 (2000)
5.5(1891)
6.2(1898)
5.5(1902)
6(1889)
5.4 (1955)
5.3(1870)
5.8 (1980)
5.1 (1980)
5.2(1889)
5.6(1861)
5.4 (1980)
5.4(1899)
8.2(1906)
6.8(1868)
5.7(1864)
5.3 (1957)
7(1838)
5.3(1864)
5.5(1903)
5.5(1856)
6.1(1858)
5.8(1866)
5.3(1856)
5.7 (1986)
5.4 (2007)
5.3 (1988)
5.9(1881)
5.5(1903)
5.5(1891)
6.2 (1984)
6.3(1865)

Napa
Sacramento
Solano
Marin
Contra Costa
San Francisco
San Joaquin
Alameda
San Mateo

San Andreas fault
Joy Woods fault
Bloomfield fault
Tolay fault
Rodgers Creek fault
Soda Creek fault
Cordelia fault
Vaca fault
Burdell Mountain fault
Point Reyes fault
Green Valley fault
Hayward fault
Concord fault
Clayton fault
Calaveras fault
Greenville fault
Midway fault
Ortigalita fault
Coast Ranges fault
Verona fault
Williams fault
Black Butte fault
Mission fault
Silver Creek fault
Crosley fault
Seal Cove fault
Pilarcitos fault
Canada fault
San Jose fault
Cascade fault
Stanford fault
Coastways fault
Butano fault

Berkeley
Oakland
San Francisco
Project Location

Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Historical Earthquake Magnitude (1836 - March 2008)
Magnitudes equal to and greater than 5 are labeled.

- 3.0 - 3.9
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7+

Sources:

1) Earthquake Data:
a) Earthquake epicenters from ANSS Composite Catalog Search, 1898 to 2008, <www.ncedc.org/anss/> (downloaded March 2008)

b) USGS Earthquake Hazards Program Search, 1735 to 1974, <http://neic.usgs.gov/neis/epic/epic_circ.html> (downloaded March 2008)

2) Faults:
a) Bryant, 2005

b) Jennings, 1994

N

1:633,600

0 5 10 20 Miles

FAULTING AND HISTORICAL SEISMICITY
New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

PLATE 7

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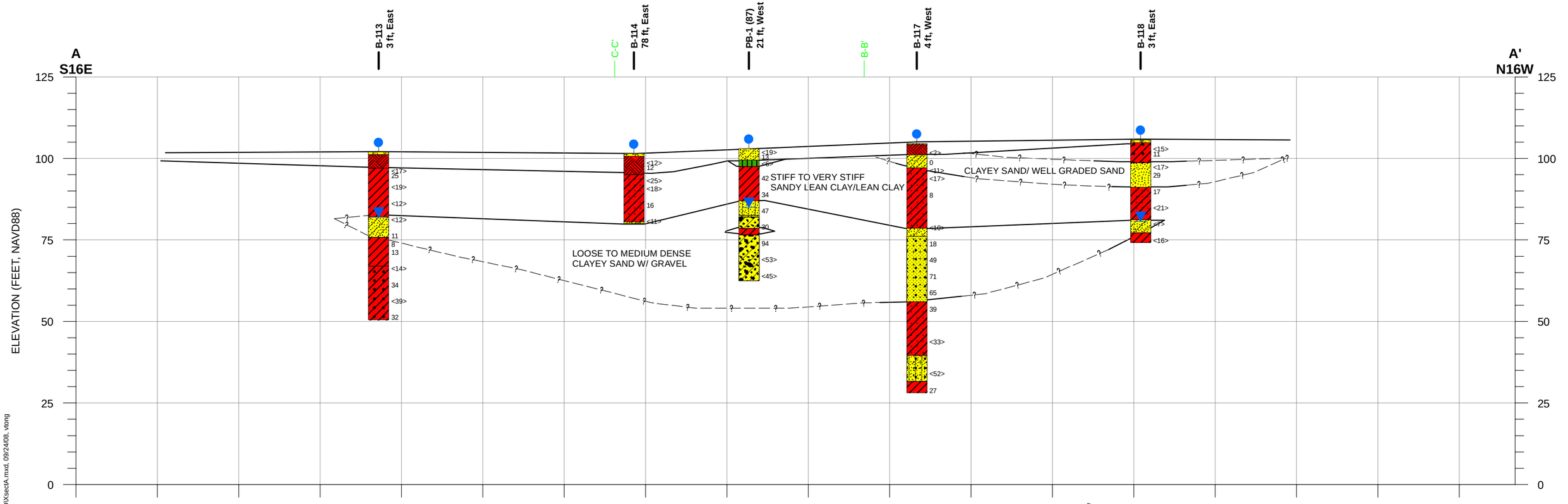
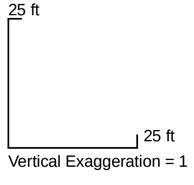
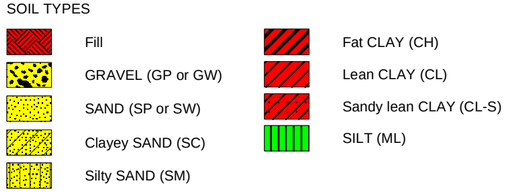
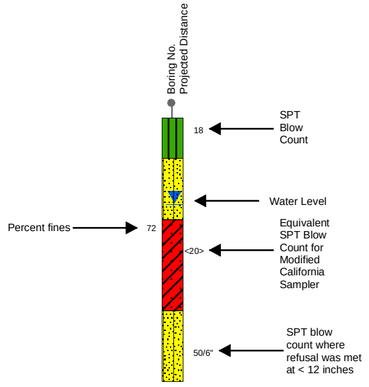


Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.







SUBSURFACE CROSS SECTION A-A'
New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland,California

PLATE 8a

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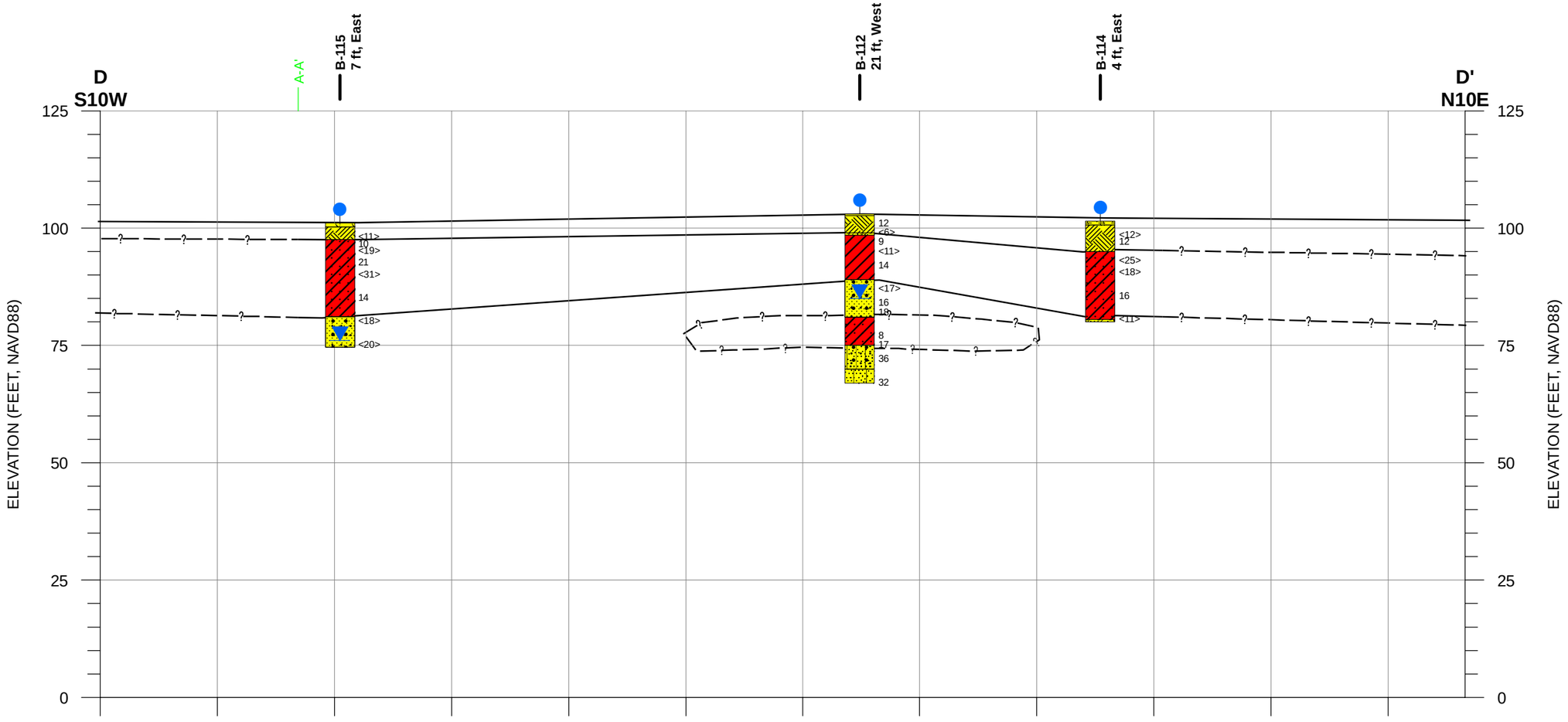
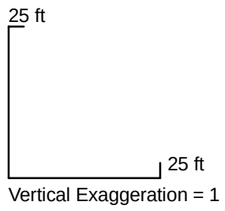
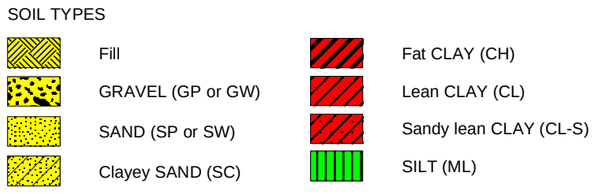
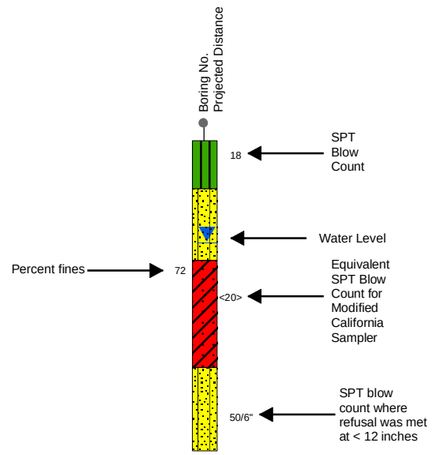


Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.







SUBSURFACE CROSS SECTION D-D'
New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland,California

PLATE 8b

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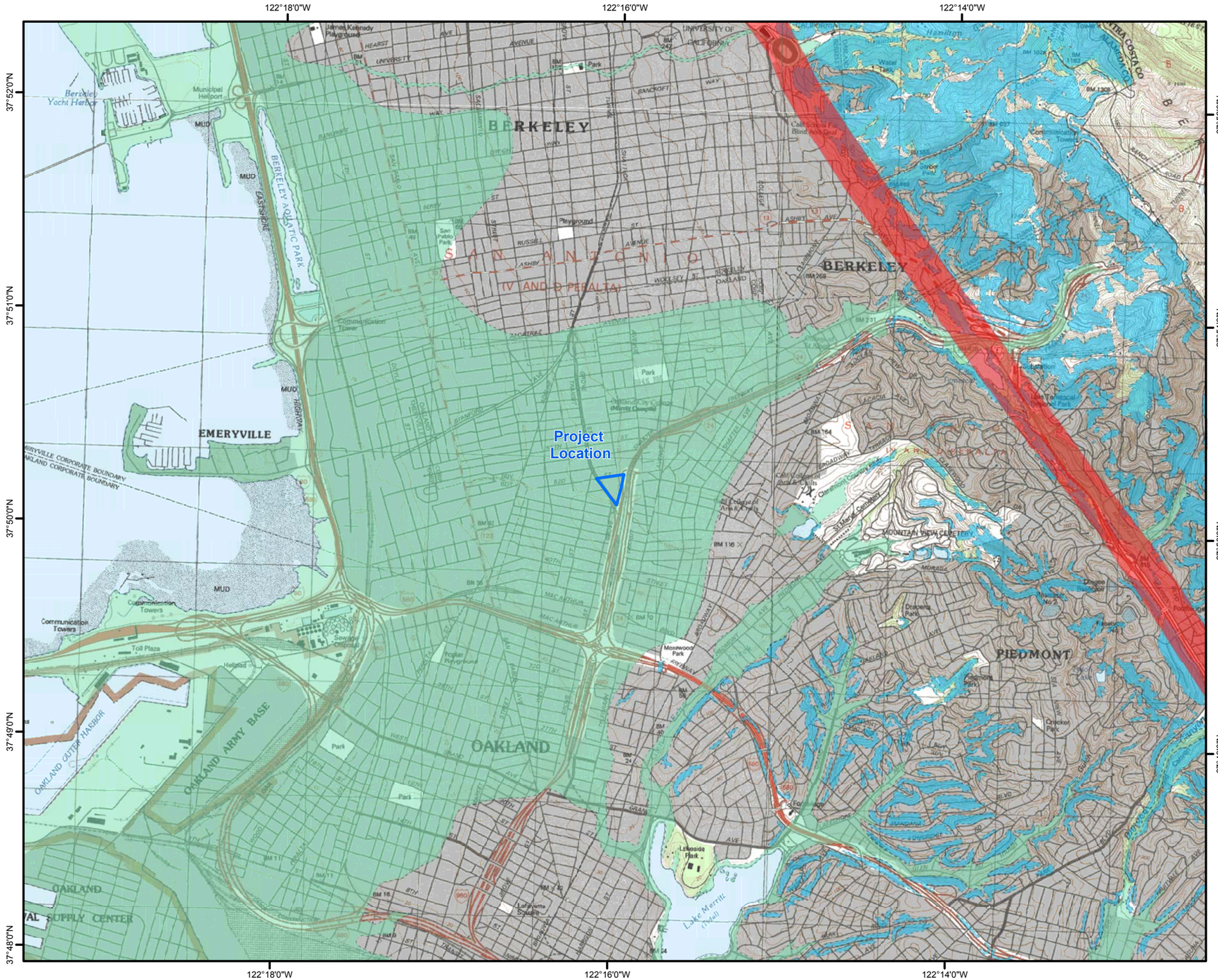


Legend

Liquefaction Hazard Zone
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacements.

Earthquake-Induced Landslide Hazard Zone
Areas where previous occurrence of landslide movement, or local topographic, and geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements.

Alquist-Priolo Earthquake Fault Zone

Sources:
1) Alquist-Priolo (AP) Special Studies Zone Boundaries: California Geologic Survey (2002)
2) Liquefaction and Landslide Special Studies Zones: California Geologic Survey <http://www.conservation.ca.gov/cgs/shzp/>

N

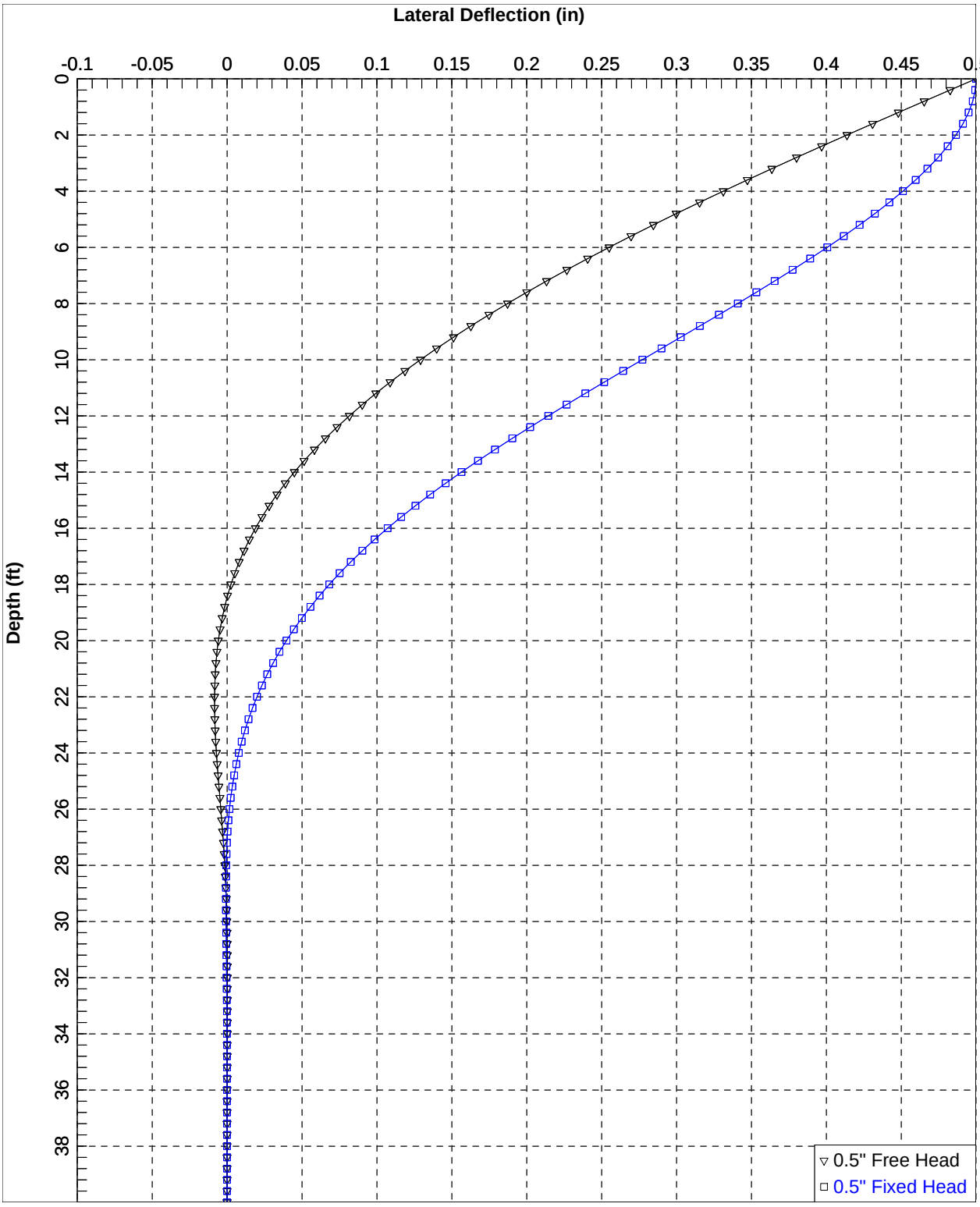
1:36,000

0 1,500 3,000 6,000 Feet

CGS SEISMIC HAZARD ZONE MAP
WITH ALQUIST-PRIOLO EARTHQUAKE ZONES
New Hospital Replacement Project-
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

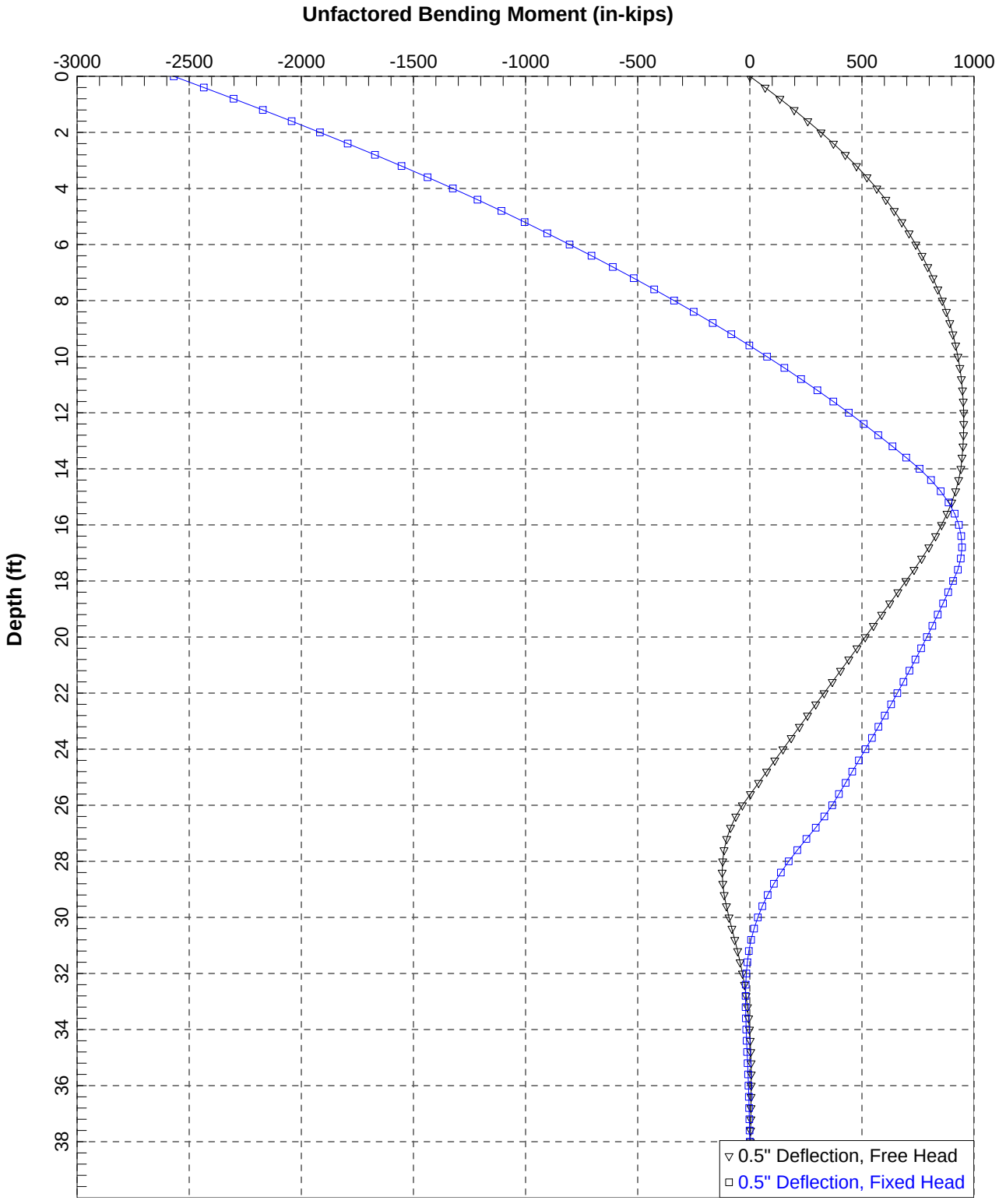
PLATE 9

N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\October_2008\mxd\Central_Utility_Plant\Plate9-Seismic_Hazard.mxd, 10/17/08, ksheil



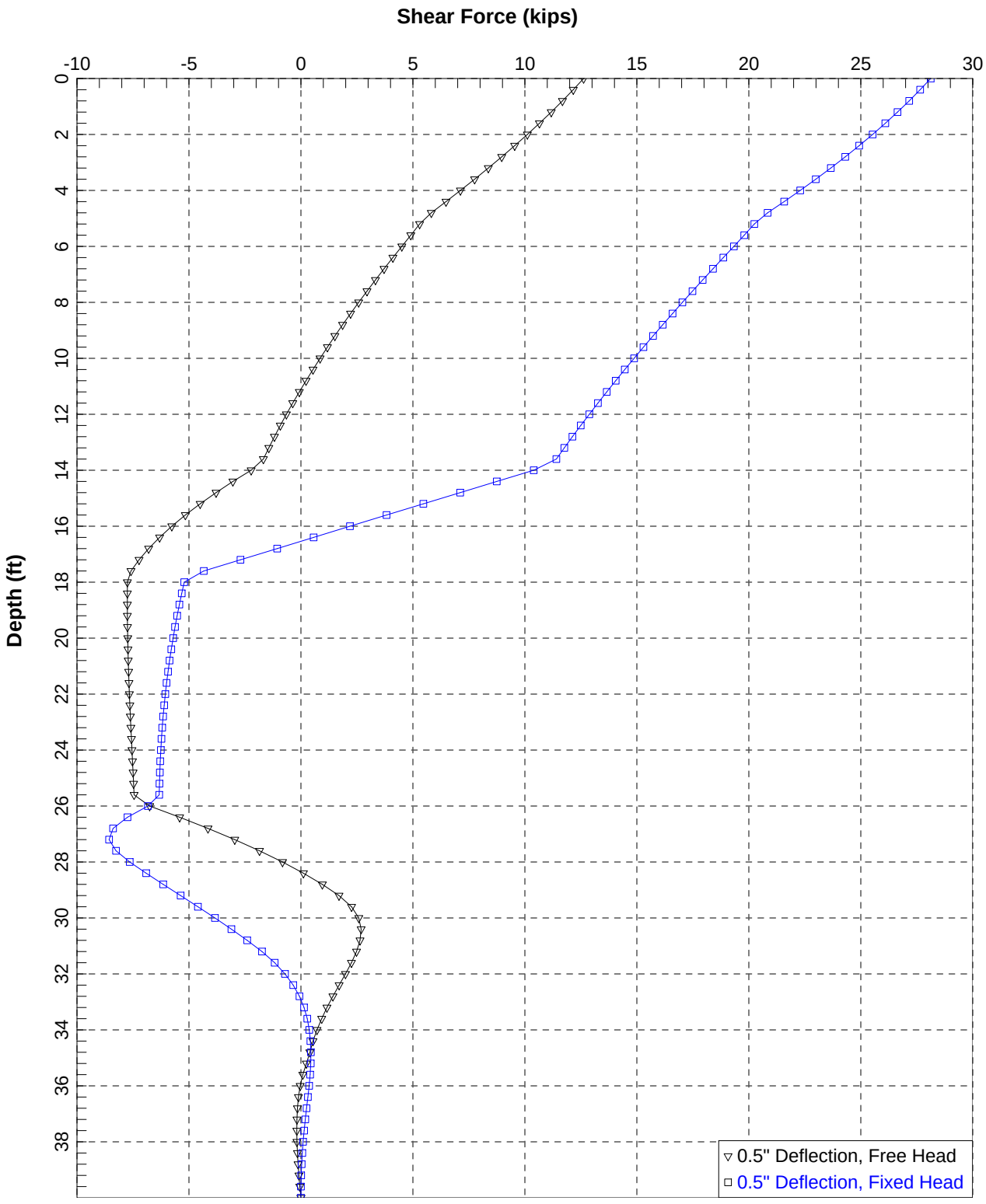
LATERAL PILE ANALYSIS
New Hospital Replacement Project -
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

PLATE 10a



LATERAL PILE ANALYSIS
New Hospital Replacement Project -
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

Plate 10b



LATERAL PILE ANALYSIS
New Hospital Replacement Project -
Central Utility Plant
Children's Hospital and Research Center
Oakland, California

Plate 10c

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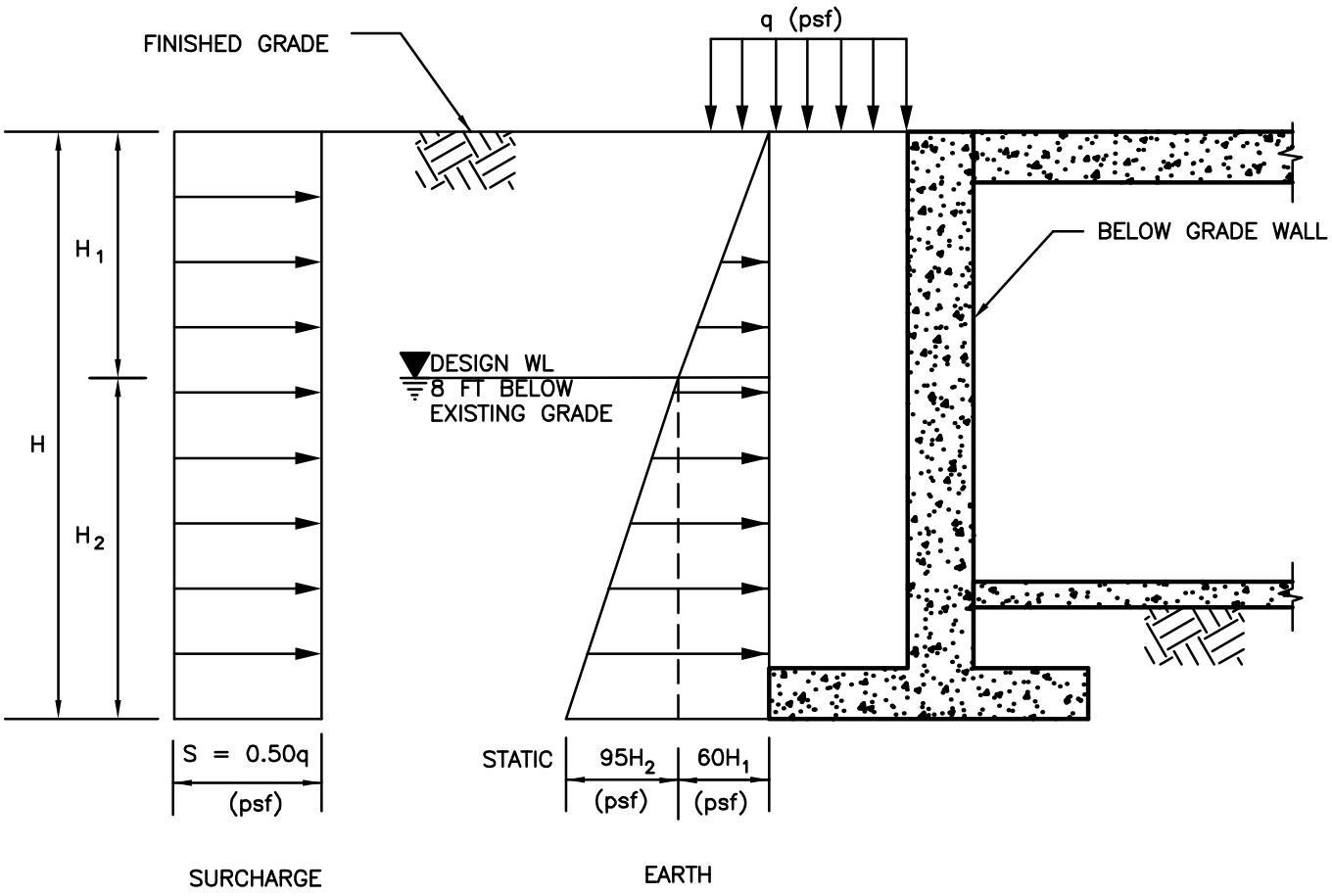

LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.50q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

1) RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE.
2) DESIGN GROUND WATER LEVEL IS 8 FEET BELOW EXISITNG GRADE.
3) USE $60H_1$ ABOVE AND $95H_2$ BELOW THE DESIGN WATER LEVEL FOR BOTH STATIC AND SEISMIC LOADING CONDITIONS.
4) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL.
5) ASSUMES LEVEL BACKFILL.

STATIC LATERAL PRESSURES ON RESTRAINED WALLS

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 11a

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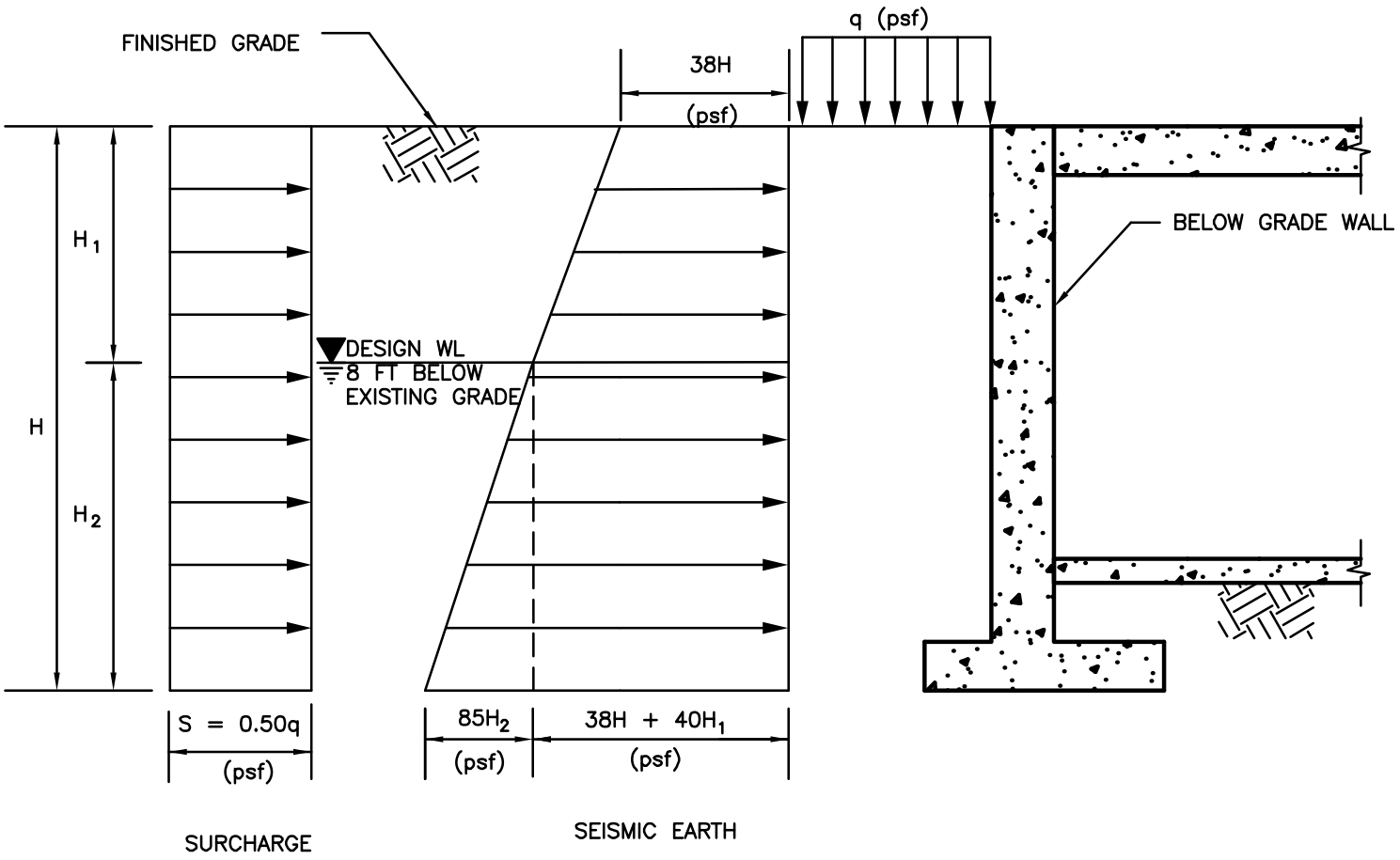


LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.50q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

1) RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE.
2) DESIGN GROUND WATER LEVEL IS 8 FEET BELOW EXISITNG GRADE.
3) USE $40H_1$ ABOVE AND $85H_2$ BELOW THE DESIGN WATER LEVEL FOR BOTH STATIC AND SEISMIC LOADING CONDITIONS.
4) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL.
5) ASSUMES LEVEL BACKFILL.

SEISMIC LATERAL PRESSURES ON RESTRAINED WALLS

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 11b

APPENDIX A
FIELD EXPLORATIONS

APPENDIX A
FIELD EXPLORATIONS

The field exploration consisted of a surface reconnaissance and a subsurface exploration program. Our subsurface investigation for the CUP included drilling three conventional geotechnical borings. The following sections contain a discussion regarding the equipment and procedures to perform the exploration. The logs of the borings, as well as a key for the classification of the soil (Plate A-1), are included as part of this appendix.

V&W Drilling, of Lodi, California, drilled the two borings for this project between August 4 and 8, 2008. The borings were drilled using a truck-mounted CME-75 drill rig equipped with 8-inch diameter hollow-stem augers, 5-inch diameter solid-flight augers, and 4 3/8-inch diameter mud rotary drilling capabilities. The borings were advanced to a maximum depth of about 26 feet below grade. The approximate locations of the borings are shown on Plate 3 – Site Plan.

Soil samples were obtained using either a standard split spoon sampler (2-inch outer diameter, 1 3/8-inch inner diameter) or Modified California Liner sampler (3-inch outer diameter, 2 3/8-inch inner diameter). The sampler type is indicated on the boring logs as designated on the Terms and Symbols, Plate A-1. Soil samples were taken semi continuously to a depth of about 10 feet and at 5-foot intervals thereafter. The Standard Penetration Test (SPT) was performed as part of the sampling procedure (in accordance with ASTM D-1586). The Standard Penetration Test is a measure of soil density and consistency. The SPT-N value (blow count) is defined as the number of blows required to drive a 2-inch diameter split-barrel sampler 12-inches, after an initial penetration of 6-inches, using a 140-pound hammer falling freely for 30-inches. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the sampler the last 12 inches. We note that samples obtained using the Modified California Liner sampler are not considered standard N-values because of the larger diameter of the samplers.

A Fugro engineer observed the drilling operation and logged the soil and rock encountered. Our inspecting engineer recorded SPT N-values. Recovered soil samples were visually examined and classified in accordance with the Unified Soil Classification System (USCS) and ASTM D-2488 (visual method). Our engineer estimated soil undrained shear strength in the field with a Pocket Penetrometer. Soil samples were transported to our office and laboratory for further examination, confirmation of classification and laboratory testing.

The surface elevations on the boring logs were estimated by interpreting the topographic contours on the project topographic map prepared by Chaudhary & Associates, Inc.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated. We do not warrant they are representative of subsurface conditions at other locations and times.

MAJOR DIVISIONS			GROUP NAMES	
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW	Well-Graded Gravel
			GP	Poorly Graded Gravel
		Gravels with more than 12% fines	GM	Silty Gravel
			GC	Clayey Gravel
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	Clean sand less than 5% fines	SW	Well-Graded Sand
			SP	Poorly Graded Sand
		Sands with more than 12% fines	SM	Silty Sand
			SC	Clayey Sand
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML	Silt
			CL	Lean Clay
			OL	Organic Silt
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH	Elastic Silt
			CH	Fat Clay
			OH	Organic Clay
HIGHLY ORGANIC SOILS			PT	Peat or Highly Organic Soils
			FILL	Debris or Mixed Fill
			AC	Asphalt Concrete Pavement with Aggregate Base

GENERAL NOTES

Classification of Soils per ASTM D2487 or D2488

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

Sloped line in break column indicates transitional boundary

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

SAMPLER DRIVING RESISTANCE

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

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
50/7"	50 blows drove sampler 7" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

STRENGTH TEST METHOD

U = Unconfined Compression
Q = Unconsolidated Undrained Triaxial
T = Torvane
P = Pocket Penetrometer
M = Miniature Vane
F = Field Vane

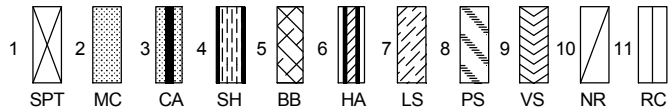
OTHER TESTS

k = Permeability
Consol = Consolidation
Gs = Specific Gravity
MA = Particle Size Analysis
EI = Expansion Index
OVM = Organic Vapor Measurement

WATER LEVEL SYMBOLS

- Initial or perched water level
- Final ground water level
- Seepages encountered

SAMPLER TYPE



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

1 SPT Sampler, driven 1 3/8" ID, 2" OD
2 MOD CA Liner Sampler 2 3/8" ID, 3" OD
3 CA Liner Sampler 1 7/8" ID, 2.5" OD
4 Thin-walled Tube, pushed 2 7/8" ID, 3" OD
5 Bulk Bag Sample (from cuttings)
6 Hand Auger Sample
7 Lexan Sample
8 Pitcher Sample
9 Vibracore Sample
10 No Sample Recovered
11 Rock Core

SOIL STRUCTURE

Fissured: Containing shrinkage or relief cracks, often filled with fine sand or silt, usually more or less vertical.

Pocket: Inclusion of material of different texture that is smaller than the diameter of the sample.

Parting: Inclusion less than 1/8 inch thick extending through the sample.

Seam: Inclusion 1/8 inch to 3 inches thick extending through the sample.

Layer: Inclusion greater than 3 inches thick extending through the sample.

Laminated: Soil sample composed of alternating partings or seams of different soil types.

Interlayered: Soil sample composed of alternating layers of different soil type.

Intermixed: Soil sample composed of pockets of different soil type, and layered or laminated structure is not evident.

CONSISTENCY

Clays	Blows/Foot SPT	Undrained Shear Strength (ksf)
Very Soft	0 - 2	0 - 0.25
Soft	2 - 4	0.25 - 0.5
Firm	4 - 8	0.5 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

RELATIVE DENSITY

Sands and Gravels	Blows/Foot SPT
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

INCREASING VISUAL MOISTURE CONTENT

Dry
Moist
Wet

Information on each boring log is a compilation of subsurface conditions and soil or rock classifications obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the time and places indicated, and can vary with time, geologic condition, or construction activity.

TERMS AND SYMBOLS USED ON BORING LOGS

PLATE A-1

LOCATION: SEE SITE PLAN, PLATE 3
N 6051529.36 E 2131714.14
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 102.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
	1			4 inches of AC and 6 inches of AB							
100				**ARTIFICIAL FILL (af)** Sandy Lean CLAY (CL): reddish brown, dry to moist, low to medium plasticity - bulk sample taken from auger cuttings		6		22	10		
5	2		(28)	**ALLUVIUM (Qal)**							
	3		25	Lean CLAY with sand (CL): very stiff, light yellowish brown, moist, medium plasticity						3.5 M	
95											
10	4		(30)	- very stiff, low to medium plasticity, fine sand	107	19				3.1 M	
90											
15	5		(19)	- stiff, moist, low to medium plasticity						2.8 P	
85											
20	6		(19)	Clayey Sand (SC): loose to medium dense, yellowish brown, wet, grades into some rectangular gravel up to 1/2"			29				
80											
25	7		11	- medium dense, fine to medium grained sand, grades to silty clay at the bottom of sampler			40				
75	8		8	Lean CLAY (CL): stiff, grayish brown, moist, medium to high plasticity							
30	9		13								
70											
35	10		(23)	Sandy Lean CLAY with gravel (CL): very stiff, light brown, moist, low plasticity, fine sand with gravel up to 1/2"							
65											

Continued

BORING DEPTH: 51.5 ft
BACKFILL: Grout
DEPTH TO WATER: 20.0 ft
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-113

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 6051529.36 E 2131714.14
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 102.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
60 / 45		11		34	- medium grained sand							
55 / 50		12		(63)	hard, moist, yellowish brown, fine sand, low to medium plasticity, trace gravel						3.5 M	
50 / 50		13		32	- very stiff, light brown, with medium grained sand							
45 / 55					End of Boring at 51-1/2 ft.							
40 / 60												
35 / 65												
30 / 70												
25 / 75												

BORING DEPTH: 51.5 ft
BACKFILL: Grout
DEPTH TO WATER: 20.0 ft
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-113

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 6051579.8 E 2131810.51
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 101.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
					4 inches of AC and 6 inches of AB							
100 / 0		1		(20)	**ARTIFICIAL FILL (af)** Sandy Lean CLAY (CL): stiff, dark brown, moist, low to medium plasticity	96	20				3.8 P	
		2		12								
95 / 5												
		3		(40)	**ALLUVIUM (Qal)** Sandy Lean CLAY (CL): very stiff, light yellowish brown, moist, fine sand, low to medium plasticity				41	26	4.5 P	
90 / 10		4		(29)	- yellowish brown, low to medium grained sand, medium plasticity	113	16				~~1.9 M~~ 1.8 P	
85 / 15		5		16	- stiff, gray				34	19		
80 / 20		6		(17)							0.9 M	
					Clayey SAND with silt (SC): loose, yellowish brown, moist, fine grained sand at the bottom of the sampler							
75 / 25					End of Boring at 21-1/2 ft.							
70 / 30												
65 / 35												

BORING DEPTH: 21.5 ft
BACKFILL: Grout
DEPTH TO WATER:
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-114

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 6051555.79 E 2131649.93
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 101.1 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
				4 inches of AC and 6 inches of AB							
100	1		(17)	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, dark brown, dry, low to plasticity		9				2.5 P	
	2		10	**ALLUVIUM (Qal)**							
5 / 95	3		(31)	Sandy Lean CLAY (CL): firm, light to dark brown, dry to moist, low plasticity - very stiff, dark brown							
	4		21	- very stiff, light brown	100	23					
10 / 90	5		(49)	- very stiff, light yellowish brown, moist, low to medium plasticity	114	17				4.5 P	
15 / 85	6		14	stiff, tan and light brown, low to medium plasticity							
20 / 80	7		(29)	Clayey SAND with gravel (SC): medium dense, light brown, moist, fine grained sand, angular gravel up to 1"			35				
							22				
25 / 75	8		(32)	▽ - light grayish brown, moist, fine sand with gravel			27			1.8 M	
				End of Boring at 26-1/2 ft.							
30 / 70											
35 / 65											

BORING DEPTH: 26.5 ft
BACKFILL: Grout
DEPTH TO WATER: 25.0 ft
COMPLETION DATE: August 4, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-115

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

APPENDIX A (CONTINUED)
HISTORICAL BORINGS

LOCATION: SEE SITE PLAN, PLATE 3
N 2131764 E 6051547
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 103.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
0				Asphalt Concrete Pavement							
	1		12	**ARTIFICIAL FILL (af)** Lean CLAY (CL): firm to stiff, black, moist, trace fine gravel							
100	2 3		(9)								
				- 6" clayey sand layer, trace gravel, from 4 to 4.5'	92	22				1.8 M 0.5 R	Corrosion
5	4		9	**ALLUVIUM (Qal)**						2.0 P	
	5 6		(18)	Lean CLAY with sand (CL): stiff to very stiff, brown, moist, fine-grained sand, low plasticity fines, trace subangular to subrounded fine gravel	102	23				3.0 M 1.0 R 2.5 P	
95											
10	7		14	- low to medium plasticity, trace fine-grained sand						3.5 P	
90											
15	8 9		(28)	Clayey SAND with gravel (SC): medium dense, brown, moist to wet, fine- to coarse-grained sand, well graded, subangular to subrounded fine to coarse gravel up to 1.25-inch in size;			14				
85	10		16	▽ medium plasticity - with red iron staining, at 18'		17	15	31	17		
20	11		18	- increased sand content, at 20'							
80				Sandy Lean CLAY (CL): firm, gray, wet, very fine-grained sand, low plasticity			66				
25	12		8								
	13		17								
75				Silty SAND with gravel (SM): dense, brown, moist, fine-to coarse-grained sand, medium plasticity, subangular to subrounded fine gravel							
30	14		36								
70				Poorly-graded SAND with silt (SP-SM): dense, brown, moist, medium-to coarse-grained, low plasticity							
35	15		32								
				End of Boring at 36'							
65											

BORING DEPTH: 36.0 ft
BACKFILL: Grout
DEPTH TO WATER: 18.0 ft
COMPLETION DATE: February 14, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-112

New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	ERT
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	9/7/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 2" AC/4" AB AGGREGATE BASE, 4 inches **CLAY (CL),** brown, silty, sandy (fine- to medium-grained), damp (with silt, some sand (fine- to medium-grained below 2½ feet)	Firm Stiff				13 9 26	23 21	96 99		LL=37, PI=21, Passing #200 Sieve = 77%
(trace gravel (subangular, fine) below 8½ feet)			10		14	19	109	2.3	
SAND (SC), motled reddish-brown to borwn, fine- to coares-grained sand, with gravel (fine and coarse, subangular to angular), some clay, some interbedded gravel, damp (some clay, moist to wet below 18½ feet)	Medium Dense		15 20		40 31	14 17	120 115	1.0	Triaxial Test, Deviator Stress=0.79 ksf
CLAY (CL), mottled light rust brown and grey, silty trace sand (fine-grained), damp	Very Stiff		25 30		17 20				
SAND (SC), light orange brown, fine- to coarse-grained, gravelly (fine and coarse, subangular to angular), with clay, moist to wet	Medium Dense		35		31 48				

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL WESTERN EXPANSION
Oakland, California

PROJECT NO.	DATE	BORING NO.
17979-CA	September 2000	EB-1

File Name: G:\ENGINEER\GINTW\PROJECTS\17979.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	ERT
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	9/7/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SAND (SC), Continued,	Medium Dense Dense		45		53				

Bottom of Boring = 45 Feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of penetration resistance values, see Appendix A.
3. Ground water level was measured at 22½ feet at the time of drilling.
4. The boring was backfilled with neat cement in accordance with Alameda County Public Works Department requirements.
5. LL = Liquid Limit, PI = Plasticity Index

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL WESTERN EXPANSION
Oakland, California

PROJECT NO.	DATE	BORING NO.
17979-CA	September 2000	EB-1

File Name: G:\ENGINEER\GINTW\PROJECTS\17979.GPJ Report Template: FUGRO Output Date: 3/14/02

Sept. 2008 Report ID: PB-1 (02)

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/12" AB									
FILL: CLAY, (CL/CH), dark brown, some gravel (subangular), trace sand, damp	Very Stiff				47				
					12				
FILL: SAND (SP), light brown, damp, (trench backfill)	Medium Dense		5		21				
CLAY (CL), brown, silty, some gravel (subangular to subrounded), trace sand (fine- grained), damp	Stiff				17				Passing #200 Sieve = 58%
(grading mottled brown and gray, some sand (fine- to coarse- grained), no gravel at 14 feet)			10		26	26	98	1.9	
			15		22	25	102		Passing #200 Sieve = 68%, LL=33, PI=19
SAND (SC), brown, fine- to coarse-grained, with gravel (subrounded to subangular), some clay, wet	Medium Dense		20		35	17			Passing #4 Sieve = 66%, #200 Sieve = 18%
CLAY (CL), reddish-brown mottled gray, silty, moist	Very Stiff		25						
(interbed of gravel between 27 and 28 feet					45	23	106	14.8	Passing #200 Sieve = 89% LL=43, PI=27

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-2

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), moist	Very Stiff								
(grading trace gravel (coarse, subrounded) at 34½ feet)			35		37				
SAND (SC), brown, fine- to coarse-grained, with gravel (coarse, subangular to subrounded), some clay and silt, wet	Dense								Passing #4 Sieve = 62%, #200 Sieve = 18%
CLAY (CL), brown mottled gray, sandy (fine- to coarse- grained), some gravel (coarse, subangular to subrounded), trace silt, damp	Hard		40		54	16			Passing #200 Sieve = 57%, LL=39, PI=23
SAND (SC), brown, fine- to coarse-grained, some clay and silt, trace gravel (fine to coarse, subangular to subrounded), moist	Dense		45		52	21			Passing #200 Sieve = 26%
CLAY (CL), brown, silty, some sand (fine- to coarse- grained), damp	Hard		50		61	25	102	18.3	

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid Limit, PI=Plasticity Index

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-2

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/12" AB underlain by geosynthetic									
CLAY (CL), brown, with sand (fine-grained), some gravel (fine, weak subangular sandstone fragments), trace silt, damp	Very Stiff				47				
					24				
			5		54	22	102		
(grading gravelly at 8½ feet)	Stiff				19				
			10						
SAND (SC), brown, fine- to coarse-grained, with gravel (coarse, subangular to subrounded), some clay and silt, damp	Medium Dense				43				Passing #4 Sieve = 63%, #200 Sieve = 20%
			15						
(grading gray-brown, fine- grained, clayey, some silt, moist to wet at 18 feet)					26	24	101	1.5	Passing #200 Sieve = 48%, LL=28, PI=7
			20						
(grading reddish-brown, fine- to coarse-grained, some gravel (fine to coarse, subrounded to subangular), some clay, wet					45	20			Passing #4 Sieve = 71%, #200 Sieve = 15%
			25						Passing #200 Sieve = 75%, LL=38, PI=24
CLAY (CL), brown mottled gray, silty, some sand (fine- to coarse- grained), damp	Very Stiff				48	21	111	9.7	

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-3

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), brown mottled gray, silty, some sand (fine- to coarse- grained), damp	Very Stiff								
GRAVEL (GC), brown, fine to coarse, subrounded to subangular, some sand (fine- to coarse- grained), some clay, wet	Dense		35		58	13			Passing #4 Sieve = 45%, #200 Sieve = 16%
SAND (SC), brown, fine- to coarse-grained, silty, trace gravel (fine to coarse, subrounded to subangular), moist	Dense		40		52	22			Passing #4 Sieve = 97%, #200 Sieve = 41%, LL=46, PI=28
CLAY (CL), rust brown, sandy (fine- to coarse- grained), some silt, wet	Very Stiff		45		43	21	112	11.6	Passing #200 Sieve = 54%, LL=44, PI=28
(grading silty, with sand (fine- grained), damp to moist at 48½ feet)	Hard		50		51	22	108		Passing #200 Sieve = 63%, LL=36, PI=19

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid Limit, PI=Plasticity Index

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-3

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/18" AB									
CLAY (CL/CH), black, with silt, trace gravel (subrounded), damp	Very Stiff				30				
					23				
GRAVEL (GC), brown, coarse, subrounded to subangular, clayey, with sand (fine- grained), damp	Dense		5		31				
CLAY (CL), brown, silty, some gravel (fine to coarse, subrounded to subangular), trace sand (fine- grained), damp	Stiff				25	18	109		
			10						
GRAVEL (GW), brown, fine to coarse, subrounded to subangular, sandy (fine- to coarse- grained), damp	Medium Dense				26				
			15						
SAND (SM), brown, fine- to coarse-grained, with gravel (fine to coarse, subrounded to subangular), some silt, moist	Dense				43	▼			Passing #4 Sieve = 61%, #200 Sieve = 17%
			20						
(grading clayey sand at 23 feet)	Medium Dense					▽			Passing #4 Sieve = 66%, #200 Sieve = 15% at 23½ ft
(lens of sandy clay at 24½ feet)					35	23			Passing #200 Sieve = 68%, LL=38, PI=22 at 24½ ft
(grading sandy gravel (fine- grained), moist at 25 feet)			25			30			
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), damp	Very Stiff				38	23	106	10.2	Passing #200 Sieve = 94%, LL=43, PI=28

FUGRO
425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-4

File Name: G:\ENGINEER\GINTWPROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), damp	Very Stiff								
	Hard				40				
SAND (SC), brown, fine- to coarse-grained, some gravel (fine, subangular), trace silt, wet	Dense		35						
(grading some silt and clay at 39 feet)			40		54	16			Passing #4 Sieve = 71%, #200 Sieve = 27%, LL=40, PI=23
			45		59				Passing #4 Sieve = 75%, #200 Sieve = 22%
CLAY (CL), brown, with sand (coarse-grained), damp	Hard		50		31				

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lbdown-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid limt, PI=Plasticity Index

FUGRO
425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-4

File Name: G:\ENGINEER\GINTWPROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	CME 75 HSA	SURFACE ELEVATION	96 feet	LOGGED BY	MHL
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	11/24/97

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT: 2-1/2-inched A.C. over 9-1/2-inches A.B.					50/5"				
FILL: SAND (SW), yellowish-brown, fine-to coarse-grained, some gravel (fine to coarse, angular), trace to some clay, dry	Dense								
	Dense				44	21	94		LL=37%, PI=21%, Passing #200 Sieve=72%
FILL: SAND (SC), yellowish-brown, fine-to-coarse-grained, some clay, trace gravel (fine to coarse, angular), dry-damp	Very Dense		5		51				
FILL: CLAY (CL), dark brown, silty, some sand (fine grained), trace gravel (fine, angular), damp									
CLAY (CL), brown to dark brown, silty, some sand (fine grained), damp	Hard				50/5"	21	107		
(grey with yellowish-red iron staining, with sand (fine-to medium-grained), trace gravel (fine to medium, rounded) below 12 feet)			10		50/6"	18	111		
			15						
SILT (ML), light grey with yellowish-red iron staining, some sand (fine grained), trace pockets of wet clay, damp	Hard				75	20	108		
			20		37				

Bottom of Boring = 20 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. The boring was backfilled with native soil cuttings immediately upon completion.

HARZA
Consulting Engineers & Scientists

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL HELISTOP
Oakland, California

PROJECT NO.	DATE	BORING NO.
L998G	December 1997	EB-1

DRILL RIG	gCME 75 HSA	SURFACE ELEVATION	96 feet	LOGGED BY	MHL
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	11/24/97

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT: 2-inches A.C. over 8-inches A.B.									
FILL: SAND (SC), brown-dark brown, fine-grained, silty, some clay, trace gravel (fine to medium, subangular), trace wood and brick debris, damp	Med. Dense								LL=31%, PI=19%, Passing #200 Sieve=35%
CLAY (CL), dark brown with yellowish-red iron staining, silty, trace to some sand (fine grained), damp	Hard		5		29				
					37				
	Very Stiff				52	21	106		
SILT (ML), grey with yellowish-red iron staining, some sand (fine grained), some clay, damp	Very Stiff		10						
SAND (SC), yellowish-brown, fine-to coarse-grained, some clay, some gravel (fine to medium, subrouned), damp-moist (yellowish-brown with red iron and black staining below 13-1/2 feet)	Med. Dense				22	17	108		
			15		25				

Bottom of Boring = 16-1/2 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. The boring was backfilled with native soil cuttings immediately upon completion.

HARZA
Consulting Engineers & Scientists

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL HELISTOP
Oakland, California

PROJECT NO.	DATE	BORING NO.
L998G	December 1997	EB-2

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	18 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
4 inches of asphaltic concrete, 5 inches of aggregate baserock									
SILT (ML), dark brown, clayey, some sand (fine-grained), trace rootlets, dry	Stiff		5		21	16	99		PI=19%, LL=36%, Passing #200 Sieve=72%
(coarse-grained sand apparent, trace gravel (fine, subangular) at 9-1/2 to 11 feet)	Very Stiff		10		39 35				PP=9.0+ ksf, TV=4.5+ ksf
CLAY (CL), olive gray with rust mottling, silty, moist	Very Stiff		15		32	22	103		PP=6.5 ksf, TV=2.8 ksf
	Stiff		20		23				PP=3.0 ksf, TV=2.3 ksf
SAND (SC), brown, fine- to coarse-grained, with clay, and gravel (fine, subangular), wet	Medium Dense		25		30 28				
CLAY (CL), rust brown to brown, with sand (fine- to coarse-grained), trace gravel (fine, angular to subrounded), moist	Hard		30		38				

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-3

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	18 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), (continued) (sandy at 34-1/2 to 36 feet)	Hard		40		93/11"				
(sand appears cemented below 40 feet)			45						
			50		89				
Bottom of Boring = 51 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 24 feet at time of drilling. After completion of drilling, ground water was measured at 18 feet. 4. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot. 5. PI = Plasticity Index, LL = Liquid Limit.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-3

Sept. 2008 Report ID: PB-3 (94)

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	20 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
6 inches asphaltic concrete, 7 inches aggregate baserock									
CLAY (CL), dark brown to black, silty, damp	Very Stiff		5		30				PP=9.0 ksf, TV=4.3 ksf
(brown, plasticity decreases below 9-1/2 feet)			10		36				PP=9.0 ksf, TV=4.5+ ksf
CLAY (CL), olive gray with rust mottling, silty, moist	Very Stiff		15		24 15	23	102		PP=8.0 ksf, TV=4.3 ksf PP=8.0 ksf, TV=2.5 ksf
(trace sand (fine- to coarse-grained) and gravel (fine, subrounded) at 19-1/2 to 20 feet)			20		32				
SAND (SC), brown, fine- to coarse-grained, with gravel (fine, subangular to subrounded), trace clay, wet	Medium Dense		25		26				
(some clay at 28 to 28-1/2 feet)									
CLAY (CL), rust brown, with gray mottling, some sand (fine- to coarse-grained), moist	Hard		30		59				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-4

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	20 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL) (continued)	Hard		40		65				
(cemented at 38 to 30-1/2 feet)			45						
(sand not apparent at 48-1/2 to 50 feet)			50		97/11"				
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 22 feet at time of drilling. After completion of drilling, ground water was measured at 20 feet. 4. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-4

Sept. 2008 Report ID: PB-4 (94)

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	101 Feet	LOGGED BY	CDT/MJ
DEPTH TO GROUND WATER	22 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/3/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), dark brown, silty, trace rootlets	Stiff								
SAND (SM), brown, fine- to coarse-grained, with gravel (fine, subangular to subrounded), trace silt, damp	Loose		5		15				
	Medium Dense		10		29				
CLAY (CL), brown, with sand (fine- to coarse-grained), trace silt, moist	Firm		15		5				
	Stiff				14	19	107	1.1	TV=2.5 ksf
(tan-gray with rust mottling, damp at 18-1/2 to 20 feet)	Stiff to Very Stiff		20		26				PP=4.0 ksf, TV=3.0 ksf
(moist to wet at 23-1/2 to 25 feet)	Stiff		25		17				
SAND (SM), tan-gray, fine- to medium-grained, some silt, wet	Medium Dense		30		13				Passing #200 Sieve=25%
(trace gravel (fine, subangular) at 33-1/2 to 35 feet)	Dense				45				

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Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-1

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	101 Feet	LOGGED BY	CDT/MJ
DEPTH TO GROUND WATER	22 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/3/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
SAND (SM) (continued)	Dense		40		14				
CLAY (CL), gray, silty, damp to moist	Stiff		45						
(with sand (fine- to medium-grained at 48-1/2 to 50 feet)	Hard		50		58				PP=9.0 ksf, TV=5.0 ksf
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 27 feet at time of drilling. 4. An observation well was installed in this boring to periodically measure the ground water depth. 5. Ground water was measured at 22 feet on 9/22/92. 6. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-1

Sept. 2008 Report ID: PB-1 (94)OW

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	97 Feet	LOGGED BY	MJ
DEPTH TO GROUND WATER	16 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/4/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
8 inches asphaltic concrete, 4 inches aggregate baserock									
CLAY (CL), dark brown, silty, trace gravel (fine, subangular), damp	Very Stiff								
(red-brown with gray mottling, trace sand (fine-grained) at 4-1/2 to 6 feet)			5		24				
			10		19				
SAND (SC), red-brown, fine- to coarse-grained, with gravel (fine, subangular), moist	Medium Dense								
(gravelly, wet at 13 to 14-1/2 feet)	Dense				35				
			15			▼			
(fine-grained sand layer at 17-1/2 feet)	Medium Dense				20				Passing #200 Sieve = 36%
			20						
					24				
			25						
CLAY (CL), tan-brown with gray mottling, trace sand (fine-grained), moist	Very Stiff				29				
			30						
(with sand at 28 to 29-1/2 feet)	Hard				55				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-6

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	97 Feet	LOGGED BY	MJ
DEPTH TO GROUND WATER	16 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/4/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL) (continued)	Hard								
SAND (SC), red-brown, fine- to coarse-grained, with clay, trace gravel (fine, angular), damp	Very Dense		40		84				Passing #200 Sieve = 21%
			45						
	Dense		50		74				
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 17-1/2 feet at time of drilling. 4. An observation well was installed in this boring to periodically measure the ground water depth. 5. Ground water was measured at 16 feet on 9/22/92.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-6

Sept. 2008 Report ID: PB-6 (94)OW

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Enc.	BORING DIAMETER	5-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
3" Asphaltic Concrete									
FILL: CLAY (CL), brown mottled rust and blue, silty, with sand (fine- to coarse-grained), some gravel (fine- to coarse-grained), trace organics	Firm				14*				PP = 1.5 tsf
	Stiff				13				
(less gravel and organics)			5						
(light brown, no organics)			10		25*	18	108	2.9	PP = 3.5 tsf
CLAY (CL), light brown mottled rust, with sand (fine- to coarse-grained), some silt, some gravel (fine-grained)	Stiff		15		16*	18	109	1.5	PP = 1.7 tsf
SAND (SM), light brown mottled gray, fine- to coarse-grained, gravelly (fine-grained), some silt and clay	Medium dense		20		29*				
Bottom of Boring = 20-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not encountered at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	1

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2" Asphaltic Concrete over 2" Gravel									
FILL: CLAY (CL), light brown mottled blue-green and black, silty, with sand (fine- to coarse-grained), trace gravel (fine-grained), trace organics	Firm				15*	18	102		PP = 1.5 tsf
	Stiff		5		10				
(dark brown mottled rust and gray, some gravel (fine-grained))									
(brown mottled rust and gray, with sand (fine- to coarse-grained), some gravel (fine- to coarse-grained))	Firm		10		9*	16	107	0.9	PP = 1.0 tsf
GRAVEL (GM), gray-brown, fine-grained, some sand (fine- to coarse-grained), trace silt and clay	Medium dense		15		21*				
CLAY (CL), tan and rust, some silt, trace sand (fine-grained)	Firm		20		7				
(some sand (fine- to coarse-grained), trace gravel (fine-grained))			25		17*	23	102	2.5	PP = 1.2 tsf
(silty, some gravel (fine-grained))	Stiff		30		27*				PP = 1.7 tsf
Bottom of Boring = 30-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not established at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	2

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Enc.	BORING DIAMETER	5-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2-1/2" Asphaltic Concrete over 4-1/2" Gravel									
FILL: CLAY (CL), brown, silty, trace sand (fine- to coarse-grained), trace gravel (fine-grained)	Firm				15*	20	90		PP = 2.0 tsf
(some sand (fine- to coarse-grained), more gravel)	Stiff		5		10				
(brown mottled rust and gray, some gravel (fine-grained))			10		38*	17	107	3.4	PP = 4.0 tsf
CLAY (CL), brown, silty, trace sand (fine- to medium-grained)	Stiff				248				
GRAVEL (GM), brown mottled rust and gray, fine- to coarse-grained, some sand (fine- to coarse-grained), trace silt and clay	Medium dense		15						
			20		37*				
Bottom of Boring = 20-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not encountered at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	3

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2" Asphaltic Concrete									
FILL: CLAY (CL), black and blue-green, silty, some sand (fine- to coarse-grained), trace gravel (fine-grained)	Stiff				23*	19	102		PP = 3.2 tsf
(brown mottled rust and gray, some gravel (fine- to coarse-grained), trace organics)			5		9				
(with sand (fine- to coarse-grained), trace gravel (fine- to coarse- grained), no organics)			10		25*	13	110	2.0	PP > 4.5 tsf
(sandy (fine- to coarse-grained))			15		8				
GRAVEL (GM), gray, fine-grained, trace sand (fine- to coarse-grained), trace silt and clay	Loose		20		6				
SILT (ML), tan mottled rust, sandy (fine-grained), trace clay, trace sand (medium- to coarse-grained)	Stiff		25		19*	28	95	2.8	PP = 1.5 tsf
(less sand)			30						

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	4

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SILT (ML), tan mottled rust, with sand (fine-grained), trace clay, trace sand (medium- to coarse-grained)	Stiff								
	Firm		35		14*	24	100		PP = 1.2 tsf
(same as above)									
			40						
(same as above)	Stiff								
CLAY (CL), orange-brown mottled black and rust, some sand (fine-grained), trace sand (medium- to coarse-grained)	Hard		45						
			50		62*				PP > 4.5 tsf
Bottom of Boring = 50-1/2 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A. 3. The groundwater level was not established at the time of drilling. 4. PP = Pocket Penetrometer 5. The boring was backfilled with cuttings and capped with asphaltic concrete.									

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	4

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 97 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	18' (See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 4" Baserock									
SAND, (fine-grained), clayey, with silt, traces of gravel (fine-grained) Liquid Limit = 30% Plasticity Index = 12% Passing #200 Sieve = 49%	dark grey	medium dense	SC-CL	1					
				2		30	17		
		loose		3		13*			
SILT, sandy, (fine-grained), trace of gravel, (fine grained) Passing #200 Sieve = 54%	tan	stiff	ML	4					
				5		9	24		
CLAY, silty, some sand, (fine to coarse-grained)	grey	stiff	CL	6					
				7					
				8					
(gravel fine grained more sand)				9					
Liquid Limit = 33% Platicity Index = 33% Passing #200 Sieve = 51%	mottled with tan	very stiff	CL	10		42*	14	114	10.5
				11					
				12					
				13					
(grading more gravel, fine-grianed)	light grey			14					
				15		34*	24	101	8.6
SAND, (fine-to coarse-grained) silty, trace of gravel, (fine grained), some clay	light grey mottled with tan	medium dense	SM	16					
				17					
				18			▽		
				19					
Passing #200 Sieve = 51%			SM-ML	20		47*			

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EXPLORATORY BORING LOG

NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K 339-7	December 1987	1

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION ± 97 Feet | LOGGED BY LG

DEPTH TO GROUNDWATER 18' (See Note | BORING DIAMETER 8 Inches | DATE DRILLED 12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse-grained), some clay and gravel (fine-grained)	light grey	medium	SC	21		47*			
GRAVEL, (fine - grained), sandy, (fine-to coarse-grained), some clay and silt	brown	medium dense	GC	22 23 24					
CLAY, silty, sandy (fine-to coarse-grained), with gravel (fine grained)	grey mottled with tan	stiff	CL	25		30*	20		
GRAVEL (fine-grained), sandy, (fine-to coarse-grained), some silt and clay	tan	very dense	GC	26 27 28 29 30		94* / 11	16	119	12.0
Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see frist page Appendix A. 3. Groundwater level was measured at a depth of 20 feet at time of drilling. 3½ hours later the groundwater level was measured at 18 feet.				31 32 33 34		85			
(sandy clay lense)				35 36 37 38 39 40		72	18		

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	1

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION ± 94 Feet | LOGGED BY LG

DEPTH TO GROUNDWATER 19½' (See Note 3) | BORING DIAMETER 8 Inches | DATE DRILLED 12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 4" Baserock									
CLAY, silty gravelly (fine-grained), some sand (fine-to coarse-grained)	dark grey	very stiff	CL	1 2		57*	19	114	
GRAVEL (fine-grained) sandy, (fine-to coarse-grained), with clay Passing #200 Sieve = 17%	grey	medium dense	GC	3 4		28*			
CLAY, silty some sand (fine-grained) traces of gravel (occasional gravel lenses)	dark grey	very stiff	CL	5 6 7 8		17			
(grading gravelly, fine-grained)	mottled with tan			9 10 11 12 13 14		58*	17		
(grading with traces of gravel fine grained)				15 16 17		47*	21		
SAND, (fine-to coarse-grained), some silt, some clay	brown	medium dense	SC-SM	18 19					
(occasional clay lense)				20		28*	22	101	

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, Calfiornia

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	2

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 94 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	19½' (See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse grained), gravelly, (fine-grained), with silt, some clay (occasional clay lense) Passing #200 Sieve = 38%	brown	medium dense	SC-SM	21–25		28			
CLAY, sandy, (fine to coarse-grained), with gravel (fine grained), silty (occasional sand or gravel lense)	light grey	firm	CL-SC	25–30		16*	22	104	0.5
(grading without gravel, less sand) Liquid Limit = 30% Plasticity Index = 16% Passing #200 Sieve = 79%		stiff	CL	30–35		20*	27	96	0.6
SAND, (fine to coarse-grained) some clay and silt, trace of gravel (fine-grained)	brown	dense	SC	35–40		39*	29		

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	2

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 94 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	19½' (See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse grained) some silt, trace of gravel (fine-grained) (occasional gravel lense)	brown	medium dense	SC	41–45		39*			
(grading gravelly)						68			
CLAY, silty, with sand (fine to coarse-grained) (trace of gravel)	light grey	hard	CL	46–49					
CLAY, silty, sandy (medium-grained)	rust	hard	CL	50		58*/6"	21		
Bottom of Boring = 51 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3, Groundwater level was measured at a depth of 19½ feet at time of drilling.				51–60					

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	2

Sept. 2008 Report ID: PB-2 (87)

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	--	LOGGED BY	F.M.
DEPTH TO GROUNDWATER	23' (SeeNote3)	BORING DIAMETER	8 Inches	DATE DRILLED	6/22/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 7" BASEROCK				1					
CLAY, silty with some sand (fine- to coarse grained)	mottled brown-black	stiff	CL	2		10	12		
CLAY, silty, sandy (fine-grained) with chunks of masonry pipe	mottled browns-blacks	stiff	CL-ML	3		10	9		
Dry Density = 104 pcf				4					
				5		20*			>5.0
(Possible Fill)				6					
				7					
CLAY, very sandy (fine to coarse grained) with gravel to ½"	mottled browns-black	hard	CL	8					
				9		78*		1.8	3.5
Liquid Limit = 35%				10					
Plasticity Index = 21%				11					
Passing #200 Sieve = 55%				12					
Dry Density = 115 pcf				13					
				14		36*			
				15					
				16					
				17					
				18					
SAND (fine to coarse grained) with clay and gravel to ½"	mottled brown-grey-black	medium dense	SC	19		23	23		
Passing #200 Sieve = 16%				20					

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EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

Sept. 2008 Report ID: PB-6 (78)

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	--	LOGGED BY	F.M.
DEPTH TO GROUNDWATER	23' (SeeNote3)	BORING DIAMETER	8 Inches	DATE DRILLED	6/21/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (fine to coarse grained) with clay and gravel to ½" (continued)	mottled brown-grey-black	medium dense	SC	21					
				22					
				23			▽		
SILT, clayey with sand (fine-grained)	mottled tan-rust	stiff	ML	24		12	25		
				25					
				26					
CLAY, silty, sandy (fine to medium grained)	blue-green	stiff	CL	27					
				28					
				29		12	19		
				30					
				31					
				32					
				33					
SAND (fine to coarse grained) gravelly to 1", with some clay	mottled brown-grey-black	dense	SC-GC	34		push 500-1000 psf	13		
				35					
				36					
				37					
				38					
(increasing clay content)				39		32	13		
Passing #200 Sieve = 19%				40					

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EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

Sept. 2008 Report ID: PB-6 (78)

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION -- | LOGGED BY F.M.

DEPTH TO GROUNDWATER 23' (SeeNote3) | BORING DIAMETER 8 Inches | DATE DRILLED 6/22/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (fine to medium grained) gravelly to 1" with some clay (continued)	mottled browns-black-red	dense	GC	41–46		52	16		
CLAY, silty with some sand (fine to medium grained)	mottled red-brown-black	hard	CL	47–49		46	20		
Bottom of Boring = 50 Feet				50					

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see page A-1.
3. Groundwater level measured at time of drilling.

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EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

Sept. 2008 Report ID: PB-6 (78)

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, sandy Liquid Limit = 22% Plasticity Index = 19% Passing #200 Sieve = 62%		dark brown	firm	CL	1		5	26		
(grading sandier with depth)					2–4					
Liquid Limit = 36% Plasticity Index = 17%			stiff		5–6		12	19		
CLAY, sandy, gravelly Dry Density = 105 pcf		tan	stiff	CL	7–11			22		1.3
(grading less gravelly with depth)					12–16		11	21		
CLAY, silty		tan	stiff	CL	17–20					

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Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

Sept. 2008 Report ID: PB-1 (74)

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty Dry Density = 102 pcf		tan	stiff	CL	21–23			24	0.6	1.2
SAND (well graded), gravelly, clayey Passing #4 Sieve = 70% Passing #50 Sieve = 29% Passing #200 Sieve = 15%		tan	medium dense	SC	24–26		19			
CLAY, silty Dry Density = 104 pcf		mottled tan & brown	very stiff	CL	29–32			29		6.1
SAND (well graded), gravelly		tan	medium dense	SW-SP	33–35		19	19		
CLAY, silty		tan	very stiff	CL	36–39					
SAND (well graded), clayey, gravelly		tan	dense	SC	40					

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO. K938-8-1421 | DATE February 1994 | BORING NO. PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (well graded), clayey, gravelly Passing #4 Sieve = 78% Passing #50 Sieve = 41% Passing #200 Sieve = 26%		tan	dense	SC	41–44		34			
CLAY, sandy, gravelly Passing #200 Sieve = 57% Dry Density = 111 pcf		tan	hard	CL	45–47			20		9.1
GRAVEL (well graded), sandy		brown	dense	GW	48–49					
CLAY, silty, sandy Liquid Limit = 35% Plasticity Index = 18%		tan	very stiff	CL	50–53		31	17		
SAND (well graded), clayey, gravelly Passing #4 Sieve = 78% Passing #50 Sieve = 42% Passing #200 Sieve = 29%		brown	very dense	SC	54–56		70			
CLAY, silty, sandy		tan	hard	CL	57–60					

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO. K938-8-1421 | DATE February 1994 | BORING NO. PB74-1

Sept. 2008 Report ID: PB-1 (74)

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/10/73

DESCRIPTION AND CLASSIFICATION					DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE BLOWS PER FT.	WATER CONTENT %	SHEAR STRENGTH BY TORVANE TSF	UNCONFINED COMPRESSIVE STRENGTH TSF
DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE						
CLAY, silty, sandy Dry Density = 124 pcf		tan	hard	CL	61					
SAND (well graded), clayey, gravelly		tan	dense	SC-SW	62		15			
					63					
CLAY, silty, sandy		brown	hard	CL	64					
Dry Density = 104 pcf					65					
					66			24		4.7
					67					
					68					
					69					
					70					
Dry Density = 113 pcf					71			19		7.7
					72					
					73					
					74					
					75					
					76					
(grading gravelly with depth)					77					
					78					
					79					
					80					

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/10/73

DESCRIPTION AND CLASSIFICATION					DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE BLOWS PER FT.	WATER CONTENT %	SHEAR STRENGTH BY TORVANE TSF	UNCONFINED COMPRESSIVE STRENGTH TSF
DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE						
CLAY, silty, sandy, gravelly		brown	hard	CL	81		73			
					82					
(grading less gravelly with depth)					83					
					84					
					85					
Dry Density = 105 pcf					86			23		5.7
					87					
					88					
					89					
(grading stiffer with depth)					90					
					91					
					92					
					93					
					94					
					95					
Dry Density = 107 pcf					96			21		6.9
					97					
					98					
					99					
					100					

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

Sept. 2008 Report ID: PB-1 (74)

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND CLASSIFICATION					DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT, %	SHEAR STRENGTH BY TORVANE KSF	UNCONFINED COMPRESSIVE STRENGTH KSF
DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE						
CLAY, silty, sandy		brown	hard	CL	101					
SAND (well graded)		brown	very dense	SC	102					
Passing #200 Sieve = 40%					103					
					104					
					105		85/11"			
					106					
					107					
					108					
CLAY, silty, sandy		brown	hard	CL	109					
					110					
					111					
					112					
					113					
					114					
(grading sandier with depth)					115					
Dry Density = 109 pcf					116			20		7.3
					117					
SAND (well graded), clayey		brown	very dense	SC	118					
					119					
					120					

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EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND CLASSIFICATION					DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT, %	SHEAR STRENGTH BY TORVANE KSF	UNCONFINED COMPRESSIVE STRENGTH KSF
DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE						
SAND (well graded), clayey		brown	very dense	SC	121					
CLAY, silty		tan	hard	CL	122					
					123					
					124					
					125					
					126					
Dry Density = 112 pcf					127			19		14.0
					128					
(sandy between depths of 129 and 130 feet)					129					
					130					
					131					
					132					
					133					
					134					
					135					
Dry Density = 104 pcf					136			23		6.8
					137					
					138					
					139					
					140					

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

Sept. 2008 Report ID: PB-1 (74)

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYM-BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH KSF
CLAY, silty		tan	hard	CL	141					
					142					
					143					
(grading sandier with depth)					144					
					145					
CLAY, silty, sandy		dark brown	hard	CL	146					
					147					
					148					
Dry Density = 114 pcf					149			19		12.8
					150					
Bottom of Boring = 150 Feet										

Notes:

(1) The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

(2) The groundwater level was not established because of the use of bentonite during drilling.

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

Sept. 2008 Report ID: PB-1 (74)

APPENDIX B
LABORATORY TESTING PROGRAM

APPENDIX B
LABORATORY TESTING PROGRAM

The laboratory-testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. A summary of the laboratory tests performed is included as Plate B-1.

The natural water content was measured on five samples recovered from the borings in accordance with ASTM Test Designation D-2216. Water contents are recorded on the boring logs at the appropriate sample depths.

Dry density was measured on four samples of the subsurface soils in accordance with ASTM Test Designation D-2937. The results of these tests are shown on the boring logs at the appropriate sample depths.

Atterberg limits were performed on two samples of the subsurface soils to estimate the range of water content over which these materials exhibit plasticity. The Atterberg limits were performed in accordance with ASTM Test Designations D-428 and D-424. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of these tests are presented on Plate B-2a, and on the logs of the borings at the appropriate sample depths. Results of Atterberg limit test from pertinent historic borings are included on Plate B-2b.

The percent passing the No. 200 sieve was measured on three samples of the subsurface soils to aid in the classification of these soils. These tests were performed in accordance with ASTM Test Designation D-1140. The results of these tests are shown on the boring logs at the appropriate sample depths.

Sieve analyses were performed on three samples of the subsurface soils to aid in the classification of these soils. This test was performed in accordance with ASTM Test Designation C136. The results of these tests are presented on Plate B-3.

Miniature vane tests were performed on three samples of the subsurface soils to estimate the undrained shear strength and residual strength of these layers to provide data for bearing capacity and foundation recommendations. These tests were performed in accordance with ASTM Test Designation D4648. The results of these tests are presented on the boring logs at the appropriate sample depths.

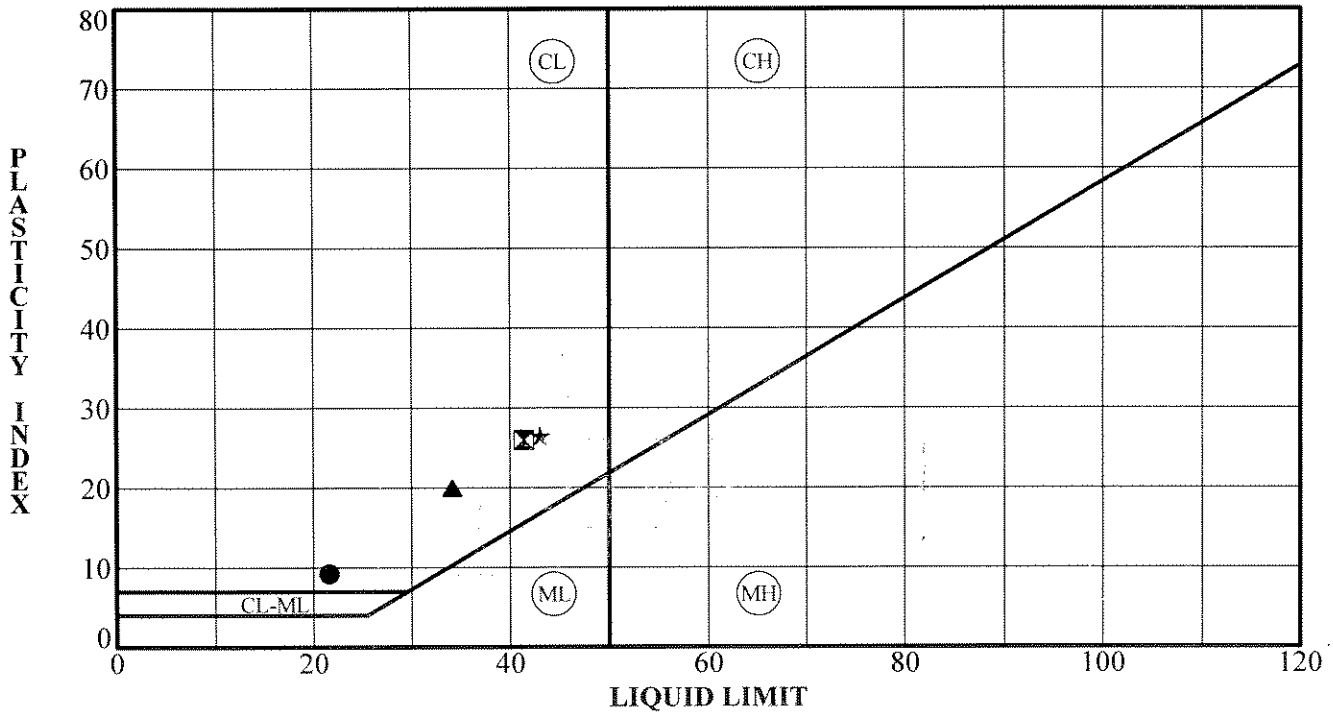
Two Resistance (R-value) tests were performed on representative samples, from Fugro's previous 2008 investigation, of the surface soils onsite to provide data for pavement design. The tests were performed in accordance with California Test Method 301-F and indicated a typical R-value of 4 and 9 at an exudation pressure of 300 pounds per square inch. The results of the tests are presented below:

R-value Test Results – Bulk Sample 1 (B-102)

Description of Material	Dry Density (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
Dark Brown fat CLAY with sand (CH)	92.1	27.7	223	0	2
	93.1	26.5	286	0	3
	94.9	25.4	342	0	8
R-Value = 4 at Exudation pressure of 300 psi					

R-value Test Results – Bulk Sample 2 (B-106)

Description of Material	Dry Density (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
Dark Brown lean to fat CLAY with sand (CL-CH)	100.6	21.8	231	0	5
	101.5	20.7	278	0	7
	105	19.5	334	0	14
R-Value = 9 at Exudation pressure of 300 psi					



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	B-113	2.5	22	9	-0.691	6		
⊠	B-114	8.5	41	26	---	---		
▲	B-114	15.0	34	19	---	---		CL
★	B-116	3.0	43	26	0.032	17		CL

FUGRO

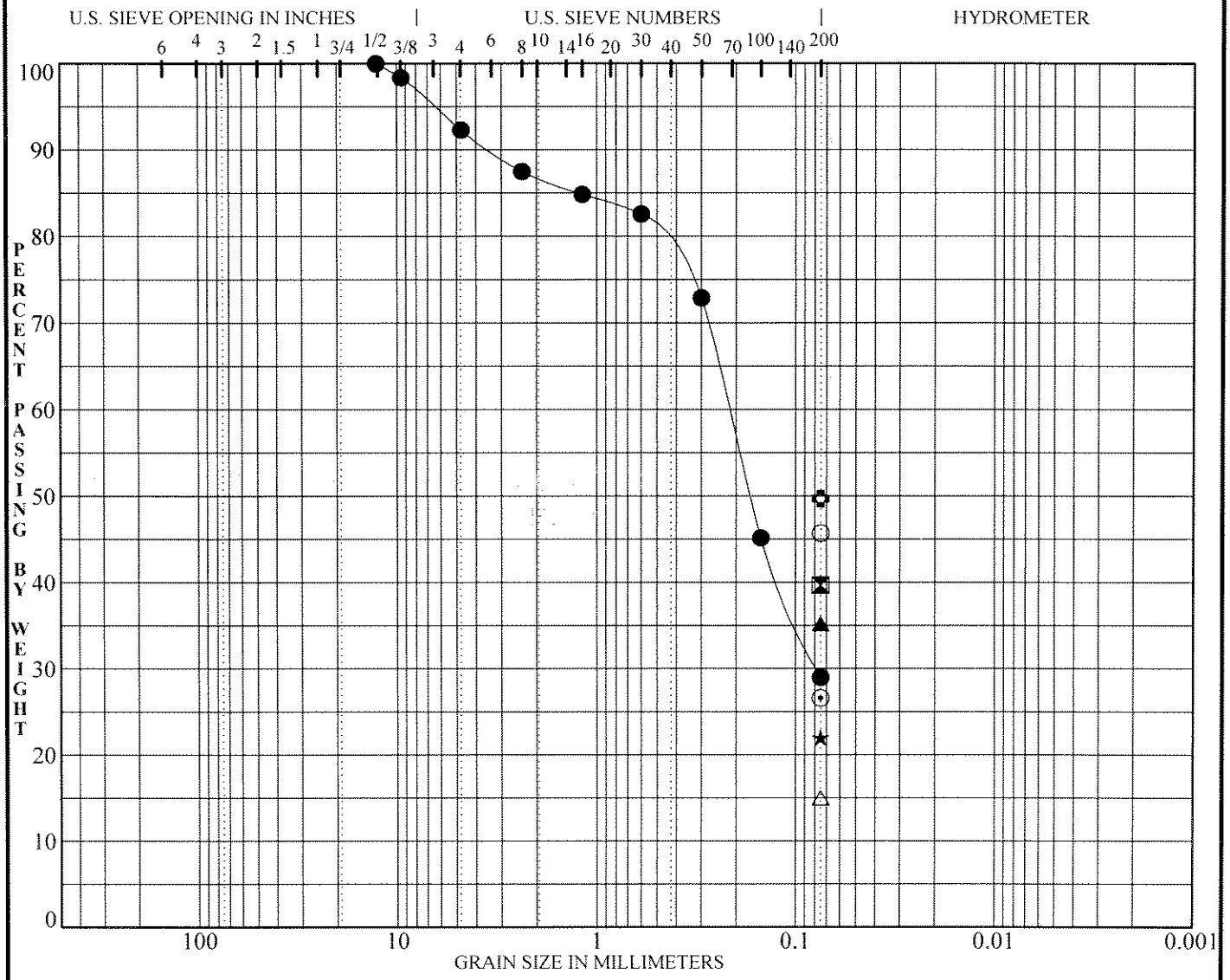
PREP'D BY: RV
APP'D BY: M. Walker
DATE: 8/21/08
DWG FILE:

PLASTICITY CHART AND DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-1
PROJECT No.
1595.002

ATTERBERG 1595.002.GPJ VERSION-012505.GDT 8/21/08



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	B-113	21.0	29	92	Brown silty SAND	SM
⊠	B-113	25.0	40			
▲	B-115	21.0	35		Clayey SAND with gravel	SC
★	B-115	23.0	22		Clayey SAND with gravel	SC
⊙	B-115	25.0	27		Clayey SAND with gravel	SC
✿	B-116	9.0	50			
○	B-116	15.5	46		Clayey SAND (SC)	SC
△	B-116	29.0	15			

FUGRO

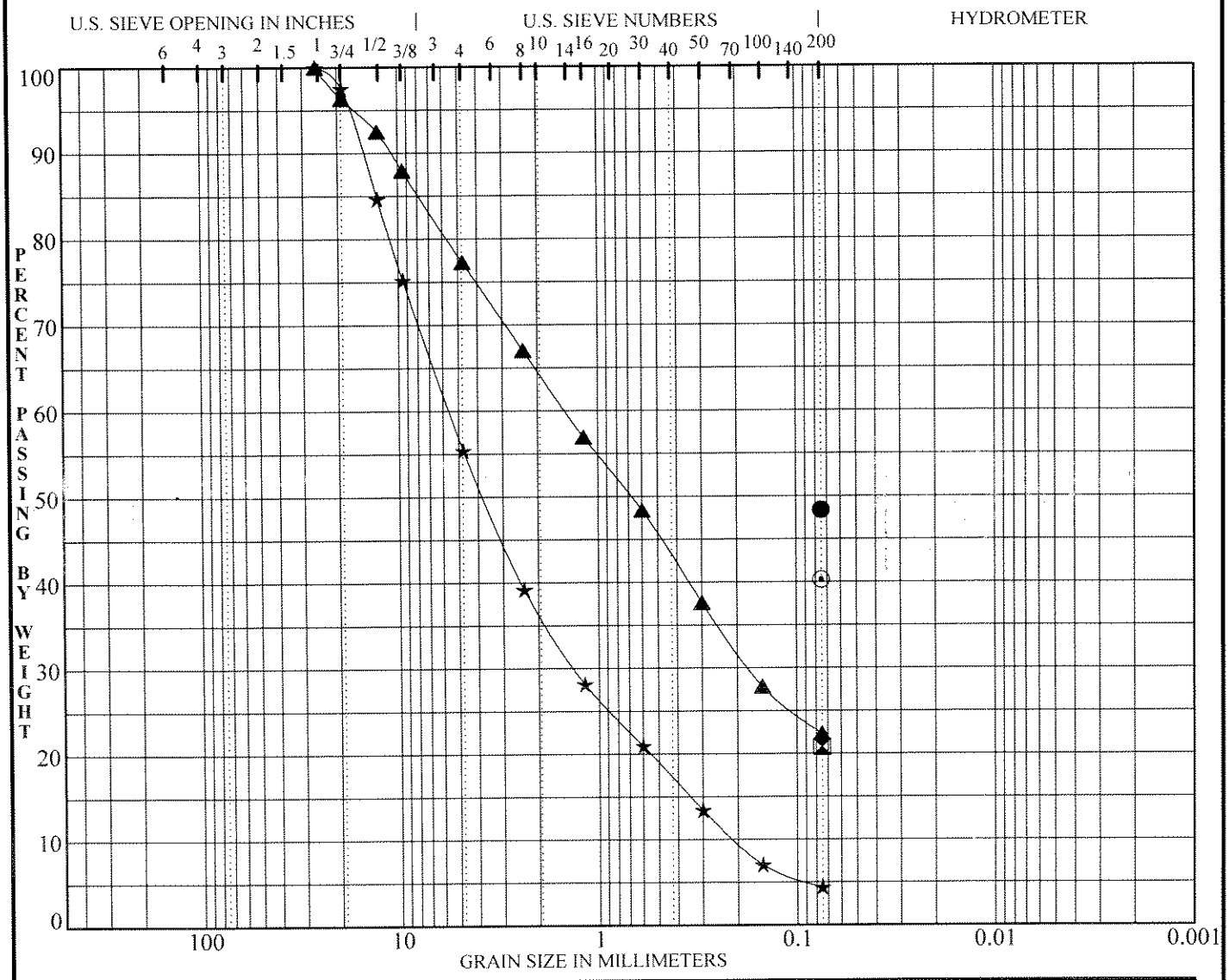
PREP'D BY: RV
APP'D BY: M. Walker
DATE: 8/21/08
DWG FILE:

GRADATION TEST DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-2a
PROJECT No.
1595.002

GRADATION_B 1595.002.GPJ STD.GDT 8/21/08



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	B-117	5.0	48		Brown clayey SAND	SC
⊠	B-117	26.0	21		Yellowish brown clayey SAND	SC
▲	B-117	30.0	22	77	Brown clayey SAND with gravel	SC
★	B-118	8.5	4	55	Brown well-graded SAND with gravel	SW
⊙	B-118	26.0	40		yellowish brown clayey SAND	SC

FUGRO

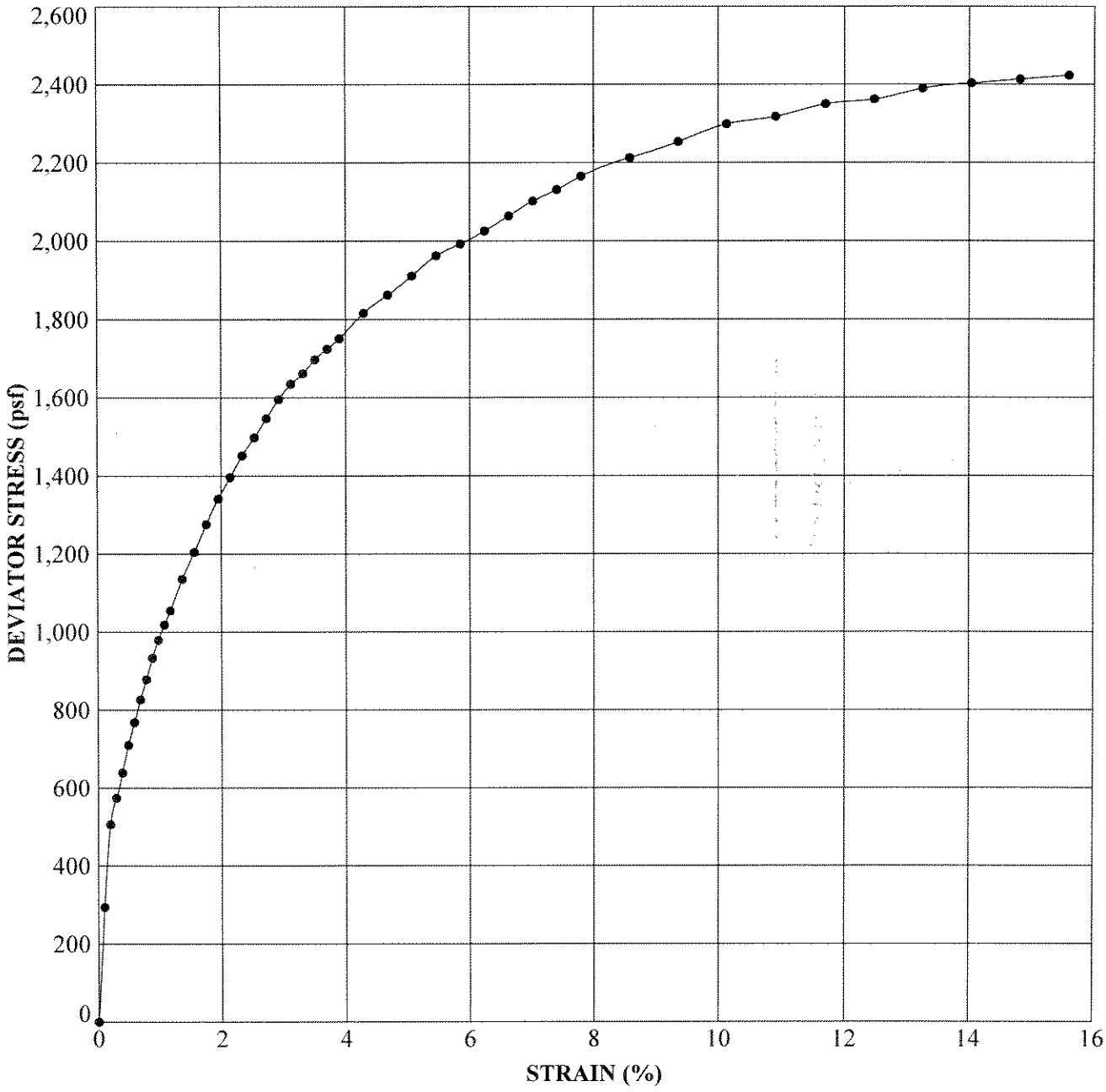
PREP'D BY: RV
APP'D BY: M. Walker
DATE: 8/21/08
DWG FILE:

GRADATION TEST DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-2b
PROJECT No.
1595.002

GRADATION_B_1595.002.GPJ_STD.GDT_8/21/08



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Peak Deviator Stress (psf)	Strain (%)
●	B-116	5.0	Dark yellowish brown sandy lean CLAY (CL)	110.5	17.6	2,423	15.0

FUGRO

PREP'D BY	RV
APP'D BY	M. Walker
DATE	8/21/08
DWG FILE	

TRIAXIAL COMPRESSION TEST DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-3
PROJECT No.
1595.002

TRIAXIAL 1595.002.GPJ STD.GDT 8/21/08

California State Certified Laboratory No.2153

C E R C O
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

19 March 2008

Job No.0803031
Cust. No.11608

Mr. Kap Gupta
Fugro West, Inc.
1000 Broadway, Suite 440
Oakland, CA 94607

Subject: Project No.: 1595.002
Project Name: Oakland – Childrens Hospital
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. Gupta:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on March 05, 2008. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, all samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from none detected to 52 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration ranges from none detected to 110 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 5.6 to 7.5 which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potentials range from 410 to 450-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No.2153

CERCO analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

Client: Fugro West
Client's Project No.: 1595.002
Client's Project Name: Oakland - Childrens Hospital
Date Sampled: Not indicated
Date Received: 5-Mar-08
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 19-Mar-2008

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0803031-001	B-104/1 @ 2.5'	420	6.8	-	1,300	-	52	110
0803031-002	B-105/6 @ 15.5'	450	7.5	-	1,700	-	N.D.	N.D.
0803031-003	B-112/3 @ 4'	410	5.6	-	1,800	-	N.D.	33

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	11-Mar-2008	11-Mar-2008	-	13-Mar-2008	-	11-Mar-2008	11-Mar-2008

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel:925.462.2771
Fax:925.462.2775

CERCO analytical, inc.

Chain Of Custody

Page of

Job No. 080203 | Client No. 11608 | Client Project I.D. 1595.002

Sample Schedule: WK MO QT AN Resample New: | Date Sampled | Date Due

Contact Name: Kap Gupta

Company Name: Fugro West Oakland

Sample Source: Oakland - Childrens Hospital

Lab No.	Sample I.D.	Date	Time	Matrix	Container	Preserv.	Qty.	ASTM Test Methods* with Brief Evaluation	CalTrans Test Methods** with Brief Evaluation
001	B-104/1 @2.5	N					1	/	
002	B-105/6 @15.5						1	/	
003	B-112/3 @4.0						1	/	

MATRIX: DW - Drinking Water; GW - Ground Water; SW - Surface Water; WW - Waste Water; SL - Sludge; S - Soil

ABBREVIATIONS: HB - Hosebib; PV - Petcock Valve; PT - Pressure Tank; PH - Pump House; RR - Restroom; GL - Glass; PL - Plastic; ST - Sterile

SAMPLE RECEIPT: Total No. of Containers: 3; Rec'd Good Cond./Cold; Conforms to Record; Sampler

Relinquished By	Date	Time
	3/5/08	rd: 10
Received By	Date	Time
	3/5/08	10:08
Relinquished By	Date	Time
Received By	Date	Time
Relinquished By	Date	Time
Received By	Date	Time

Sample Location/Instructions

* Includes Redox, pH, Resistivity (100% Saturation), Chloride and Sulfate

** Includes pH, Minimum Resistivity, Chloride & Sulfate

There may be an additional charge for submitting soil in brass tubes.

APPENDIX C
SEISMIC HAZARD ANALYSIS AND GROUND MOTIONS

APPENDIX C
SEISMIC HAZARD ANALYSIS AND SITE-SPECIFIC GROUND MOTIONS

1.0 INTRODUCTION

This appendix documents the seismic hazard evaluation study consisting of probabilistic seismic hazard analysis (PSHA) and deterministic analysis conducted to update the design ground motion criteria per the 2007 California Building Code (2007 CBC) code for New Hospital Replacement Project – Central Utility Plant for Children's Hospital & Research Center at Oakland (CHRCO). The CHRCO site is located 52nd Street in Oakland, Alameda County, California.

The geotechnical study conducted by Harza Kaldveer (Now Fugro) in 1994 included a seismic hazard evaluation. Since January 2008, California has adopted the 2007 CBC; therefore our seismic hazard evaluation study as summarized in this appendix has been revised to meet the requirements of the 2007 CBC.

Site-specific, PSHA and deterministic seismic hazard analysis (DSHA) have been conducted in accordance with the 2007 CBC requirements to develop design ground motion criteria for a Maximum Considered Earthquake (MCE) and the calculation from the MCE of a design response spectrum.

The scope of work for this aspect of the project consisted of:

- Characterizing the subsurface conditions, based on the available subsurface information;
- Identifying potential seismic sources and fault parameters;
- Performing PSHA to generate acceleration response spectra at the ground surface for horizontal and vertical components; and
- Comparing the PSHA and DSHA results with deterministic limit and code-based simplified spectra to develop the site-specific design spectrum for the project according to the 2007 CBC.

2.0 SUBSURFACE CONDITIONS

The soils encountered across the site in our borings typically consisted of a layer of lean clay fill, over interlayered clayey sands to sandy clays with varying gravel content. The clay fill was observed to a typical depth of about 5 feet. Stiff lean clay was generally encountered between depths of 5 and 20 feet. Below this stiff clay, interlayered clayey sands and sandy clays, as well as some gravel layers, were observed to depths ranging from 24 to 46 feet. The previous deep boring, PB-1 (74), shows a lean clay layer below 50 feet to a depth of 150 feet with two clayey sand layers encountered from 102 to108.5 feet and 118 to 121 feet. The subsurface conditions at the site of each proposed structure are presented in Section 4.3 of this report. The boring logs are presented in Appendix A. In general, the subsurface conditions across the site are fairly similar and consist of fill underlain by alluvial deposits.

Idealized Soil Profile

Idealized soil profile, design unit weight, and shear strength parameters were developed based on the results of the field investigations and laboratory testing conducted by Fugro. The profile is based on the field and laboratory test data from twelve exploratory borings conducted for an investigation.

We have assumed a VS30 value of 270 m/s for the site based on published correlations between shear wave velocity and SPT N-values (Ohta & Goto, 1976, Imai and Tonouchi, 1982). This is consistent with published values of Vs30 for Quaternary alluvium in the San Francisco Bay Area (Wills and Clahan, 2006).

Based on the idealized shear wave velocities, the entire project site is considered to be Site Class "D" per CBC 2007. We note that this takes into consideration the potentially liquefiable zones encountered in borings B-112 (08) and B-113. The effects of the soil softening (and lowering of soil G_{max}) in these zones is expected to have minimal effect on the propagation of seismic waves within the soil profile. Thus, we have characterized the site as Site Class "D" rather than "F" according to the 2007 CBC.

3.0 METHODOLOGY FOR DEVELOPMENT OF DESIGN RESPONSE SPECTRA

The procedure for development of a design response spectrum for this site consisted of the following three steps:

1. Conducting probabilistic seismic hazard analysis (PSHA) using the Next Generation Attenuation (NGA) relationships and in accordance with the 2007 CBC requirements.

2. Applying the factors required by the OSHPD Code Application Notice, dated September 30, 2008, to account for rotation of motion. Factors used can be found in Campbell and Borzorgnia (2007) report, Table 4.3, Median of Maximum Rotated Component ratios.

3. Conducting deterministic seismic hazard analysis (DSHA) to determine the largest mean plus one sigma ($M+\sigma$) acceleration response spectrum from characteristic earthquakes on all known active faults in the region.

4. PSHA and DSHA results were used to develop design ground motion criteria for a Maximum Considered Earthquake (MCE) and a Design Response Spectrum according to 2007 CBC that is applicable at the foundation level.

4.0 PROBABILISTIC SEISMIC HAZARD ANALYSES

Introduction

Probabilistic seismic hazard analyses were performed to evaluate the probability of occurrence of ground motion corresponding to the 2475-year return period event in order to

develop the Maximum Considered Earthquake (MCE) per CBC 2007. The spectral analyses consider site-specific subsurface conditions and the regional seismic setting.

Tectonic/Seismic Setting

The Oakland Children's Hospital is located in the Coast Ranges geomorphic province of California, which is characterized by northwest-southeast trending valleys and ridges. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates and subsequent strike-slip faulting along the San Andreas Fault zone. The potential for surface fault rupture at the site is low. The site lies approximately 2 miles from the closest active fault zone (Hayward fault).

Identification and Characterization of Potential Seismogenic Sources

A potential seismogenic source is one that exhibits historic displacement, historic earthquake activity, or geologic evidence of activity during the latest Holocene (approximately the past 11,000 years). All significant seismogenic sources within 100 km of the site were identified and characterized with respect to geometric and geologic conditions that affect the development of seismic source parameters for use in the development of the earthquake ground motions. The key geometric, geologic and seismologic characteristics that were compiled or estimated from available data for each potential seismogenic source include:

- Style of Faulting (e.g. strike-slip, reverse, normal);
- Closest source-to-site distance;
- Total fault length and rupture length;
- Rupture width;
- Slip rate; and
- Maximum magnitude (M_{max}).

Table C-1 lists the key fault parameters for the sources found within a 25 km radius of the site. These parameters were subsequently used in the seismic hazard analyses and were adapted from the Working Group on California Earthquake Probabilities (WGCEP, 2003).

Table C-1: Geologic Characteristics of Significant Seismogenic Sources within 25 km of the Site

Potential Seismogenic Source [1]	Type [2], [3]	Closest Distance in km (miles)	Slip Rate (mm/yr)[9]	M_{max}[9] [4]
Hayward Rogers Creek (HS)	RL-SS	3.3 (2.1)	9.0	6.67
Hayward Rogers Creek (HN)	RL-SS	3.3 (2.1)	9.0	6.49
Hayward Rogers Creek (HS+HN)	RL-SS	3.3 (2.1)	9.0	6.91
Hayward Rogers Creek (HN+RC)	RL-SS	3.3 (2.1)	9.0	7.11
Hayward Rogers Creek (HS+HN+RC)	RL-SS	3.3 (2.1)	9.0	7.26
Hayward Rogers Creek (FLOATING)	RL-SS	3.3 (2.1)	9.0	6.90
Background	-	-	Activity Rate (M>5.0) 0.45 events/yr	7.25
Calaveras (CN)	RL-SS	20.6 (12.8)	6.0	6.78
Calaveras (CC+CN)	RL-SS	20.6 (12.8)	6.0	6.90
Calaveras (CS+CC+CN)	RL-SS	20.7 (12.9)	6.0	6.93
Calaveras (Floating)	RL-SS	20.7 (12.9)	6.0	6.20
Mt Diablo (MTD)	RL-SS	25.2 (15.7)	2.0	6.65
Concord (CON)	RL-SS	25.2 (15.7)	4.0	6.25
Concord (CON+GVS)	RL-SS	25.2 (15.7)	5.0	6.58
Concord (CON+GVS+GVN)	RL-SS	25.2 (15.7)	5.0	6.71
Concord (FLOATING)	RL-SS	25.2 (15.7)	5.0	6.20
San Andreas (SAP)	RL-SS	25.7 (16.0)	17.0	7.15
San Andreas (SAS+SAP)	RL-SS	25.7 (16.0)	17.0	7.42
San Andreas (SAN+SAP+SAS)	RL-SS	25.7 (16.0)	24.0	7.76
San Andreas (SAP+SAN+SAO)	RL-SS	25.7 (16.0)	24.0	7.83
San Andreas (SAS+SAP+SAN+SAO)	RL-SS	25.7 (16.0)	24.0	7.90
San Andreas (FLOATING)	RL-SS	25.7 (16.0)	24.0	6.90

Notes:
1. Various fault segments and rupture scenarios were considered as described in WGCEP (2003)
2. Fault parameters were adapted from the CGS fault database (2003) and WGCEP (2003).
3. SS = Strike Slip Fault, RL = Right Lateral, R = Reverse (Dip in degrees), O = Oblique
4. M_{max}= Maximum moment magnitude with highest weightage in the seismotectonic model (per WGCEP 2003)
5. SAS =Santa Cruz Mountain, SAP = Peninsula, SAN = North Coast North, SAO = North Coast South
6. SGN = San Gregorio North, SGS = San Gregorio South
7. HN = Hayward North, HS = Hayward South, RC = Rodgers Creek
8. CON = Concord, GVS = Green Valley South, GVN = Green Valley North
9. Only Mean slip rate and M_{max} values are listed here. WGCEP (2003) provides upper- and lower-bound values that were also used with appropriate weighting factors. Those are not listed here for brevity.

Seismic Hazard Analysis Methodology

Probabilistic seismic hazard analyses (PSHA) were carried out using the computer program HAZ38 (Abrahamson, 2007). The program employs the analytical procedure to compute seismic hazard originally developed by Cornell (1968).

The activity of the seismic sources was modeled using the Youngs and Coppersmith (1985) composite model to simulate the relative distribution of magnitudes for each fault. In implementing the model in the hazard code, it was considered that for each fault the mean characteristic magnitude (M_{char}) is the maximum magnitude (M_{max}) tabulated in Table C-1. The composite model includes a characteristic earthquake distribution for the large magnitude earthquakes ($M > M_{char} - 0.25$) and an exponential distribution for smaller magnitude earthquakes ($M \le M_{char} - 0.25$). The slope of the exponential part (i.e. b-value) was assigned a value of 0.8 based on regional seismicity data.

The probability density function of the Youngs and Coppersmith model is given by:

$$f_m^{YC}(M) = \begin{cases} \frac{1}{1+c_2} \cdot \frac{\beta \cdot \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 1.25)\right)}{1-\exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)} & for \quad \overline{M}_{char} - 0.25 \le M \le \overline{M}_{char} + 0.25 \\ \frac{1}{1+c_2} \cdot \frac{\beta \cdot \exp\left(-\beta \cdot (M - M_{\min})\right)}{1-\exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)} & for \quad M_{\min} \le M \le \overline{M}_{char} - 0.25 \end{cases}$$

where

$$c_2 = \frac{0.5 \cdot \beta \cdot \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 1.25)\right)}{1-\exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)},$$

$\beta = b \cdot \ln(10)$, and

$\overline{M}_{char}$ is the mean characteristic magnitude of the fault here taken equal to M_{max}.

The seismic hazard analyses were conducted using the recently developed Next Generation of Attenuation (NGAs) Relationships developed for shallow crustal earthquakes and considered the influence of near-source and rupture directivity effects. These relationships used were developed as part of the PEER NGA program (http://peer.berkeley.edu/). These are:

- Abrahamson and Silva (2008);
- Boore and Atkinson (2007);
- Chiou and Youngs (2006); and,
- Campbell and Bozorgnia (2007).

PSHA Results

Figure C-1 shows the horizontal component conducted for the idealized site average shear wave velocity of 270 m/s. The motion presented is adjusted for motion directivity by applying the factors presented in Campbell and Borzorgnia (2007). The resulting spectrum used for 2007 CBC comparison (not for use in design of hospital structures) is presented in Table C-2.

Table C-2: 2 Percent in 50 Year Rotated Spectrum

Period (seconds)	Median Maximum Rotated Spectrum for Code Comparison
0.01	1.140
0.10	2.101
0.20	2.591
0.30	2.633
0.40	2.646
0.50	2.525
0.60	2.385
0.75	2.171
1.0	1.831
1.5	1.388
2.0	1.073
3.0	0.688
4.0	0.495

Deaggregation of Hazard

Deaggregation of the hazard shows that a majority of the hazard comes from events with magnitudes ranging between 6.5 and 7.5 with distances between 0 and 5 km. This hazard is associated with events on the Hayward-Rodgers Creek fault that lies approximately 3.3 km from the site.

5.0 DETERMINISTIC HAZARD ANALYSIS

The M+σ deterministic spectrum was obtained from an earthquake occurring approximately 3.3 km from the project site with a magnitude of 7.26, which is the maximum M_{max} value for the rupture of the entire Hayward-Rodgers Creek fault. The M+σ deterministic spectrum has then been increased per the OSHPD Code Application Notice directive by the factors presented in Campbell and Borzorgnia (2007) for median maximum rotated spectrum. The resulting spectrum is shown on Figure C-2 and tabulated below.

Table C-3: M = 7.26, r = 3.3km Deterministic Spectrum Rotation Calculation

Period (seconds)	Median Maximum Rotated Spectrum for Code Comparison
0.01	0.831
0.10	1.327
0.20	1.767
0.30	1.888
0.40	1.932
0.50	1.927
0.60	1.826
0.75	1.689
1.0	1.453
1.5	1.085
2.0	0.855
3.0	0.556
4.0	0.410

6.0 DEVELOPMENT OF SPECTRA PER CBC 2007

Site-specific ground motion procedures were undertaken to develop the design spectrum in accordance with CBC 2007 and Chapter 21 of ASCE 7-05. Section 21.2 requires the calculation of the response spectrum for a Maximum Considered Earthquake (MCE) and the design spectrum is calculated as 2/3 of the MCE spectrum.

The MCE is defined both probabilistically and deterministically. The Design Response Spectrum is calculated from the following comparisons of probabilistic MCE and deterministic MCE:

The lesser of:

(1) the 2 percent probability of exceedance in 50 year (i.e. 2475 year return period MCE) probabilistic ground motion calculated with 5 percent damping; and,

(2) The greater of:

a. M+σ deterministic ground motion calculated for 5 percent damping, and,

b. A deterministic lower bound spectrum calculated according to ASCE 7-05 Section 21.2.2.

The deterministic spectrum was calculated using an M 7.4 earthquake occurring approximately 3.3 km from the project site, on the Hayward Fault. The comparison of the MCE using these spectra and the procedures mentioned in ASCE 7-05 is shown on Figure C-3.

Design Response Spectrum

The site-specific MCE spectrum described in this report takes into account the variation of near-surface stratigraphy at the project site. Therefore, the spectrum is applicable directly at the foundation level. The Design Response Spectrum is defined in ASCE 7-05 as:

The greater of:

(1) The site-specific MCE calculated above, and,

(2) 80 percent of the design response section calculated according to ASCE 7-05 Section 11.4.

This comparison and the final Design Response Spectrum are shown graphically on Figure C-4.

Per ASCE 7-05 Section 21.4, the parameters S_{DS} and S_{D1} can be obtained from the site-specific design spectrum. The S_{D1} value is calculated as the larger of: (1) spectral acceleration at 1 second and (2) two times the spectral acceleration at 2 seconds. The S_{DS} value is calculated as the larger of: (1) spectra acceleration at 0.2 second, and (2) 90 percent of the largest spectral acceleration for any period greater than 0.2 second. Based on these criteria, the design parameters for the project are:

S_{DS} = 1.178 and S_{D1} = 1.140.

The ordinates of the calculated spectra are presented in the following table.

Table C-4: Horizontal Response Spectra Ordinates

Period (seconds)	Maximum Considered Earthquake (g)	Design Response Spectrum (g)
0.01	1.140	0.760
0.10	1.500	1.000
0.20	1.767	1.178
0.30	1.888	1.259
0.40	1.932	1.288
0.50	1.927	1.285
0.60	1.826	1.217
0.75	1.689	1.126
1.0	1.453	0.968
1.5	1.085	0.723
2.0	0.855	0.570
3.0	0.556	0.371
4.0	0.410	0.273

7.0 SEISMIC DESIGN CATEGORY

From the mapped values presented in ASCE 7-05 and the 2007 CBC, the site S_S value is 1.73 and the S_1 value is 0.64. Our site-specific study indicates that the S_{DS} value is 1.18 and the S_{D1} value is 1.14. The 2007 CBC §1613A.5.6 indicates that the Seismic Design Category for Occupancy Category I, II, III, and IV structures is "D".

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APPENDIX D
LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

APPENDIX D
LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

INTRODUCTION

This appendix presents a summary of the procedures used to evaluate the potential for liquefaction of soil at the site. The liquefaction evaluations at the site involved the following steps:

1. Evaluating the generalized soil descriptions and laboratory index test results to identify layers that may be potentially liquefiable;
2. Estimating the seismically induced cyclic demands on the potentially liquefiable soil layers;
3. Estimating the liquefaction resistance of the potentially liquefiable soil layers from boring data;
4. Calculating the factor of safety against liquefaction for the potentially liquefiable layers; and
5. Estimating the magnitude of liquefaction induced settlement in layers with a factor of safety less than 1.1

EVALUATION OF POTENTIALLY LIQUEFIABLE SOIL LAYERS

The susceptibility of a granular soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in: a) distribution of grain size, b) soil density, c) aging and cementation, d) fines content, and e) plasticity characteristics of the fines.

Potentially liquefiable layers are identified as relatively loose, submerged, granular sediments. Liquefaction potential is believed to decrease with depth, and current procedures are believed to be overly conservative for depths greater than 20 meters (Davis and Berrill, 1982). Furthermore, aging tends to increase the liquefaction resistance of soil deposits, and Pleistocene soils are considered to have an extremely low potential for liquefaction.

Cohesive materials may also have the potential to liquefy. Generally, if the liquid limit of the cohesive materials is greater than 35 or the soil meets one of the other criteria specified in Seed, et al. (2001), the cohesive layer is not considered to be liquefiable.

Evaluation of the liquefaction susceptibility of coarse-grained layers and deposits was based on the empirical procedure recommended by the National Center for Earthquake Engineering Research (NCEER, 1997) and summarized by Youd *et. al.* (2001). In the NCEER procedure, potentially liquefiable soil strata are identified as those layers that are relatively loose, submerged granular sediments. Soil strata above the assumed groundwater level were considered not susceptible to liquefaction. A design groundwater depth of 8 feet was used in our analyses.

NCEER recommendations were used to compute liquefaction susceptibility using boring SPT data. The available cyclic shear resistance of the potentially liquefiable strata encountered (based on the normalized SPT N-value) was compared to the estimated cyclic shear stress that would be induced by a given earthquake. The estimated factor of safety against liquefaction is the ratio of the available cyclic shear resistance to the induced cyclic shear stress.

The estimated factors of safety against liquefaction at various depths are a function of the estimated overburden pressure at that depth. In general, higher factors of safety against liquefaction are calculated for higher overburden pressures.

For use with the NCEER procedure, sample depths, fines contents, soil unit weights and field SPT N-values were interpreted and summarized from the borings. The SPT data were then normalized to an overburden pressure of 1 atmosphere and corrected for fines content. A hammer efficiency factor of 1.3 was used to correct the field SPT N-values because the SPT sampling procedure used an automatic trip hammer with a higher efficiency than a standard rope and cathead.

For this evaluation, sublayers were identified at each boring location. Typically, the overall stratigraphic description was obtained from the boring logs, and sublayers were assigned based on SPT blow counts and fines contents. Usually, one sublayer was created for each sample obtained in the boring and that layer was assigned the blow count associated with the sample. A fines content values was assigned to each of those sublayers on the basis of either laboratory tests performed on the sample or the nearest fines content measurement considered representative of the sublayer. Unless bounded by one of the stratigraphic layers shown on the boring logs or the estimated groundwater table, sublayer boundaries were selected midway between adjacent samples.

The seismic hazard analyses for the project (Appendix C) have defined the following design basis earthquake (DBE) values:

- A magnitude 7.26 event on the Hayward fault
- A peak horizontal ground acceleration (PGA) of 0.76g

SUMMARY OF RESULTS AND CONCLUSIONS

The subsurface soils encountered in the borings consist primarily of fine-grained sediment (clay and sandy clay), and have a low susceptibility to liquefaction and dynamic densification. The clayey sand layers contain a significant amount of fines and are typically dense with corrected SPT N-values, (N1)60, greater than 30. These clayey sand layers are unlikely to liquefy during a design seismic event, based on our analyses. However, the liquefaction analyses did identify two isolated, thin layers (B-115 at the new hospital and B-112 at the CUP) that may be susceptible to liquefaction and liquefaction-induced settlement during a design. Our analyses also suggest that isolated liquefaction could occur at these two locations (B-109 and B-112) during a magnitude 7.26 earthquake with a PGA greater than about 0.3g.

The calculation spreadsheets for estimating the liquefaction potential based on the borings are attached in this appendix.

The potential for significant liquefaction and liquefaction-induced settlements at the site is considered low based on the evaluation of the boring data. The underlying soils are predominantly fine-grained and that the identified potentially liquefiable layers: 1) are anticipated to experience relatively minor strains (1 inch or less), 2) do not appear to be laterally extensive layers, and 3) are limited in thickness.

GROUND SETTLEMENT FROM EARTHQUAKE SHAKING

Ground settlement (and other ground effects) during seismic shaking can be caused by liquefaction, lateral spreading and compression of loose fill with voids, rubble, animal burrows or other similar conditions. Settlement from liquefaction or in loose, dry fill can be estimated using empirical procedures. These estimates are based on level ground conditions and are described below. Earthquake-induced settlement can cause downdrag on deep foundations, distress to pavements, gaps between ground supported pavements/structures and deep supported structures and damage to structures founded on shallow foundations.

Liquefaction Induced Settlement

Liquefaction-induced settlements were estimated using the procedure of Tokimatsu and Seed (1984). Using this procedure, volumetric strains were estimated that could occur within submerged layers from dissipation of seismically induced pore pressures. The estimated volumetric strains were then used in conjunction with the corresponding layer thicknesses to evaluate the magnitude of seismically induced settlements at the boring locations. The estimated settlements for the design earthquake (PGA = 0.76g, Magnitude = 7.26) are about 1.5 inches in B-112 and B-113. Differential settlement should be anticipated on the same order of magnitude as the total settlements above because these potentially liquefiable materials were identified as isolated layers within a dense layer with a low potential for liquefaction.

Seismically Induced Settlement of Dry Fill

Earthquake shaking can also result in seismically induced settlement of relatively loose, dry, granular materials. However, the upper fill is generally cohesive and stiff and therefore settlement of the dry fill is only expected to occur in areas where voids exist.

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-112
Date:	9/23/2008
Calculated By:	SC

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY (CL	0	125	12	80	22	N/A	0.0
2	Lean CLAY (CL)	5	125	9	90	20	N/A	0.0
3	Lean CLAY (CL)	8	125	14	90	20	N/A	0.0
4	Clayey SAND (SC)	14	125	18	14		0.59	0.5
5	Clayey SAND (SC)	17	125	17	15		0.46	0.8
6	Sandy CLAY (CL)	22	125	13	66	20	N/A	0.0
7	Silty SAND (SM)	28	125	36	15		N/A	0.0
8	SAND with Silt (SP-SM)	33	125	32	10		N/A	0.0
9	SAND with Silt (SP-SM)	40	125	32	10		N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	1.3

TOTAL SETTLEMENT (inches)	1.3

Table D-1: Liquefaction Analysis for B-112

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY (CL	0.0	5.0	2.5	125.0	12	80	313	0	313	1.70	1.3	1.00	0.75	1.00	20	5.00	1.20	29	0.40	1.21	0.99	0.49	1.15	N/A
2	Lean CLAY (CL)	5.0	3.0	6.5	125.0	9	90	813	0	813	1.57	1.3	1.00	0.75	1.00	14	5.00	1.20	22	0.24	0.71	0.98	0.49	1.15	N/A
3	Lean CLAY (CL)	8.0	6.0	11.0	125.0	14	90	1375	187	1188	1.30	1.3	1.00	0.75	1.00	18	5.00	1.20	26	0.32	0.96	0.97	0.56	1.15	N/A
4	Clayey SAND (SC)	14.0	3.0	15.5	125.0	18	14	1938	468	1470	1.17	1.3	1.00	0.85	1.00	23	2.20	1.04	26	0.32	0.32	0.96	0.63	1.15	0.59
5	Clayey SAND (SC)	17.0	5.0	19.5	125.0	17	15	2438	718	1720	1.08	1.3	1.00	0.85	1.00	20	2.50	1.05	24	0.27	0.27	0.95	0.67	1.15	0.46
6	Sandy CLAY (CL)	22.0	6.0	25.0	125.0	13	66	3125	1061	2064	0.98	1.3	1.00	0.95	1.00	16	5.00	1.20	24	0.27	0.82	0.94	0.70	1.15	N/A
7	Silty SAND (SM)	28.0	5.0	30.5	125.0	36	15	3813	1404	2409	0.91	1.3	1.00	0.95	1.00	41	2.50	1.05	45	N/A	N/A	0.93	0.72	1.15	N/A
8	AND with Silt □(SP-S	33.0	7.0	36.5	125.0	32	10	4563	1778	2784	0.85	1.3	1.00	1.00	1.00	35	0.87	1.02	37	N/A	N/A	0.88	0.71	1.15	N/A
9	AND with Silt □(SP-S	40.0	N/A	N/A	125.0	32	10	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2
*8	$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$		
*9	If PI of Fines>15, Increase CRR by 300%		
*10	rd =	1.0 - 0.00765z	z ≤ 9.15m
		1.174 - 0.0267z	z > 9.15m
*11	Magnitude Scaling Factor (NCEER, 1997, mean values)		

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table D-2: Liquefaction Analysis for B-112

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-113
Date:	9/23/2008
Calculated By:	SC

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	8
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	95
Design Ground Water Elev. (feet)	87

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY (CL)	0	107	15	90	10	N/A	0.0
2	Lean CLAY with Sand (CL)	5	107	13	60	10	N/A	0.0
3	Lean CLAY with Sand (CL)	5	107	13	90		N/A	0.0
4	Clayey SAND (SC)	20	125	12	35		0.57	0.9
5	Lean CLAY (CL)	26	125	10	90	10	N/A	0.0
6	Sandy Lean CLAY with Gravel (CL)	35	125	34	50	10	N/A	N/A
7							N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.9

TOTAL SETTLEMENT (inches)	0.9

Table D-3: Liquefaction Analysis for B-113

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY (CL	0.0	5.0	2.5	107.0	15	90	268	0	268	1.70	1.3	1.15	0.75	1.00	29	5.00	1.20	39	N/A	N/A	0.99	0.49	1.15	N/A
2	an CLAY with Sand (	5.0	0.0	5.0	107.0	13	60	535	0	535	1.70	1.3	1.15	0.75	1.00	25	5.00	1.20	35	N/A	N/A	0.99	0.49	1.15	N/A
3	an CLAY with Sand (	5.0	15.0	12.5	107.0	13	90	1338	0	1338	1.22	1.3	1.15	0.75	1.00	18	5.00	1.20	26	0.32	0.32	0.97	0.48	1.15	N/A
4	Clayey SAND (SC)	20.0	6.0	23.0	125.0	12	35	2515	936	1579	1.13	1.3	1.15	0.95	1.00	19	5.00	1.20	28	0.37	0.37	0.95	0.74	1.15	0.57
5	Lean CLAY (CL)	26.0	9.0	30.5	125.0	10	90	3453	1404	2049	0.99	1.3	1.15	0.95	1.00	14	5.00	1.20	22	0.24	0.24	0.93	0.77	1.15	N/A
6	Lean CLAY with Gra	35.0	N/A	N/A	125.0	34	50	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
7	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
8	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.15
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2

*8 $$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z (z ≤ 9.15m); 1.174 - 0.0267z (z > 9.15m)

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table D-4: Liquefaction Analysis for B-113

APPENDIX F4

Supplemental Geotechnical Recommendations, Bulk Liquid Oxygen Storage Tank Project

FUGRO CONSULTANTS, INC.

1000 Broadway, Suite 440
Oakland, California 94607
Tel: (510) 268-0461
Fax: (510) 268-0137

May 17, 2011
Project No. 04.71110027

Children's Hospital & Research Center at Oakland
747 52nd Street
Oakland, California 94609

Attention: Mr. Doug Nelson

Subject: Supplemental Geotechnical Recommendations, Bulk Liquid Oxygen Storage Tank Project, Children's Hospital and Research Center, Oakland, California

Dear Mr. Nelson:

Fugro Consultants, Inc., (Fugro) is pleased to present this letter presenting our supplemental geotechnical recommendations for the design of new bulk liquid oxygen storage tanks at Childrens Hospital and Research Center in Oakland (CHRCO). Fugro previously prepared a report titled, *Phase I Geotechnical and Geohazard Evaluation, New Hospital Replacement Project – New Patient Tower, Children's Hospital & Research Center, Oakland, California*, dated March 30, 2009, discussing the site conditions, regional geology, and subsurface conditions of the existing CHRCO main campus, and presenting geotechnical recommendations for the design of a New Patient Tower adjoining the south side of the existing Main Hospital, as well as the retrofit of an existing helistop structure south of the Main Hospital into a new Service Building. Subsequently, we understand that CHRCO has undertaken the design and construction of interim seismic upgrades to the existing facilities, including upgrading of the bulk liquid oxygen storage system for the hospital.

PROJECT BACKGROUND AND DETAILS

Project Description

As part of the current seismic retrofit work at CHRCO, we understand that the existing bulk oxygen storage system is to be relocated and replaced. The project will involve the construction and installation of two new replacement bulk liquid oxygen storage tanks at the southwest corner of the existing Main Hospital building in a landscape and equipment area between the hospital and Martin Luther King Jr. Way (Plate 1). Two new above-ground steel tanks, about 32 and 16 feet in height, will be constructed on an at-grade, 3-foot thick, trapezoidal-shaped structural concrete mat about 20 feet wide and 27.5 to 32.5 feet long. The larger tank (No. 1) will be about 7 feet in diameter and weigh about 89,000 pounds fully loaded. The smaller tank (No. 2) will be about 5.5 feet in diameter and weigh about 23,900 pounds fully loaded. Each tank will be supported by three steel I-beam legs anchored by bolts into the structural slab foundation mat.

OSHPD Compliance

We understand that the California Office of Statewide Health Planning and Development (OSHPD), as part of their review of this project, had noted that the proposed bulk oxygen storage tank project was not specifically addressed in our March 30, 2009 geotechnical report, specifically with regard to the design of the tank foundations at the proposed location adjacent to the southwest corner of the Main Hospital building. This supplemental letter, combined with our existing report, attached to this letter, is intended to satisfy OSHPD geotechnical and geohazard requirements for the project, including the guidelines presented in *California Geological Survey – Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.* The most recent revision of Note 48 is dated January 2011.

Site Conditions

Site conditions in the area of the proposed oxygen tanks were explored by previous borings drilled in the project vicinity for previous site improvements in the area. Previous applicable exploration points are shown on Plate 1. Previous Borings PB-6 (1978) and EB-2 (2002; referred to as PB-2 on Plate 1) were closest to the project site, and judged to best represent the subsurface conditions below the project site. Our previous borings in the immediate project vicinity indicate that below existing pavement sections, the subsurface materials consist of 5 to 8 feet of stiff silty clay or medium dense sand fill materials, underlain primarily by alluvial layers of stiff to hard, sandy to silty clay, and medium dense to dense sand and gravelly sand to explored depths on the order of 50 feet. The surficial clay materials appear to be of moderate expansion potential. Free groundwater in the borings was encountered at depths of 20 and 23 feet. Copies of the pertinent boring logs are contained in our 2009 geotechnical report, which is attached to this letter.

CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the design details of the proposed bulk oxygen storage tank project, it is our opinion that the geohazard study conclusions as well as the recommended seismic design criteria presented in our March 30, 2009 report remain applicable to the currently proposed project. In our opinion, the proposed steel tanks may be supported by the proposed structural mat foundation. Supplemental foundation design criteria specific to the proposed bulk storage oxygen tanks are presented in the following paragraphs. Unless otherwise noted below, earthwork and foundation subgrade preparation recommendations presented in the March 30, 2009 report continue to apply for the current project.

Foundation Subgrade Preparation

The structural slab foundation should be supported by a minimum three-foot thickness of compacted engineered fill. The existing surficial fill materials underlying the site at the proposed tank construction may be reworked to construct this engineered fill layer. The engineered fill layer should extend outward in all directions from the perimeter of the structural mat a distance of at least three feet, where possible. Following the required subexcavation, the exposed

subgrade should be should be scarified to a depth of at least 6 inches, moisture conditioned to at least slightly above optimum moisture content, and compacted to at least 95 percent of the soil's maximum dry density based on ASTM Test Method D1557, latest version. The scarified and recompacted layer can be considered to comprise the lower most portion of the engineered fill layer. Engineered fill should be moisture conditioned as necessary, and spread and compacted in lifts not exceeding 8 inches in pre-compacted thickness to at least 95 percent of the soil's maximum dry density. The moisture content of the natural on-site expansive clayey soils reused as fill should be 2 to 3 percent above the optimum moisture content for the soil at the time of compaction.

The completed compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. If the subgrade is allowed to become dry, it should be moisture conditioned to eliminate shrinkage cracks.

Structural Mat Design

The proposed tanks may be supported on a mat foundation underlain by subgrade prepared in accordance with the recommendations presented in this letter. The mat should also be reinforced so that it is capable of cantilevering a minimum distance of 3 feet and free span a minimum diameter of 8 feet across the interior of the foundation. We recommend using the maximum allowable bearing pressures presented in Table 1 below. Design ultimate bearing pressure (FS = 1.0) is also shown for reference.

Table 1. Bearing Pressures for Mat Foundation

Load Condition	Allowable Bearing Pressure (psf)
Dead Load (FS = 3.0)	2,000
Dead plus Live Loads (FS = 2.0)	3,000
Total Loads (including wind or seismic) (FS = 1.5)	4,000
Ultimate Bearing Pressure (FS = 1.0):	6,000

We recognize that OSHPD requires geotechnical factors of safety equal to or greater than the overstrength factor used in lateral force resistance. The structural engineer may apply the design values in Table 1 provided they meet this requirement.

We recommend that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be assumed for the design of a mat foundation supported on properly prepared subgrade. This value is based on a 1-foot-square bearing area and needs to be scaled to account for mat foundation size effects. This value represents a load – deflection relationship and is thus not subject to design factors of safety. In addition, an effective plasticity index of 25 may be used for design.

Long-term resistance to lateral loads may be provided by friction along the base of the mat and by passive pressure acting on the sides of the foundation. A friction coefficient of 0.30 times the dead load may be used to evaluate the allowable frictional resistance along the bottom of the mat bearing on subgrade prepared as discussed above. In addition to friction, a long-term passive pressure equal to an equivalent fluid pressure of 300 or 150 pounds per cubic foot (pcf) can be used for lateral load resistance against the edge of the mat perpendicular to the direction of loading where the foundation is poured neat against engineered fill or the existing surficial fill material, respectively. However, unless directly confined by overlying concrete or pavement, the passive resistance contribution of the uppermost one foot of soil should be neglected.

The passive resistance is based on a factor of safety of 2.0. However, relatively large deflections would be required to mobilize the ultimate passive resistance. Therefore, in order to limit deformations to less than about 1/4-inch, we recommend that the passive resistance be considered as an ultimate value.

Based on the existing subsurface information, total and differential settlement of the mat due to new foundation loads is anticipated to not significantly affect the proposed improvements (i.e., less than 1/2 inch). Similarly, based on our evaluation of the subsurface conditions, settlement of the mat foundation due to seismic loading is also not anticipated to be significant at this location.

LIMITATIONS

The conclusions and recommendations presented in this letter supplement those of our report dated March 30, 2009, and were made in accordance with generally accepted, local and current geotechnical engineering principles and practices at the time of our study. No warranty is made, either express or implied.

The analyses and recommendations contained in this letter are based on discussions with the project team and data obtained from subsurface exploration conducted during relevant previous explorations. These explorations indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. Variations may exist and conditions not observed or described in this report could be encountered during construction. If conditions other than those described in this letter are encountered at the specific project site during construction, we should be notified so that we can provide additional recommendations, if warranted.

Fugro should be contracted to observe the placement and compaction of the engineered fill layer underlying the slab foundation, and observe and test the completed subgrade prior to slab construction in order to confirm that the fill layer was prepared in general accordance with our geotechnical recommendations, and the subsurface conditions upon which our conclusions and recommendations in this letter are based.

This letter has been prepared for the exclusive use of CHRCO and its consultants for specific application to the proposed Bulk Liquid Oxygen Storage Tank project as described

herein. In the event that there are any changes in the ownership, nature, design, or location of the proposed project, or if any future additions are planned, the conclusions and recommendations contained in this letter should not be considered valid unless 1) the project changes are reviewed by Fugro, and 2) conclusions and recommendations presented in this report are modified or verified in writing. Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

CLOSING

We appreciate this opportunity to be of continuing service to CHRCO and the project design team. If you have any questions, please call Corey Dare at (510) 268-0461.

Sincerely,

FUGRO CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER MARTIN J. WALKER No. 2935 Exp. 12/31/2012 GEOTECHNICAL STATE OF CALIFORNIA

Martin J. Walker, P.E., G.E., LEED AP
Project Engineer

REGISTERED PROFESSIONAL ENGINEER COREY T. DARE No. 2613 Exp. 6-30-2012 GEOTECHNICAL STATE OF CALIFORNIA

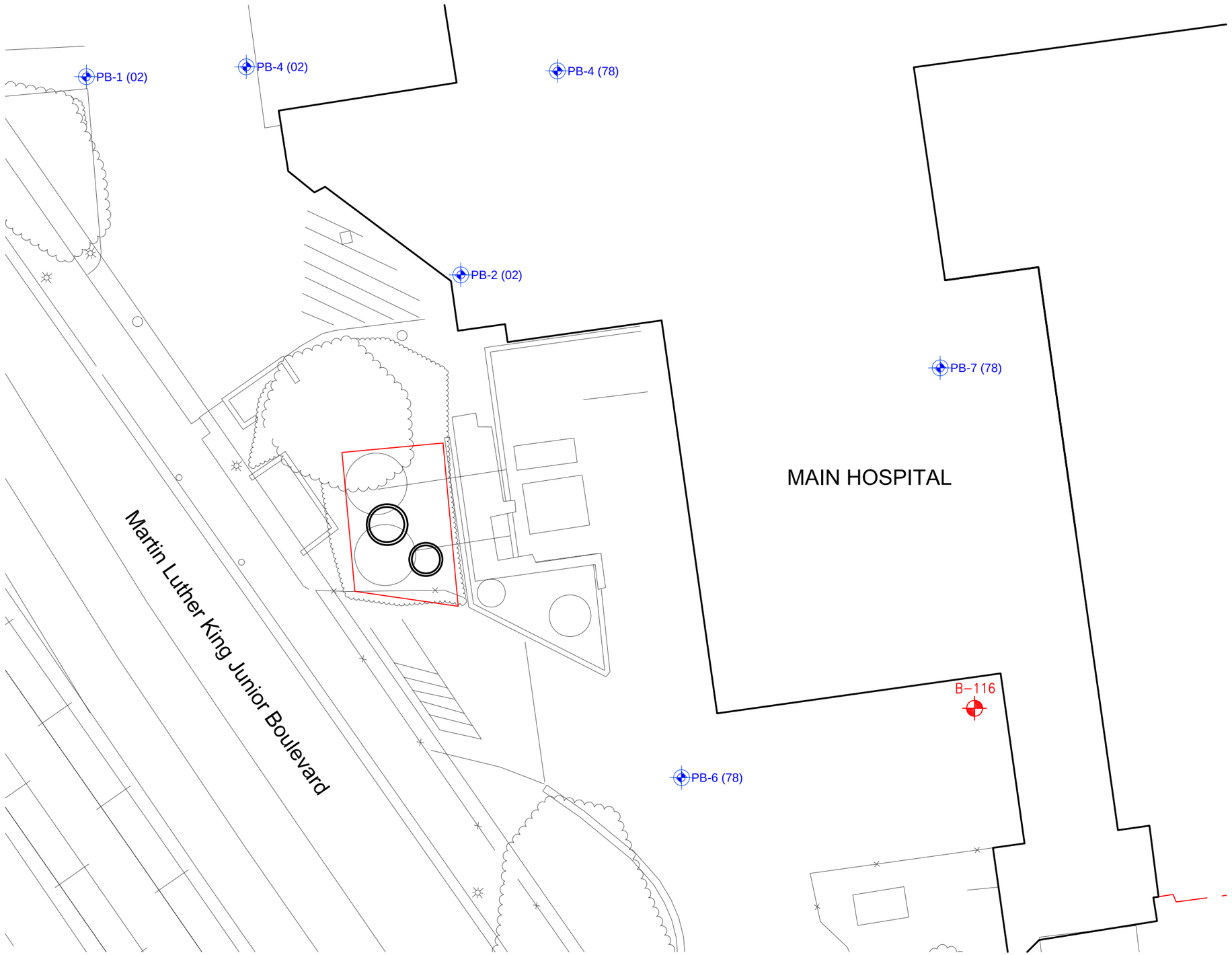
Corey T. Dare, P.E., G.E.
Principal Engineer

MJW/CTD:afp

Attachments: Plate 1 – Site Plan
Fugro Geotechnical and Geohazard Evaluation, New Hospital Replacement Project, dated March 30, 2009

Copies Submitted: (3 + PDF) Addressee
(1+PDF) Mr. Aaron Hanigan, HDR Architecture, Inc.

PLATE 1
SITE PLAN



LEGEND

B-116 APPROXIMATE LOCATION OF BORING (2008)

PB-6 (78) APPROXIMATE LOCATION OF PREVIOUS BORINGS

APPROXIMATE LOCATION OF PROPOSED TANK PAD

APPROXIMATE LOCATION OF PROPOSED REPLACEMENT OXYGEN STORAGE TANKS

NOTES:

(78) Kaldveer "Addition to Children's Hopsital"
Project No. K480-J

(02) Fugro "Western Expansion"
Project No. 21110-GI

NORTH
0 20 20
FEET

SITE PLAN
Bulk Liquid Oxygen Storage Tank Project
Children's Hospital and Research Center
Oakland, California

PLATE 1

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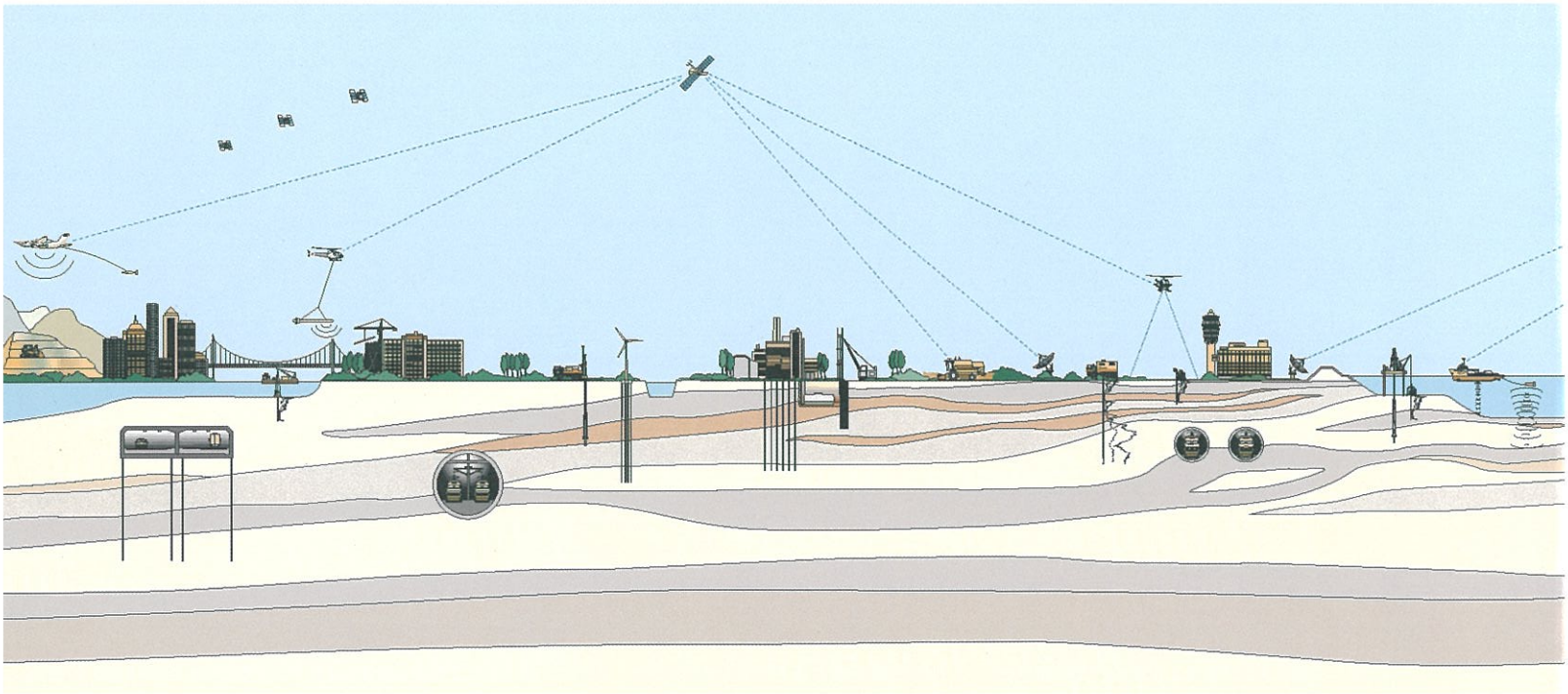
FUGRO GEOTECHNICAL AND GEOHAZARD EVALUATION REPORT
NEW HOSPITAL REPLACEMENT PROJECT
MARCH 2009

FUGRO WEST, INC.

PHASE I
GEOTECHNICAL AND GEOHAZARD EVALUATION
NEW HOSPITAL REPLACEMENT PROJECT
NEW PATIENT TOWER
CHILDREN'S HOSPITAL & RESEARCH CENTER
OAKLAND, CALIFORNIA

Prepared for:
CHILDREN'S HOSPITAL & RESEARCH CENTER AT OAKLAND
C/O HARRIS & ASSOCIATES

March 2009
Fugro Project No. 1595.002



FUGRO WEST, INC.

1000 Broadway, Suite 440
Oakland, California 94607
Tel: (510) 268-0461
Fax: (510) 268-0545

March 30, 2008
Project No. 1595.002

Children's Hospital & Research Center at Oakland
Doug Nelson c/o Sharon Knowlton
747 52nd Street
Oakland, California 94609

Attention: Mr. Doug Nelson

Subject: Geotechnical and Geologic Hazards Evaluation
New Hospital Replacement Project – New Patient Tower, Oakland, California

Dear Mr. Nelson:

Fugro is pleased to submit this geotechnical and geologic hazards evaluation report presenting the results of our field explorations, laboratory-testing program, and engineering analyses for the New Hospital Replacement – New Patient Tower Project, located in Oakland, California.

We appreciate this opportunity to be of service to you. Please contact us if you have any questions regarding the information presented in this report.

Sincerely,

FUGRO WEST, INC.

Martin Walker, P.E.
Project Engineer

Corey T. Dare, P.E., G.E.
Principal Engineer

William H. Godwin, C.E.G.
Principal Geologist

Ronald L. Bajuniemi, P.E., G.E.
Principal Consultant

MW/CTD/WHG/RLB:rh

Copies Submitted: (2 + PDF) Addressee
(6 + PDF) Tom Onnen, HDR

CONTENTS

Page

1.0 INTRODUCTION 1
1.1 Project Description 1
1.2 OSHPD Compliance 1
1.3 Scope of Services 2
2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING 3
2.1 Review of Existing Data 3
2.2 Aerial Photograph Review 4
2.3 Field Exploration 6
2.4 Laboratory Testing 6
3.0 GEOLOGIC SETTING 7
3.1 Regional Geology 7
3.2 Regional Seismicity 8
3.2.1 Historic Seismicity 10
4.0 SITE CONDITIONS 11
4.1 Surface Conditions 11
4.2 Site Geology 11
4.3 Subsurface Conditions 11
4.3.1 Patient Tower and Service Building Remodel 12
4.4 Groundwater 13
5.0 GEOLOGIC HAZARDS EVALUATION 14
5.1 Surface Fault Offset Hazard 14
5.2 Shaking Hazards 15
5.2.1 Ground Motions 15
5.2.2 Liquefaction 16
5.2.3 Dynamic Densification 18
5.2.4 Lateral Spreading 18
5.2.5 Tsunami and Seiche 19
5.2.6 Landsliding 19
5.2.7 Other Shaking Hazards 20
5.3 Other Geologic Hazards and Complicating Site Conditions 20
5.3.1 Flooding and Inundation Due to Dam or Embankment Failure 20
5.3.2 Debris Flow 21
5.3.3 Land Subsidence 21
5.3.4 Soil Expansion 21
5.3.5 Corrosion Potential 21
5.3.6 Naturally Occurring Asbestos (NOA) 23
5.3.7 Radon-222 24
5.4 Conclusions - Geologic Hazards 24
5.5 Other Hazards and Complicated Site Conditions Not Addressed In This Report 25

CONTENTS – CONTINUED

Page

6.0 DISCUSSIONS AND CONCLUSIONS 25
6.1 Expansive Soils 25
6.2 Undocumented Fill 25
6.3 Groundwater 26
6.4 Foundation Support 26
6.5 Construction Considerations 27
6.5.1 Existing Structures and Foundations 27
6.5.2 Excavation Shoring and Dewatering 27
6.5.3 Earthwork 28
6.5.4 Height Restrictions During Construction 28
6.5.5 Construction Quality Control 28
7.0 RECOMMENDATIONS 29
7.1 Seismic Design Criteria 29
7.1.1 Seismic Parameters 29
7.1.2 Site Class, Seismic Design Parameters, and Seismic Design Category 30
7.2 Site Preparation and Grading 30
7.2.1 Site Preparation 30
7.2.2 Subgrade Preparation 31
7.2.3 Reworking Undocumented Fill 31
7.2.4 Engineered Fill Materials 32
7.2.5 Fill Placement and Compaction 32
7.2.6 Trench Backfill 32
7.2.7 Surface Drainage 33
7.3 Temporary Shoring and Construction Slopes 33
7.3.1 General 33
7.3.2 Temporary Construction Slopes 33
7.3.3 Temporary Shoring 34
7.3.4 Construction Dewatering 34
7.4 Foundations 34
7.4.1 Existing Foundations at Helistop 34
7.4.2 Support of Appurtenant Structures 35
7.4.3 Conventional Drilled Pier Foundations 36
7.4.4 Large-Diameter Drilled Piers 38
7.5 Lateral Loads 39
7.5.1 Long-Term/Static Lateral Loads 39
7.5.2 Temporary Seismic Lateral Loads – Shallow Foundations 40
7.5.3 Temporary Seismic Lateral Loads – Drilled Piers 40
7.6 Concrete Slabs 41
7.6.1 Patient Tower Structural Slab Foundation 41
7.6.2 Interior Slabs-on-Grade – Services Building 42

CONTENTS – CONTINUED

Page

7.6.3 Exterior Slabs-on-Grade 42
7.7 Retaining/Below Grade Walls 43
7.7.1 Unrestrained Walls 43
7.7.2 Restrained Walls (Basement Walls) 44
7.8 Pavements 45
7.9 Additional Geotechnical Services 47

8.0 LIMITATIONS 48

9.0 REFERENCES 49

TABLES

Page

1. Aerial Photographs Examined 4
2. Large Magnitude (M≥6.0) Regional Earthquakes Within 100 Miles of the Site 10
3. Summary of Corrosion Test Results for Buried Concrete 22
4. Summary of Corrosion Test Results for Buried Ferrous Metal 23
5. Seismic Parameters 29
6. Site-Specific Seismic Design Parameters 30
7. Allowable Bearing Pressures for Spread Footings on Native Soil Subgrades 36
8. Allowable Bearing Pressures for Spread Footings on 18" Engineered Fill 36
9. Drilled Pier Design Skin Friction 37
10. Passive Soil Resistance (P) versus Displacement (Δ) Relationship 40
11. Lateral Capacity of Conventional Drilled Piers 41
12. Lateral Capacity of Large-Diameter Drilled Piers 41
13. Recommended Pavement Design Alternatives 46

PLATES

Plate

Vicinity Map 1
Topographic Site Map 2
Site Plan 3
Creek Location Map 4
Geologic Map 5
Regional Fault Map 6a
Vicinity Fault Map 6b
Faulting and Historical Seismicity 7
Cross-Section A-A' 8a
Cross-Section B-B' 8b
Cross-Section C-C' 8c
CGS Seismic Hazard Zone Map with Alquist-Priolo Earthquake Zones 9

CONTENTS – CONTINUED

Plate

Lateral Pile Capacity for Conventional Drilled Piers 10a-10c
Lateral Pile Capacity for Large Diameter Drilled Piers 11a-11c
Static Lateral Pressures on Unrestrained Walls 12a
Seismic Lateral Pressures on Unrestrained Walls 12b
Static Lateral Pressures on Restrained Walls 13a
Seismic Lateral Pressures on Restrained Walls 13b

APPENDICES

APPENDIX A FIELD EXPLORATIONS

Key to Soil Classification and Test Data ..... Plate A-1
Boring Logs ..... B-113 through B-118
Historical Boring Logs

APPENDIX B LABORATORY TESTING PROGRAM

Summary of Laboratory Test Results ..... Plate B-1
Plasticity Chart and Data ..... Plate B-2a
Plasticity Chart and Historic Data ..... Plate B-2b
Gradation Test Data ..... Plate B-3
Cerco Corrosion test results and Evaluation

APPENDIX C SEISMIC HAZARD ANALYSIS

APPENDIX D LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

1.0 INTRODUCTION

This report presents the results of the geotechnical study, and geohazard evaluation conducted by Fugro West, Inc. (Fugro) for the New Patient Tower project for Children's Hospital & Research Center at Oakland (CHRCO). The New Patient Tower will be located within the hospital campus bounded by Martin Luther King Jr. Way to the west, Highway 24 to the east and 52nd Street to the north in Oakland, Alameda County, California as shown on Plate 1 – Vicinity Map. The site coordinates are:

Latitude: 37.8370 º
Longitude: -122.2672 º

The project site is plotted on a 7 ½-minute United States Geological Survey (USGS) quadrangle topographic map on Plate 2 – Topographic Site Map.

1.1 PROJECT DESCRIPTION

Based on our conversations with the project design team and a conceptual site plan by HDR, the project architect and structural engineer, we understand that the New Patient Tower will be constructed at the location of the existing B/C Wing structure on the north side of the project site. The proposed building will be a 5 to 6-story structure with +/- 170-bed capacity within an approximate footprint area of 22,000 square feet. The building will have a basement floor bearing at approximate Elevation +83 feet[1] (about 19.5 feet below existing site grades). Because the footprint of the New Patient Tower coincides with the existing B/C Wing, the field exploration activities for this investigation were to be completed in two phases. The first phase consists of soil borings that can be conducted around existing buildings. The second phase consists of two confirmation soil borings that can only be completed once demolition of the B/C Wing is completed, thus allowing access to the ground surface in these locations. At the time of this report, the first phase has been completed.

Also to be constructed as part of this project is a Services Building (SB), which will consist of a retrofit of the existing elevated helistop structure south of the Main Hospital into an enclosed building. Appurtenant facilities, such as a pedestrian bridge or pedestrian tunnel connecting the SB and Patient Tower, are still under consideration. We assume that as a minimum, the SB will include the construction of a new ground floor level, with a concrete, interior slab-on-grade floor, at or within 5 feet of existing site grades. A new Central Utility Plant (CUP) building will also be constructed along the eastern edge of the campus adjacent to Highway 24 and south of the existing Main Hospital. A Geotechnical Evaluation and Geohazard Evaluation report for the CUP has been issued under a separate cover.

1.2 OSHPD COMPLIANCE

California hospitals are classified as critical structures according to the California Code of Regulations, Title 24, California Building Code. The Office of Statewide Health Planning and Development (OSHPD) reviews proposed new construction of hospital facilities. California

[1] Unless otherwise noted, all elevations in this report are in reference to the North American Vertical Datum, 1988 (NAVD88)

Geological Survey (CGS) serves under contract as the advisor to OSHPD for engineering geology and seismology issues including the review of geologic hazard reports.

This report was prepared to satisfy the guidelines presented in *California Geological Survey – Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.* The most recent revision of Note 48 is October 2007.

One of the CGS requirements is that approximately one subsurface exploration point be completed for every 5,000 square feet of proposed building footprint area. Previously completed subsurface explorations and laboratory data can be used to partially satisfy this requirement. New borings, included as part of this study, represent some of the additional borings necessary to meet the 5,000 square foot criteria. The locations of the borings were chosen based on accessible areas of the site to adequately characterize subsurface conditions at the location of the New Patient Tower building. CGS has indicated it will permit conditional approval of the project pending the completion and presentation of the second phase of exploration when the existing structure is demolished.

The SB, planned as retrofit of the existing helipad, will not be considered an essential facility. Design and recommendations for this structure are not expected to undergo review by OSHPD and CGS.

1.3 SCOPE OF SERVICES

The purpose of our geotechnical field exploration and laboratory-testing program was to obtain information on subsurface conditions to evaluate the geotechnical aspects of the various portions of the site where structures will be located. The scope of our services included:

- Compiling and reviewing available geotechnical and geologic data that is contained in our files and is pertinent to the project vicinity. This includes a series of previous reports prepared for various developments at the site;
- Conducting first phase of field exploration and laboratory-testing program to supplement the available information on subsurface conditions at the proposed development locations;
- Evaluating the geologic hazards at the site (a stability evaluation of the existing Caltrans Highway 24 entrance ramp is beyond this scope of work);
- Performing engineering analyses of the collected data and developing geotechnical recommendations and design criteria for the proposed New Patient Tower;
- Preparing this geotechnical report presenting the results of our geotechnical field exploration, laboratory testing program, discussion of geotechnical issues and geologic hazards and our geotechnical recommendations.
- Performing engineering analyses of the collected data and developing geotechnical recommendations and design criteria for the proposed New Patient Tower;
- Reviewing previous reports for the helistop structure and providing supplemental recommendations for support of the remodeled structure, if needed.

2.0 DATA REVIEW, EXPLORATION AND LABORATORY TESTING

The data review, exploration, and laboratory-testing program described herein were developed to provide general characterization of the subsurface materials.

2.1 REVIEW OF EXISTING DATA

Prior to conducting our field exploration and laboratory testing, Fugro reviewed relevant information relating to geotechnical, geologic, and seismic issues at the site. Fugro's local predecessor companies completed several geotechnical investigations at the Children's Hospital site since 1974. Historic boring locations from our previous work at the site are shown on Plate 3 – Site Plan. Data from these borings was used in preparing the cross sections presented in this report, and for our engineering analyses. We reviewed the results of these previous studies performed at the site, including the following reports that contain information directly related to the current project:

- Fugro (May 2008) "New Hospital Replacement Project": Fugro performed geotechnical, geological and seismic hazard evaluation for the new hospital replacement project including a 12-story hospital building within a neighboring residential area. Various analyses had been performed as part of the scope satisfying the CGS requirements. However, due to various reasons, the location of the new structure was moved within the campus by proposing the demolition of the B/C wing. This report was used to supplement our current evaluation.
- Fugro (2002) "Western Expansion": Fugro performed a geotechnical study for the western expansion of the hospital. Fugro also completed a supplemental field exploration program including four soil borings and extensive lab testing. The results of this investigation were used to evaluate the liquefaction potential at the site as part of OSHPD Permit Number HS-002307-01. This report was used to supplement our current liquefaction evaluation.
- Harza (1997) "Children's Hospital Helistop": Harza Consulting Engineers (Harza) (local predecessor company acquired by Fugro) completed two soil borings for the hospital helistop. The helistop is located southerly of the area for the Patient Tower project. The information in this report was used as a primary source of subsurface information for the SB retrofit.
- Harza (1994) "Proposed Patient Services Pavilion, Phase I": Harza completed six borings as part of a geotechnical and seismology study for the proposed patient services pavilion. These borings extended to depths up to 51 feet below grade. Laboratory testing was completed on samples collected from these borings, including index and strength tests. Two borings were converted to monitoring wells to observe groundwater levels. This report was used to supplement our current study.
- Harza Kaldveer (1992) "Geotechnical Investigation, Proposed Cath Laboratory": Harza Kaldveer completed two soil borings as part of a geotechnical investigation for a proposed Cardiac .Catheterization Laboratory located east of the Main Hospital Building and west of Dover Street. These borings extended to a maximum depth of 20-1/2 feet below grade.

- Kaldveer (1992) "Geotechnical Investigation, Ambulatory Services Center": Kaldveer Associates, Inc. (Kaldveer) (predecessor company acquired by Harza) completed two soil borings for the proposed Ambulatory Services Center located on the north side of 52nd Street at the Children's Hospital complex. These borings extended to a maximum depth of 59-1/2 feet below grade.
- Kaldveer (1992) "Geotechnical Investigation, CHORI Laboratory Addition": Kaldveer completed four soil borings for the proposed Children's Hospital Oakland Research Institute (CHORI) Laboratory Addition building at southern end of the Children's Hospital complex. The borings extended to a maximum depth of 50-1/2 feet below grade. This investigation was performed concurrently with the proposed Ambulatory Services Center.
- Kaldveer (1987) "Foundation Investigation for Nursing Tower": Kaldveer completed two borings and related laboratory testing for the nursing tower on the south side of the Children's Hospital campus.
- Kaldveer (1986) "Foundation Investigation, Proposed Medical Office Building and Parking Structure": Kaldveer completed eight soil borings to a maximum depth of 60-1/2 feet below grade for the proposed medical office building and parking structure located on the northern side of the Children's Hospital campus.
- Kaldveer (1978) "Geotechnical Investigation for Addition to Children's Hospital": Kaldveer completed seven borings to a maximum depth of 60 feet in the location of the addition located adjacent to the existing hospital.

A complete list of references used in this study is included in Section 9.0 – References. Pertinent previous boring logs used to interpret subsurface conditions at the proposed structures are included in Appendix A.

2.2 AERIAL PHOTOGRAPH REVIEW

Six sets of vertical angle, black and white aerial photographs taken between 1930 and 1990 were examined both in stereo pairs and as single photos. Stereo views are used to determine relief and shape of landforms, which are commonly controlled by bedrock type and tectonic movement. Single photographs are reviewed for tonal variations and patterns that often go unnoticed in stereo because of the high relief. The historic photos were obtained from the Pacific Aerial Surveys library. A list of the photographs reviewed is provided in Table 1 below.

Table 1. Aerial Photographs Examined

Flight Date	Approximate Scale	Serial Number
6/12/90	1:12,000	AV3845-7-21/22
5/2/69	1:12,000	AV 902-07-17/18
7/25/63	1:36,000	AV 550-08-19/20
5/3/57	1:12,000	AV 253-08-21/22
3/24/47	1:20,000	AV 11-04-11/12
1930	1:9,500	GY-30-38/39/69/70

Each photoset was examined for changes in vegetation and geologic conditions related to soil movements and bedrock conditions. Interpretations are generally based on three factors: tone, texture and trend. Tonal and textural variations, if present, may be directly related to 1) the rocks or soils underlying the site, 2) water content of the underlying material, 3) natural vegetation (commonly controlled by local geology and water content), or 4) human activities (surface modification) including agriculture. Directional trends are observed as linear features.

The examination of aerial photographs for this investigation included the identification of geologic hazards. Landslides are identifiable through such features as headscarps, concave slopes, lateral drainages, hummocky terrain and lobate toes. Faults are expressed by offset tonal or textural bands, tonal lineaments, isolated lines of springs or trees, abrupt changes in slope or drainage direction, and by streams with anomalously straight or curved courses.

Our photograph review indicates no presence of historic landslides and no features indicative of active faulting at the site. Linear features associated with the Hayward fault are visible (1963 photo) about 2 miles east of the site. No other features relating to active faulting were observed in the vicinity of the project site.

The earliest aerial photographs reviewed for the site were from 1930. At that time, the area surrounding the hospital was developed as a predominately residential neighborhood. The site appears to be developed with an early hospital tower. However, the tower in the photo is not in the configuration of today. To the south of the site, in the vicinity of the intersection of Martin Luther King Way and 47th Street, Temescal Creek is visible. The location of Temescal Creek is shown on Plate 4 – Creek Location Map, and discussed in Section 3.1 below. The close proximity of this creek implies the presence of paleochannels beneath the site, created from a meandering path. No additional features indicative of geologic hazard are visible within or trending towards the subject property. No significant changes were made before the 1947 photos.

The 1957 aerial photograph shows the existing Children's Hospital tower building as the structure appears today. The hospital is surrounded on all sides by residential property. Temescal Creek is visible on the same path as earlier photos. No features indicative of any geologic hazard are visible within or trending towards the subject property.

By 1963 the main u-shaped hospital tower has been constructed. This structure is in the same configuration today, and residential neighborhoods continue to surround the site. Temescal Creek is visible to the south; however, more of the creek path has been diverted into underground culverts.

By 1969 additional hospital towers have been constructed directly northwest of the main tower. Caltrans has constructed Highway 24 in the right-of-way directly east of the hospital. The elevated roadway includes retaining wall and embankment slope with an approximate inclination of 2:1 (horizontal to vertical).

The 1990 photograph was the earliest photograph reviewed after the 1989 Loma Prieta Earthquake. No features were readily observed as being related to earthquake-induced damage, such as faulting, liquefaction, or lateral spreading.

2.3 FIELD EXPLORATION

Fugro's field investigation for the project is included drilling six new geotechnical borings. The borings were located in the field by our engineer based on existing site features, and to provide adequate coverage beneath the proposed hospital structure.

V&W Drilling, Inc. of Stockton, California drilled the geotechnical borings on August 8 and 9, 2008. The six borings completed for the New Patient Tower, SB, and CUP structures were numbered B-113 through B-118. The borings extended to depths of 25 to 75 feet below existing grade. All borings were backfilled with cement grout in accordance with our Alameda County drilling permit.

A Fugro engineer observed the drilling operation and logged the soil encountered. Our engineer recorded SPT N-values and estimated shear strength of the soils using a Pocket Penetrometer. Recovered soil samples were visually examined and classified in accordance with the Unified Soil Classification System (USCS) and ASTM D-2488 (visual method). Soil samples were transported to our office and laboratory for further examination, confirmation of classification and laboratory testing.

Detailed descriptions of the soils encountered in each of the borings are presented on the boring logs in Appendix A. The boring logs depict location specific subsurface conditions. The approximate locations of the borings were estimated by taping from existing landmarks and the locations should be considered accurate only to the degree implied by the method used. Elevations shown on the boring logs are in reference to the North American Vertical Datum of 1988 (NAVD88), and were estimated from contours on the project topographical survey provided by Chaudhary & Associates, Inc. The subsurface conditions encountered in the soil borings are summarized in Section 4.3.

2.4 LABORATORY TESTING

Geotechnical laboratory testing was conducted on the soil samples collected from the borings at Fugro's soil mechanics laboratory in Oakland, California. The geotechnical laboratory test program included: classification tests (gradation, fines content, Atterberg limits, water content, unit weight), and miniature vane strength tests. The results of the laboratory tests are presented on boring logs (Appendix A) at the appropriate sample depths, and in Appendix B - Laboratory Test Results.

Corrosion testing was conducted on three soil samples at CERCO Analytical, Inc., in Pleasanton, California. Corrosion potential is discussed further in Section 5.3.5 – Corrosion Potential. Results of the corrosion testing are provided in Appendix B.

3.0 GEOLOGIC SETTING

3.1 REGIONAL GEOLOGY

The site is located within the San Francisco Bay alluvial plain. This area, along with the Diablo Mountain Range, and the Santa Cruz Mountains are within the Coast Range Geologic Province, a belt of sedimentary, volcanic, and metamorphic rocks that extend from southern California to Oregon. The structural geology of the Coast Range is complex and dominated by transpressive stress caused by the interaction between the Pacific and North American plates. This stress, concentrated along faults within the San Andreas fault system, results in the series of northwest-trending ridges and valleys that characterize the area. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates, subduction of the Pacific Plate beneath the North American Plate, and subsequent strike-slip faulting along the Proto-San Andreas, and San Andreas fault system. Strike-slip motion along the plate boundaries replaced subduction several million years ago (Atwater, 1970).

Geologic formations in the San Francisco Bay region range in age from Jurassic (190 to 135 million years ago) to recent Holocene (less than 11 thousand years ago). The oldest rocks are deformed Mesozoic sedimentary and volcanic rocks of the tectonically accreted Mesozoic Franciscan Complex and the contemporaneous Great Valley Sequence. During the last glacial maximum, significant relief of the bedrock was formed as a result of incision of creeks and streams to reach the elevation of the global sea level; approximately 140 feet lower than sea level today. As sea level rose with the onset of the current interglacial, lowland areas in the San Francisco Bay Area were subject to the deposition of a transgressive sequence of alluvial sediments, ranging in age from Pleistocene to the present. Younger alluvial, marine and marsh deposits have accumulated in the valleys in the region as a result of weathering in the surrounding mountains and sea level rise.

The regional geology, as mapped by Graymer (2000) is presented on Plate 5 – Geologic Map. Geologic units that are exposed within a mile of the site include the sandstone and quartz diorite members of the Novato Quarry terrane of the Franciscan Complex (Kfn, Kfgm), and surficial alluvial sediments (Qhaf, Qhl, Qpaf). Holocene aged alluvial fan deposits (Qhaf) underlie the site. Graymer (2000) described this formation as "alluvial fan and fluvial deposits, [a] brown or tan, medium dense to dense, gravelly sand or sandy gravel that generally grades upward to sandy or silty clay." An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site as shown on Plate 5 – Geologic Map. The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. The fault does not appear to impact the Pleistocene aged alluvial deposits mapped in the vicinity. The fault location is poorly constrained, and is not considered active, nor has the fault been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

Radbruch and Case (1967) map the site as being underlain by the Temescal Formation (Qtc). The Temescal Formation fills old channel meanders (of uncertain location) that were cut into the underlying San Antonio Formation (Qsu), and as such do not uniformly underlie the

area. The thickness of the Temescal Formation varies from 3 to 18 feet. The nomenclature "Temescal Formation" was first used by Lawson (1914) to comprise several presumably contemporaneous alluvial units of different origin, lithology, and physical properties. The source material in the vicinity of the project location is assumed to be the San Antonio Formation, which consists of "clay, silt, sand, and gravel, some pebbles". Most beds contain flakes or pebbles of white Claremont chert. Clays of this formation are often montmorillonite, and highly plastic. The maximum thickness of the San Antonio Formation is unknown.

Sowers (1993, rev. 2000) mapped modern day hydrology of Oakland and its vicinity, and where available, historic creek paths were included. Temescal Creek was mapped as being contained in a culvert, passing directly south of the project location (Plate 4 – Creek Location Map). As described in Section 4.3, Subsurface Conditions, paleochannels resulting from the meandering path of the creek underlie portions of the project site.

The regional geomorphology is generally controlled by the active Hayward fault, which is located approximately 2 miles east–northeast of the project. Local geomorphology is controlled by fluvial deposition associated with Temescal Creek.

3.2 REGIONAL SEISMICITY

Geologists and seismologists recognize the San Francisco Bay Area as one of the most active seismic regions in the United States. Dominated by the San Andreas fault system, the Bay Area is comprised of mostly northwest-trending strike-slip faults driven by the interaction of the Pacific and North American Plates. Movement between these two plates is predominantly accommodated on the San Andreas, Hayward-Rogers Creek, Calaveras, San Gregorio, and Concord-Green Valley faults. The major fault in the system is the San Andreas fault, a major rift in the earth's crust that extends for at least 750 miles.

In 2003, the Working Group on California Earthquake Probabilities (WGCEP), in conjunction with the United States Geological Survey (USGS), published an updated report evaluating the probabilities of significant earthquakes occurring in the Bay Area over the next three decades (2002-2031). WGCEP found a 62 percent probability that at least one magnitude 6.7 or greater earthquake will occur in the San Francisco Bay region before 2031. This probability is an aggregate value that considers seven principal Bay Area fault systems and unknown faults (background values). The San Francisco Bay region continues to be seismically active. The principal active faults in the Bay Area include the San Andreas, Hayward, Calaveras, and the San Gregorio faults. Earthquakes occurring along these faults are capable of generating strong ground shaking at the project site. The locations of significant Bay Area faults are shown on Plate 6a – Regional Fault Map, and Plate 6b – Vicinity Fault Map.

The closest active fault to the site is the Hayward fault. The Hayward fault was originally mapped by Lawson (1908), as part of his report to the State Earthquake Investigation Commission regarding the April 18, 1906 San Francisco earthquake. This report included a "comparison with other severe earthquakes in the same region", where he compiles data obtained from newspapers and eyewitness accounts. The fault was named after the city of Haywards (now Hayward), the site of the greatest damage.

The Hayward fault is the southern part of an extensive fault system that includes the Hayward, Rodgers Creek, Healdsburg, and Maacama faults. This system of faults extends over 280 kilometers (km) from San Jose to Mendocino County (Steinbrugge and others, 1987). The Hayward fault extends from the southwestern margin of the East Bay Hills for a distance of 105-km from San Pablo Bay in the north to an area near Mount Misery, east of San Jose, where slip is transferred to the Calaveras fault via a complex system of oblique slip and reverse faulting. The fault is divided into three segments: the 42-km long northern segment which extends from the northwestern margin of San Pablo Bay to the vicinity of San Leandro, the 34-km long southern segment, which extends from San Leandro to the Warm Springs district of Fremont, and the 28-km long southeast Extension segment which extends from the Warm Springs District to the vicinity of Mount Misery (Bryant and Cluett, 2000). The segment closest to the project site is the northern Hayward fault. Based on earthquake focal mechanisms, the Hayward fault is near vertical. However, secondary fault traces are common and exhibit west-vergent structures that may merge at depth with the Hayward fault. Geomorphic evidence for the northern segment of the Hayward fault includes, offset stream channels, sag ponds, and distress to cultural features as a result of fault creep.

A definitive recurrence interval has not been developed for this segment, but Lienkaemper and others (1997) concluded there have been at least four surface rupturing earthquakes in the last 2,250 radiocarbon years. The most recent event has been dated as after 1640 AD, but before 1776 AD. The 1836 magnitude 6.8 earthquake was originally believed to have occurred on this segment of the Hayward fault, however, recent work by Toppozada and Borchardt (1998) re-evaluated this earthquake to locate the epicenter further south, east of Monterey Bay.

The latest assessment of the U.S. Geological Survey Working Group on California Earthquake Probabilities (2003) has given the northern segment of the Hayward fault an 11 percent probability of exceedance, and the Hayward-Rodgers Creek fault (in its entirety) a 27 percent probability of exceedance, in the 30 year period ending in 2031 for a magnitude 6.7 or greater earthquake. Holocene displacement on the Hayward fault is predominantly right lateral, although varying degrees of vertical slip are indicated by the presence of escarpments along the fault's surface trace. Lienkaemper and Borchardt (1992) evaluated the aseismic slip rate for the Hayward fault. They found the area with the highest aseismic slip, 9 mm/yr (along the southern Hayward fault segment in the vicinity of Fremont, California), and assumed that it represented the minimum slip rate of the fault at depth. The historic surficial slip rate measured in the Oakland area only accounts for approximately 4 mm/yr of this total. It is assumed, therefore, that the Hayward fault in this area is locked at depth and has the potential to rupture in large events.

The approximate distances of the site to the closest known mapped active faults are summarized in Table C-1 in Appendix C. Table C-1 summarizes the seismic sources and their pertinent characteristics. These data were used as part of the site-specific seismic hazard analyses discussed in Appendix C. The distances to faults were estimated using Boore et al. (1997). Fault parameters were adapted from the California Geologic Survey (CGS) fault database (2003).

Earthquakes on these or other active faults (including unmapped faults) could cause strong ground shaking at the site. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site, and other factors.

The project site is not located within an Alquist-Priolo Earthquake Fault Zone (discussed in further detail in Section 5.1 below).

3.2.1 Historic Seismicity

Since the first historical reports from the late 1700s, major earthquakes have been recorded along the San Andreas fault system in the Bay Area. Table 2 presents large magnitude ($M_w \geq 6.0$) earthquakes that have occurred within 100 km of the site between 1800 and 2007. The locations of the closest historic earthquakes to the project are shown on Plate 7 – Faulting and Historical Seismicity. The computer program EQSEARCH, v3.00 (Blake, 2001) was used to compile the seismicity information presented in Table 2.

Table 2. Large Magnitude (M≥6.0) Regional Earthquakes Within 100 Miles of the Site

Epicenter Location	Date	Moment Magnitude	Distance (mi)	Distance (km)	Compass Direction to Epicenter
San Francisco	June 21, 1808	6.0	12.9	20.7	Southwest
Hayward	October 21, 1868	7.0	13.2	21.3	South
San Francisco	April 18, 1906	8.25	15.8	25.4	Southwest
San Francisco Peninsula	June 1, 1838	7.0	17.9	28.8	Southwest
Antioch	May 19, 1889	6.25	23.0	37.0	East
Mare Island	March 31, 1898	6.5	26.0	41.9	Northwest
San Jose	December 26, 1858	6.25	30.8	49.6	Southwest
Vacaville	April 19, 1892	6.5	41.5	66.8	Northeast
South Santa Cruz Mountains	October 8, 1865	6.5	42.2	68.0	South
1984 Morgan Hill	April 24, 1984	6.1	47.5	76.4	Southeast
1911 Morgan Hill	July 1, 1911	6.5	49.5	79.7	Southeast
Winters	April 21, 1892	6.25	49.9	80.4	Northeast
Loma Prieta	October 18, 1989	7.1	59.2	95.3	South
Gilroy	June 20, 1897	6.25	71.6	115.2	Southeast
Pajaro Gap	April 24, 1890	6.25	74.4	119.7	South
Pajaro Gap	October 18, 1800	7	79.4	127.8	South
Pacific Ocean	October 22, 1926	6.1	84.9	136.6	South

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The topography of the area surrounding Children's Hospital is shown on Plate 2 – Topographic Site Map. Note that Plate 2 presents a USGS topographic map, with elevations referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). The standard conversion from NGVD29 to North American Vertical Datum, 1988 (NAVD88) is to add 2.38 feet to elevations referencing the NGVD29 datum. The local topography and surface conditions at each proposed structure are discussed below.

The proposed Patient Tower structure will be located on the portion of the CHRCO campus south of 52nd Street. The southern portion of the CHRCO campus occupies a triangular-shaped piece of land bordered by an entrance ramp for westbound Highway 24 to the east, Martin Luther King Jr. Way to the west, and 52nd Street to the north. A hospital parking garage and hospital office building occupy a portion of the area north of 52nd Street and are connected to the existing CHRCO structures by a bridge at the second story. The geotechnical reports for the existing hospital structures were referenced previously in this report.

The footprint of the proposed Patient Tower partially coincides with the existing B/C Wing of the hospital. This wing will be demolished to accommodate construction of the new tower. The site of the Tower is also occupied by an existing loading dock, paved traffic loop landscape areas including mature trees and small bushes, and a smoking enclosure. The area surrounding the proposed tower and existing helistop are paved with asphaltic concrete and Portland cement concrete. A utility easement extends roughly east-west across the site to the south of the existing B/C Wing.

According to a site survey conducted by Kellogg Aerial Surveys dated October 22, 2007, the CHRCO site is relatively flat, with a gentle slope increasing from about Elevation 99 feet at the western end of the proposed hospital to about Elevation 105 feet at the eastern end of the proposed hospital. Directly east of the proposed hospital, the Highway 24 ramp abutment rises at a slope of approximately 2.5:1 (horizontal to vertical) to about Elevation 130 feet.

4.2 SITE GEOLOGY

The CHRCO site is located on a gradually sloping outwash plain located between the Berkeley Hills to the east and San Francisco Bay to the west. The native soils at the site consist of Quaternary age alluvial deposits likely including materials from the San Antonio and Temescal Formations, according to Radbruch and Case (1967) and Helley, Lajoie and Burke (1972). Graymer (2000) maps the site as Holocene age alluvial and fluvial deposits, as shown on Plate 5 – Geology Map.

4.3 SUBSURFACE CONDITIONS

Our subsurface investigation indicates that about 4 to 8 feet of clay fill with varying amounts of sand and gravel exists at the proposed development locations. Beneath the fill, the alluvial stratigraphy encountered across the site typically consisted of a layer of stiff lean clay,

over interlayered clayey sands to sandy clays with varying amounts of gravel over stiff lean clay. The clayey sands encountered within the interbedded layer are interpreted to be the paleochannels associated with a meandering paleo-Temescal Creek. These deposits appear to be at various stages of development of the rubification process (e.g., the reddening of soils through the release and precipitation of iron as an oxide during weathering). In general, there appear to be two paleochannel deposits that may or may not be laterally continuous across the site (see Plates 8a through 8c). One is shallow and appears to be relatively laterally continuous with a greater percentage of fines, than the other deeper, more poorly graded deposit. These deposits may or may not connect at depth. One previous 150-foot deep boring on 52nd Street, PB-1 (74), shows lean clay below 50 feet to a depth of 150 feet with two clayey sand layers encountered between 102 feet and 108½ feet and 118 feet and 121 feet. The subsurface conditions at the site south of 52nd Street of the proposed structures are discussed below. The geologic units are described in order of increasing depth. Detailed descriptions of the deposits encountered in each of the borings are presented on the boring logs in Appendix A. Interpreted subsurface profiles are shown on Plates 8a through 8c.

The attached boring logs and related information depict location specific subsurface conditions, encountered during our field exploration. The passage of time could result in changes in the subsurface conditions.

4.3.1 Patient Tower and Service Building Remodel

The subsurface stratigraphy at the proposed Patient Tower and Service Building Remodel area south of 52nd Street is generally characterized by four layers, described below in order of increasing depth: I) lean clay fill; II) lean clay alluvium; III) interlayered clayey sand and sandy clay alluvium with varying amounts of gravel and IV) lean clay alluvium to the maximum depth explored during this portion of our subsurface investigation of 75 feet below existing site grades.

The lean clay fill was encountered below the surface topsoil and asphalt paving and extended to depths ranging from 4 feet to 8 feet. This fill is generally characterized as dark brown to black, firm to stiff, lean clay with trace organics. The fill has a moderate to high expansion potential, with measured Plasticity Index ranging from 22 to 28 percent. Fugro does not have documentation indicating that these fills were properly compacted during placement.

The clayey fill layer encountered beneath the asphalt at the traffic loop was of similar thickness and consists of black to yellowish-brown, firm to stiff, sandy lean clay to dense to very dense clayey sand with varying gravel content. Atterberg limit test results show the Plasticity Index in the beneath the traffic loop fill ranges from 12 to 21 percent, indicating a moderate expansion potential.

Beneath the fill is a layer of stiff to very stiff lean clay with varying amounts of sand and gravel. This clay layer was consistently observed to be about 10-feet thick in the borings. Water content measured on samples in this layer ranged from 14 to 26 percent, with an average of about 20 percent. Undrained shear strength (S_u) measurements in this layer from both miniature vane and unconfined compression tests range from about 1.6 kips per square foot

(ksf) to over 4 ksf. In Boring B-118, clean sand and angular gravels were encountered at a depth of about 12 to 13 feet, grading back to the clay profile by 15 feet below grade.

Below the stiff clay layer, we observed interlayered sandy clays and clayey sands with varying amounts of gravel. The individual clayey sand and sandy clay layers range in thickness from about 5 to 10 feet. Gradual transitions were often observed between the sandy material and the clayey material. Laboratory testing indicates the sandy material typically contains a significant amount of clay (at least 10 percent and often higher than 35 percent) and the clay material contains significant amounts of sand. Both the sand and the clay layers contain a trace to significant amounts of gravel. The sandy units were typically medium dense to dense, and the clay units were typically firm to stiff.

Below the interlayered sands and clays, we observed a very stiff to hard lean clay layer. This layer was typically encountered about 40 to 50 feet below grade in the more southerly borings. Approximately 50 feet below site grade, very stiff, lean clays are interbedded with dense to very dense sand or gravel lenses with significant fines (silt or clay) content. Recorded SPT N-values below 50 feet were very high, typically in excess of 30 blows per foot.

4.4 GROUNDWATER

Groundwater levels were observed to vary significantly across the site during our subsurface investigation. Groundwater was encountered as shallow as about 8½ feet below grade in historical (February 2008) Boring B-101 and as deep as about 40 feet below grade in Boring B-104. During our most recent investigation in August, 2008, groundwater was encountered in Borings B-113, B-115, B-116 and B-118, at between 20 feet and 25 feet below grade. Groundwater was typically measured within 30 minutes of completing the boreholes because the borings were backfilled immediately after drilling with cement grout in accordance with our Alameda County drilling permit. Note that the borings may not have been left open for a sufficient period of time to establish equilibrium ground water conditions.

As part of the geotechnical investigation for the Patient Services Pavilion, Harza (1994) installed two monitoring wells at the existing hospital. One Monitoring Well, EB-1 (OW), is located near the southern courtyard and the other Monitoring Well, EB-6 (OW), is located in the physicians parking lot near the entrance gate (approximate locations are shown on Plate 3 – Site Plan). Water levels were recorded in September of 1992 (dry period), approximately 50 days after drilling, and in March of 2008 (wet period) as part of this study. Groundwater in EB-1 (OW) was measured at 22 feet (Elevation 79 feet) in 1992 and at 21 feet (Elevation 80 feet) in 2008. For EB-6 (OW), groundwater was measured at 16 feet (Elevation 86.5 feet) in 1992 and 12.5 feet (Elevation 90 feet) in 2008.

Historically, groundwater levels at the Children's Hospital site have varied considerably. Observed levels range from 6 feet below grade in Boring PB-3 (86b) (K876-3, 1986), at the Family House east of Dover Street, to 23 feet below grade in Boring PB-5(86a) (K928-1, 1986), located in the current parking garage area. Boring PB-5 (86a) was converted into a temporary observation well and groundwater was recorded at a depth of 17 feet below grade three weeks after drilling. Near the Ambulatory Services Center, north of 52nd Street and to the west of Dover Street, groundwater was recorded at depths of 12 to 23 feet during investigations in 1986, 1990 and 1991.

The CGS Seismic Hazard Zone Report for the Oakland West Quadrangle (1991) presents a map that shows approximate depths to historic high ground water. Groundwater depth contours presented on this map suggest historic high ground water at the Children's Hospital site to be on the order of about 7.5 feet below grade.

Based on the depths of observed shallow groundwater in borings and historical well measurements, and to account for varying site topography (typically less than 2 feet), we have estimated a design ground water depth of 8 feet below ground surface for use in our analyses for the proposed New Patient Tower. This depth corresponds to an approximate Elevation of +93 feet at the proposed location of the New Patient Tower. Fluctuations in the groundwater level may occur from changes in seasons, variations in rainfall, and other factors.

5.0 GEOLOGIC HAZARDS EVALUATION

Geologic hazards can be grouped into seismic and non-seismic categories. Seismic hazards can be further subdivided into surface fault offset, ground shaking, seismic ground deformation and water movements (tsunamis & seiches). Seismic ground deformation includes liquefaction, dynamic densification and landsliding. Certain types of ground deformation and water movement, however, can also occur under non-seismic conditions, so there is some overlap between seismic and non-seismic hazards. In general, the list of potential geologic hazards can also include such rare occurrences as mine collapse, hydrocompaction and peat oxidation. These hazards are rare and do not warrant discussion in connection with the project site as peat deposits were not encountered in the explorations. In addition, there is no mention of mines underlying the site in the literature we reviewed.

Historical evidence and the results of current seismological research indicate that at least one moderate to severe earthquake will occur sometime during the design life of the proposed development. Detailed discussions of severe ground shaking and other geologic hazards with respect to the site are presented below.

5.1 SURFACE FAULT OFFSET HAZARD

The potential for surface fault rupture at the site is low. No known active fault traces are mapped crossing the Children's Hospital Oakland site, based on existing geologic maps and reports including Graymer (2000) and Helley and Graymer (1997). The site is not located within a Fault-Rupture Hazard Zone as designated by the Alquist-Priolo Earthquake Fault Zoning Act (1972), as mapped on the official Earthquake Fault Hazard Zone Maps issued by the State of California (1982). The site is approximately 2 miles west of the closest active fault zone (Hayward fault) depicted on the Special Studies Zones map (CGS, 1982), as shown on Plate 9 – CGS Seismic Hazard Zone Map. The Fault Activity Map from the CGS digital database (CGS, 2000), as shown on Plates 6a (regional view) and Plate 6b (vicinity view), shows the Hayward fault to the east of the site.

An unnamed, concealed thrust fault is mapped approximately 0.5 miles east of the project site (Plate 5 – Geologic Map). The southern terminus of this fault is truncated by a northeast trending strike-slip fault that has in turn been truncated by the Hayward fault. Based on Graymer (2000), this fault does intersect the ground surface within Pleistocene aged alluvial

deposits near the site vicinity. The fault location is poorly constrained, and is not considered active, nor has it been included in the State of California Alquist-Priolo Earthquake Hazard Zones. As mapped, this concealed thrust fault does not appear to pose a surface-fault rupture hazard.

No indications of surface fault displacement were found in our reconnaissance of the site. While the possibility of new fault breaks developing during future earthquakes cannot be precluded, historic occurrences of surface fault rupture have generally followed pre-existing active fault traces.

5.2 SHAKING HAZARDS

Strong ground shaking at the Children's Hospital site will likely occur during a moderate to severe earthquake occurring on one of the active Bay Area faults. Strong ground shaking can cause the structures to shake and also has the potential to induce other phenomena that can indirectly cause damage to structures. These phenomena include: soil liquefaction, seismically induced waves such as tsunamis and seiches, inundation from dam or embankment failure, landsliding, lateral spreading, differential compaction, and ground cracking. Detailed discussions of these phenomena with respect to the site are presented in the subsequent paragraphs.

Youd and Hoose (1978) compiled data related to damage caused by historic earthquakes in the San Francisco Bay region, including the 1906 San Francisco and 1868 Hayward Earthquakes. To the south, in the vicinity of the Alameda and Oakland border, "… the [1906] earthquake did not produce, relatively speaking, much destruction in these [Oakland, Alameda, and Berkeley water systems] works." Damage was limited to minor settlement and pipeline breaks, stream bank failures, and minor lateral displacements on the order of 8 inches. No features were noted in the project area as a result of the 1906 San Francisco Earthquake. In 1868, "…portions of the wharves were carried away in some instances, while walls were cracked in almost every house". No features were noted in the Children's Hospital vicinity or surrounding areas as being related to liquefaction.

Liquefaction data are also available for the 1989 Loma Prieta Earthquake. Tinsley and others (1998) mapped liquefaction features nearby in Berkeley and Emeryville. Damage included sand boils, cracks, ground settlement, lateral spreading and pipeline breaks. "Lateral spreading and settlement caused extensive pavement damage to the freeway road surface. Pavement cracking was oriented parallel to the shoreline, with a total lateral movement of 30 to 120 mm. Several cracks were more than 50 m long." No other features indicative of liquefaction or lateral spreading were identified in the vicinity of the Children's Hospital site as a result of the 1989 Loma Prieta Earthquake.

5.2.1 Ground Motions

Site-specific probabilistic seismic hazard analyses were conducted to generate horizontal response spectra. The spectra were modified to incorporate the near source effects, and factors were applied to generate fault normal spectra. Detailed information regarding our site-specific seismic hazard evaluations are presented in Appendix C.

The design response spectrum and Maximum Considered Earthquake (MCE) events were developed per the 2007 CBC requirements and per the analysis procedures outlined in the design guidelines prepared by the American Society of Civil Engineers (ASCE), ASCE 7-05. Site-specific response spectra were developed for this project. The development of site-specific spectra included conducting probabilistic hazard analyses for 2475-year return period, as well as conducting deterministic checks that are required by ASCE 7-05. Additional modifications to the ground motion were calculated according to the OSHPD Code Application Notice (CAN) NO. 2-1802A.6.2, effective September 30, 2008, and CAN NO. 2-1614A.1.2 dated January 26, 2009. Detailed information regarding the development of site-specific spectra is presented in Appendix C. The design peak horizontal ground acceleration (PGA) for the site is 0.76g, based on our site-specific analyses (Table C-4).

The hazard analyses and ground motion evaluations were performed based on our understanding that the New Patient Tower structure will be founded on a deep foundation system. If the foundation system is significantly changed, the applicability of the results presented in Appendix C should be re-evaluated with the selected alternate foundation system. The results presented in Appendix C are applicable only at the foundation level, based on the criteria mentioned above.

5.2.2 Liquefaction

The susceptibility of a soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in a) distribution of grain size, b) soil density, c) aging, d) cementation, e) fines content, and f) plasticity characteristics of the fines. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface, a depth usually considered to be less than about 50 feet.

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers located close to the ground surface. When very loose to medium dense saturated granular soils are shaken, their tendency to contract and compress may lead to the development of excess pore pressures. If the seismic shaking is strong enough and duration long enough, the buildup in pore pressure can produce a significant loss of shear strength. Downward movement (settlement) can occur as a result of soil strength loss below foundations bearing on liquefied material, or as a result of the excess pore water pressure dissipation and redistribution of the soil particles to a denser configuration.

Liquefaction is said to occur when the excess pore pressure equals the initial effective stress in the soil. If the shaking continues after the onset of liquefaction, ground distress may occur (e.g., sand boils, settlement, lurching, and lateral deformation). Liquefaction may also cause a loss of capacity of shallow or deep foundations, loss of lateral capacity or stiffness, and lateral ground spreading.

The liquefaction susceptibility of the project site is considered to be moderate, based on the Association of Bay Area Governments (ABAG) Liquefaction Susceptibility Map (2000). Witter and others (2006) mapped the quaternary deposits and evaluated the liquefaction susceptibility for the San Francisco Bay Area. The map shows the soils at the site as Holocene

aged alluvial fan deposits, and as such, the liquefaction susceptibility is dependent on the depth to groundwater in the vicinity. The liquefaction susceptibility is considered moderate for groundwater depths of 10-30 feet below ground surface and high for groundwater less than 10 feet below ground surface. The site has a moderate to high liquefaction susceptibility as mapped by Witter and others (2006). The site is located in an area designated as a zone of required investigation for liquefaction by California Geological Survey (CGS), as shown on the Seismic Hazard Zones Maps, West Oakland Quadrangle (CGS, 2003), included as Plate 9 – CGS Seismic Hazard Zone Map.

Our liquefaction evaluation was conducted using the procedures recommended by the National Center for Earthquake Research, and summarized by Youd et al (2001). The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.76g. This PGA value represents the design response spectrum ground motion at the site, based on our probabilistic seismic hazard analyses discussed in Appendix C. An anticipated earthquake magnitude (maximum moment magnitude) of 7.26, representing the upper bound M_{max} for a full rupture of the Hayward-Rogers Creek fault, was used in conjunction with the PGA value assigned in the liquefaction analyses. The liquefaction-induced settlements were estimated using the procedure of Tokimatsu and Seed (1984).

The calculations of liquefaction potential and corresponding settlement are included in Appendix D. Our liquefaction analyses show that the materials beneath the proposed New Patient Tower are generally cohesive and/or dense enough to have a low potential for liquefaction. This low potential for liquefaction in the new borings agrees well with previous liquefaction studies around the CHRCO campus that also found low potential for liquefaction. Our previous liquefaction studies included a detailed study for the western expansion of the hospital (OSHPD Permit Number HS-002307-01). This previous study included four soil borings and extensive lab testing to evaluate the liquefaction potential at the site. The study showed the clayey sand layers to be "non-liquefiable" according to the procedure by Youd et al (2001). Additionally, liquefaction analyses of the borings performed as part of the geotechnical investigation for the medical office and parking structure by Kaldveer (1990) indicated the sand layers are medium dense to dense and liquefaction potential is remote.

However, our analyses identified two isolated borings (B-112 and B-113), with relatively thin layers of clayey sand that may be susceptible to liquefaction and subsequent densification and settlement following a design level earthquake. The proposed New Patient Tower is located approximately 200 feet from the locations of borings indicating potentially liquefiable soil strata. The Tower will be supported on a deep foundation system (discussed below), so the potential for liquefaction-related damage to the structure is considered low. The proposed New Patient Tower will be located sufficiently far from the affected soils such that downdrag loads will not significantly impact the foundation system.

The saturated clayey sand materials with the potential for liquefaction identified in the borings are likely part of paleochannels associated with Temescal Creek. The isolated loose zones are within what are typically dense granular layers with low potential for liquefaction. The potential for significant liquefaction and large liquefaction-induced settlements or other ground failures at the site is considered low, based on the evaluation of the borings and our review of previous liquefaction studies at this site. Additionally, the effects on seismic waves passing

through isolated zones of softening soil are not sufficient to consider the Site Class as "F" according to the CBC. Based on conversations with CGS, we understand that the interpretation of the Site Class based on liquefaction potential should take into consideration the likely effects of liquefied soils on propagating ground motions.

5.2.3 Dynamic Densification

Volumetric compression, or soil densification, is settlement caused by earthquake-induced ground shaking that causes soil particles to compress together. Dry cohesionless soils, such as sands and gravels, are susceptible to this type of settlement. Earthquake induced settlements of dry cohesionless soils depends on three main factors, including the relative density of the soil, the maximum shear strain induced by the design earthquake and the number of shear strain cycles which can be related to earthquake magnitude.

We used the methodology proposed by Tokimatsu and Seed (1987) to estimate the degree of dynamic densification potential in the unsaturated alluvial clay and clay fill above groundwater at the Children's Hospital site. This methodology is outlined below:

1. Estimate the shear stress in the soil induced by the earthquake (peak horizontal ground acceleration value);
2. Estimate the volumetric strain of the soil based on this shear stress value using the chart developed by Tokimatsu and Seed (1987);
3. Adjust the unidirectional volumetric strain to account for the multidirectional shaking effects of an earthquake;
4. If the moment magnitude earthquake being considered is different than 7.5, adjust the volumetric strain by a ratio value based on the earthquake magnitude; and
5. Based on the thicknesses of the dry cohesionless soil layers and the adjusted volumetric strain values, compute the total predicted settlement.

The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.76g. This PGA represents the design response spectrum ground motion at the site, based on our probabilistic seismic hazard analyses, as discussed in Appendix C. An anticipated earthquake magnitude (maximum moment magnitude) of 7.26, representing the upper bound M_{max} for a full rupture of the Hayward-Rogers Creek fault, was used in conjunction with the PGA value assigned in the dry dynamic densification analyses.

The calculations for the soil dry densification evaluation are included in Appendix D. The soils above groundwater at our site were primarily clay. No dynamic settlement was identified in any of our 7 borings, based on our evaluation. The potential for dynamic densification of unsaturated soils above groundwater at this site is considered low.

5.2.4 Lateral Spreading

Lateral Spreading occurs where the contact between a layer of liquefiable material and the material below is sloped. Saturated sands lose their strength during an earthquake and

become fluid-like and mobile. As a result, the ground may undergo large permanent displacements that can damage underground utilities and well-built surface structures. Lateral spreading involves displacement of large blocks of ground down gentle (greater than 0.1 percent) slopes or towards the open face of slopes such as a stream channel. In areas where there is no open face, buckling of the overburden is often observed.

Historic earthquakes in the Bay Area have produced lateral spreading features. However, large-scale lateral spreading is considered unlikely at this site because the site is essentially level and the probability for liquefaction at the site is considered low, except in localized areas (discussed in Section 5.2.2). In addition, the transitions between cohesive layers and granular layers were observed to be gradual and occur at varying depths. Liquefaction-induced lateral spreading along the contact between cohesive materials and granular materials is unlikely. Liquefaction induced lateral spreading into buried paleochannels is unlikely because the paleochannels are typically dense and have a low potential for liquefaction, as discussed above.

5.2.5 Tsunami and Seiche

During a major earthquake, strong waves such as tsunamis or seiches may be generated in large bodies of water and may cause damage to structures affected by them. A tsunami (or seismic sea wave) is an open ocean phenomenon caused by faulting, volcanism or other abrupt movements on the ocean floor. A seiche is a wave that occurs in an enclosed basin as a result of fault displacement in the basin bottom, large landslides into the basin, or from periodic oscillation or sloshing of the water in the basin in response to ground shaking.

The potential for tsunami and seiche hazards at this site is considered remote. The site is located at an elevation of about 100 feet according to the project survey, and about 1.8 miles east of San Francisco Bay. There are no large enclosed bodies of water in the vicinity of the site. Damaging tsunamis are not common on the California coast. Most California tsunamis are associated with distant earthquakes (often those in Alaska or South America), not with local earthquakes. Devastating tsunamis have not occurred in historic times in the Bay Area. Because of the lack of reliable information about the kind of tsunami run-ups that have occurred in the prehistoric past, there is considerable uncertainty over the extent of tsunami run-up that could occur at the site. However, the Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region (Ritter & Dupre, 1972) does not indicate that the site is in an area that may be inundated by a tsunami with a run-up of 20 feet at the Golden Gate Bridge.

5.2.6 Landsliding

A landslide is a movement of a mass of soil down a steep slope when the soil cannot support the weight of overlying soil or rocks. Landslides vary in size and rate of movement. Slides can occur slowly over time or suddenly. Areas susceptible to landslides are those where masses of soils are weakly supported because of natural erosion, changes in ground water or surface water patterns, or human activities such as undercutting. Heavy rains or seismic shaking can also trigger landslides.

A large earthquake in the San Francisco Bay Region may cause movement of active slides and could trigger new slides. However, no significant slopes are present at the site, and evidence of previous or active slides was not observed during our site investigation. The site is not within a mapped area of existing or potential slope instability. The site is not located within a State of California designated seismically induced landslide hazard zone, as shown on Plate 9 – CGS Hazard Zone Map. The site is not located within a City of Oakland landslide Hazard Zone (Miles and others, 2001). The potential for landsliding at the site is considered low.

The nearby Highway 24 entrance ramp to the east of the site includes retaining walls and embankment slopes with an approximate inclination of 2.5:1 (horizontal to vertical). This roadway is about 25 to 30 feet higher than the surrounding relatively flat terrain. We have assumed for purposes of this study that the Caltrans structures were engineered and constructed such that a low potential for sliding exists at the slope. An evaluation of the stability of the Caltrans Highway 24 entrance ramp embankment is beyond the scope of work for this study.

5.2.7 Other Shaking Hazards

Ground shaking during an earthquake could cause objects within buildings that are not rigidly attached to the building structure (such as desks and bookshelves) to undergo some differential movements with respect to the structure. Building construction should include designs that reduce such potential differential movements and the adverse effects of such movements where they cannot be prevented.

We have also considered the possibility of the occurrence of other seismic hazards. Differential compaction is considered unlikely because of the abundance of very stiff clayey soils and bedrock and low potential for liquefaction. Ground cracking and lurching can be caused by any of the phenomena discussed above.

5.3 OTHER GEOLOGIC HAZARDS AND COMPLICATING SITE CONDITIONS

5.3.1 Flooding and Inundation Due to Dam or Embankment Failure

The site is located at an elevation of approximately 100 feet (relative to NAVD88) and about 1.5 miles west of San Francisco Bay. The potential for flooding from San Francisco Bay is remote. The site is not within the flood plain of any nearby water body. The National Flood Insurance Program designates flood prone areas. The site is designated as Zone C, an area of minimal flooding (FEMA, 1982). Flooding resulting from ponding of locally heavy rainfall was not included in our evaluation.

Dams and reservoirs, which hold large volumes of water, represent a potential hazard from failure caused by ground shaking. The site is in the path of inundation from failure of Lake Temescal dam, approximately 2 miles (3.2km) northeast of the project site (owned by the East Bay Regional Park District). The state's Division of Safety of Dams last inspected this dam in July 2002. At the time, no issues necessitating corrective action were presented and the dam was "judged satisfactory for continued operation" (City of Oakland General Plan, 2004).

5.3.2 Debris Flow

The project area is about 2 mile from the nearest zone designated as a debris-flow source area (Oakland Hills) in the Principal Debris-Flow Sources Areas in Alameda County map (USGS, 1997), and is therefore not likely to be impacted by debris flows.

5.3.3 Land Subsidence

Land subsidence is a gradual settling or sudden sinking of the Earth's surface caused by subsurface movement of earth materials. The principal causes of subsidence are aquifer-system compaction, drainage of organic soils, underground mining, hydrocompaction, sinkholes, and thawing permafrost. The conditions needed for these hazards are not known to exist within the project area.

5.3.4 Soil Expansion

The near surface clay fill soils observed at the proposed hospital location have Plasticity Index (PI) ranging from about 15 to 28 percent, based on laboratory testing from current and previous borings at the site. These PI values indicate a moderate to high expansion potential. In addition, an expansion index test was performed as part of the previous study for the adjacent office building and parking structure (Kaldveer, 1990). This test showed an Expansion Index of 115 in the fill material, indicating a high expansion potential. These results suggest that the clayey soils can be classified as "expansive" as defined in Section 1802A.3.2 of the 2007 California Building Code.

Expansive pressures on the floor slabs for the New Patient Tower and proposed pedestrian tunnel are likely to be minimized because of their depth below existing site grade. However, expansive materials surrounding the structures may be subjected to volume changes during seasonal fluctuations in moisture content, which could affect minor slabs-on-grade and landscaping hardscape such as sidewalks and exterior slabs. These improvements could be subjected to volume changes during seasonal fluctuations in moisture content. We have provided recommendations below to reduce the effects of swelling and shrinkage of these materials in later sections of this report.

5.3.5 Corrosion Potential

Corrosion testing was conducted on samples of the surficial material from historical Borings B-104 at a depth of 2½ feet, B-105 at a depth of 15½ feet and B-112 at a depth of 4 feet. The corrosion testing was conducted at CERCO Analytical Laboratory in Pleasanton, California. The corrosivity testing included resistivity analysis, chloride ion concentration, sulfate ion concentration, pH testing, and redox potentials. The corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. The results of the laboratory tests and details of the results are presented in the letter reports provided by CERCO Analytical, Inc., attached in Appendix B.

The results of our corrosion tests generally show slightly more corrosive soils than identified in previous corrosion studies performed at the nearby hospital office and parking structure site by Kaldveer (1990). The Kaldveer study included two resistivity traverses and one sulfate content test. The historic resistivity tests indicated resistivity values ranging from 3,066 to 11,975 ohm-cm; indicating slightly to moderately corrosive conditions. The sulfate test indicated a sulfate content of 0.005 percent; suggesting negligible sulfate attack potential.

Tests for Buried Concrete

Soluble sulfate concentration, chloride ion concentration, and pH tests were performed on the three samples. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures. The table below presents the depth at which the samples were collected and the laboratory test results.

Table 3. Summary of Corrosion Test Results for Buried Concrete

Test No.	Boring	Depth	Sulfate (mg/kg)	Chloride (mg/kg)	pH
1	B-104	2.5 ft	110	52	6.8
2	B-105	15.5 ft	N.D.	N.D.	7.5
3	B-112	4 ft	33	N.D.	5.6

N.D. = none detected

CERCO's evaluation detected sulfate ion concentrations in tested samples ranging from none detected (N.D.) to 110 mg/kg. The sulfate concentrations are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel. Section 1904.3 of the 2007 California Building Code requires that concrete exposed to sulfate containing soils comply with the provisions of ACI 318, Section 4.3. According to ACI 318 sulfate exposure may be considered negligible if water-soluble sulfate in soil is less than 0.1 percent by dry weight. The water-soluble sulfate was measured to be about 110 mg/kg (ppm) or less (0.0011 percent by dry weight). Therefore, the test results suggest Type II Portland cement is suitable for onsite below-grade concrete structures.

CERCO's evaluation detected chloride ion concentrations in tested samples ranging from none detected (N.D.) to 52 mg/kg. The chloride concentrations are determined to be insufficient to attack steel embedded in a concrete mortar coating. Section 1904.4 of the 2007 California Building Code requires that reinforcement in concrete be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.4. Table 4.4.1 of ACI 318 suggests commencing mitigative measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06 percent by dry weight. The chloride content measured in soil samples was about 52 mg/kg (ppm) or less (.00052 percent), which does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures.

CERCO's evaluation indicates the pH of the soils tested ranges from 5.6 to 7.5, which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Soils with a pH less than 6.0 are considered corrosive, and corrosion prevention measures need to be considered for structures placed in this acidic soil.

For specific long-term corrosion control design recommendations, we recommend that a corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures and steel pipe coated with cement-mortar.

Tests for Buried Ferrous Metals

In addition to the pH tests described above, the resistivity and redox tests were performed on the soil samples discussed above. These tests may be evaluated together to provide an indication of the corrosion potential of the soil environment on buried ferrous metals such as steel or cast-iron pipes. The results of these tests are presented in the table below.

Table 4. Summary of Corrosion Test Results for Buried Ferrous Metal

Test No.	Boring	Depth	Resistivity (ohm-cm)	Redox Potential (mV)	pH
1	B-104	2.5 ft	1,300	420	6.8
2	B-105	15.5 ft	1,700	450	7.5
3	B-112	4 ft	1,800	410	5.6

ND = none detected

CERCO classified the soil samples as "corrosive" based on the resistivity measurements. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron pipeline should be protected against corrosion as well.

The measured soil reduction-oxidation (redox) potentials range from 410 to 450 mV and are considered indicative of aerobic soil conditions.

Appendix B presents the complete results of corrosion tests and recommendations provided by CERCO. For specific long-term corrosion control design recommendations, we recommend that a corrosion engineer evaluate the corrosion potential of the soil environment on buried metal.

5.3.6 Naturally Occurring Asbestos (NOA)

Inhalation of asbestos fibers may cause cancer. Most commonly, asbestos occurrences are associated with serpentinite and partially serpentinized ultramafic rocks. Asbestos occurs naturally in certain geologic settings in California. Exposure and disturbance of rock and soil that contains asbestos can result in the release of fibers to the air and consequent exposure to

the public. Asbestos most commonly occurs in ultramafic rock that has undergone partial or complete alteration to serpentine rock (proper rock name serpentinite) and often contains chrysotile asbestos. In addition, tremolite, another form of asbestos, can be found associated with ultramafic rock, particularly near faults. Sources of asbestos emissions include: unpaved roads or driveways surfaced with ultramafic rock, construction activities in ultramafic rock deposits, or rock quarrying activities where ultramafic rock is present.

The bedrock underlying the site has the potential of containing asbestos fibers; however, bedrock is anticipated to be at a depth of about 300 feet below grade and construction activities should not penetrate through the Quaternary alluvium overburden and expose ultramafic bedrock. Our exploration did not expose alluvium derived from ultramafic material. The potential for NOA hazards at the site is low.

5.3.7 Radon-222

Radon gas is a naturally occurring radioactive gas that is invisible and odorless. Radon forms from the radioactive decay of small amounts of uranium and thorium naturally present in rocks and soils. Some rock types, such as black shales and certain igneous rocks, can have uranium and thorium in amounts higher than is typical for the earth's crust. Increased amounts of radon will be generated in the subsurface at locations where these rocks are found. Because radon is a gas, it can easily move through soil and cracks in building slabs or basement walls and concentrate in a building's indoor air. Areas with higher amounts of radon in the underlying rocks and soil are likely to have higher percentages of buildings with indoor radon levels in excess of U.S. Environmental Protection Agency guidelines, and incidences of very high indoor radon levels are more likely in these areas. Long term exposures to elevated levels of radon can increase the risk of lung cancer.

In California, the Department of Public Health (CDPH), Radon Program, collects radon test data for buildings throughout the state. The data are maintained in a digital database. The database is used to help CDPH determine areas with excessive indoor radon levels, determine areas that may need testing, and inform the public of the results. This database is organized by zip code, and is **not** to be considered a site specific investigation. The United States Environmental Protection Agency (EPA) recommends that individuals avoid long-term exposures to radon concentrations above 4 picocuries per liter (pCi/L). The Oakland zip code 94118 has had three tests conducted, none returning with levels exceeding 4 pCi/L.

5.4 CONCLUSIONS - GEOLOGIC HAZARDS

Based on the results of our review and evaluation, the primary geologic hazards at the Children's Hospital campus include the high expansion potential of the clay soils, a corrosive soil environment, a potential for moderate to strong ground shaking, and minor localized areas of potential liquefaction. Details regarding specific geologic hazards are presented above. Practically all structures within the San Francisco Bay Area will experience similar ground shaking effects during a moderate to strong earthquake.

5.5 OTHER HAZARDS AND COMPLICATED SITE CONDITIONS NOT ADDRESSED IN THIS REPORT

Under the "Exceptional Geologic Hazards and Complicated Site Conditions" of CGS Note 48 are a number of exceptional items not typically applicable statewide including:

- Phase I & II Environmental Site Assessment;
- Hazardous Materials;
- California Environmental Quality Act;
- Groundwater Quality;
- Onsite Septic Systems; and
- Non Tectonic Faulting and Hydrocollapse of Alluvial Fan Soils;

These exceptional items have not been evaluated in detail as part of this report, and are considered outside of the current scope of Fugro's work.

6.0 DISCUSSIONS AND CONCLUSIONS

We believe that redevelopment of the Children's Hospital is feasible from a geotechnical and geological standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections.

6.1 EXPANSIVE SOILS

As discussed in Section 5.3.4, the moderate to high expansion potential of the clayey surface soils encountered onsite is typically a consideration for foundation design. These materials could be subjected to volume changes during seasonal fluctuations in moisture content. However, we anticipate the Patient Tower will be supported on a structural slab basement floor supported entirely by the deep foundations, and thus expansion pressures of site soils on the building will not be a factor in the Tower foundation design. Note that special design considerations will apply for the design of exterior slabs, as discussed in Section 7.6.3 - Exterior Slabs on Grade.

6.2 UNDOCUMENTED FILL

Our subsurface investigation indicates that about 4 to 8 feet of clay fill with varying amounts of sand and gravel exists at the proposed development locations. This information agrees well with the historic borings drilled at this site that also indicate a fill layer. Where this fill will support appurtenant improvements, we recommend the fill be reworked to conform to engineered fill requirements as discussed below (see section 7.2.3). We assume that the proposed New Patient Tower and pedestrian tunnel will have finished floor elevations below the

layer of fill, and will be supported on deep foundations and not be affected by the undocumented fill conditions.

6.3 GROUNDWATER

Based on our recent borings, as well as previous borings, groundwater was encountered at depths as shallow as 7.5 feet below grade, and fluctuates seasonally. We anticipate that groundwater will be encountered in the excavation for the New Patient Tower, requiring dewatering or sumps to establish a stable working base for construction.

We understand that the lowest level floor slab at the hospital will be about 19.5 feet deep (approximate Elevation +83) and will be below the design groundwater depth of 8 feet. Therefore, the below-grade walls and subfloors should be designed to resist lateral and uplift hydrostatic pressures and appropriately waterproofed to help prevent the potentially damaging migration of water into the structure. Permanent groundwater lowering systems such as subdrains should not be employed because lowering the water table in this area would induce consolidation settlement in the surrounding clay soils and cause settlement of adjacent structures.

However, as an alternative to a permanent drain system, a local system of drains may be used to reduce pressure on the basement floor slab. Our subsurface and historical boring investigation indicates ground water fluctuates between 7.5 and 15 to 20 feet below site grades. A partial-depth drain system installed around the basement could be used in conjunction with structural floor slab held down by drilled piers deriving resistance from skin friction. The goal of the drains would be to prevent hydrostatic uplift pressure from exceeding a design threshold during temporary periods of elevated ground water levels. The drains would not be intended to provide constant dewatering. The drains could function in this capacity if installed to approximately 15 feet below existing grades (approximate Elevation +88 feet), thereby effectively halving the required design uplift pressures. Extending the drains significantly deeper would have the potential to intersect a year-round groundwater level and draining otherwise permanently saturated soils and causing the potential for settlements.

6.4 FOUNDATION SUPPORT

Suitable foundation options for the proposed Patient Tower are discussed below. Our design recommendations for each foundation system are discussed in Section 7. Because the helistop structure is already completed, we are not providing foundation recommendations specifically for new foundations for support of that structure. Recommendations for the SB ground floor interior slab-on-grade are presented below.

The New Patient Tower should be supported on a drilled pier foundation system with a structural slab designed to resist hydrostatic uplift, depending on the needs of the structural engineer. A spread footing foundation system is not feasible because of the magnitude of the structural loads and because the basement level will be below groundwater. Isolated drilled piers could be used in combination with a structural floor slab to resist uplift pressures. We do not believe that driven piles are a feasible foundation option because the active hospital and

residential neighborhood are sensitive to noise and vibration. Proprietary deep foundation alternatives may also be feasible, but research of these is beyond the scope of this study.

6.5 CONSTRUCTION CONSIDERATIONS

6.5.1 Existing Structures and Foundations

Existing structures, associated foundation systems and buried utilities exist at the proposed Patient Tower site and will require demolition as part of the construction. The contractor and project structural engineer should be aware that buried obstructions are likely to be encountered during construction of the new Patient Tower, and that foundation layout should take into account the potential for the existing obstructions.

The New Patient Tower site is currently occupied by a multi-story hospital wing that includes a basement level. Information from historical geotechnical reports indicates the existing structure is supported on drilled piers. In addition, basement levels may be encountered that will need to be cut off or removed and backfilled.

In order to prevent "hard spots" beneath any future development at the site, all foundations (including basement walls and deep foundations) should be removed to a depth at least 5 feet below the planned final grade, or a minimum three feet below the bottom of new pier caps, grade beams, or other conflicting new foundation elements, whichever is deeper. (Note that this requirement is not meant to extend to three feet below the bottom of the piers, which will be tipped below existing foundation elements.) Holes should be drilled in any basement slabs left in place to prevent water from becoming trapped in the buried concrete structure. The contractor should prepare detailed documentation of the location, size and depth of all foundations that are not completely removed.

6.5.2 Excavation Shoring and Dewatering

Excavations will be required to construct the below grade portions of the New Patient Tower, to install utilities, to rework undocumented fill and to remove locally weak or unsuitable soils, if encountered. Based on the currently proposed location and likely depth of the New Patient Tower basement, excavations will likely extend below the foundation influence of nearby structures. Underpinning of neighboring structures may not be needed, but shoring will likely be needed to limit the size of the excavation because of the depths of excavation required and spatial constraints. The shoring system must be designed to resist loads from foundations of neighboring structures. All excavations that will be deeper than 5 feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards.

Groundwater was observed to vary significantly in our borings and monitoring wells, and should be anticipated during excavations deeper than about 8 feet. To help reduce the impacts of water drawdown on surrounding facilities, a relatively impervious shoring system, such as an interlocking steel sheet pile system, should be installed. The shoring system should penetrate deep enough into the relatively impermeable clay soils below the clayey sand and gravel layer beneath the base of the excavation such that the shoring walls would act as a water cutoff.

Given the relatively large size and depth of excavation that will be required, we anticipate that a dewatering system that includes a series of wells may be required in addition to sump pumps. In addition, grading of the excavation subgrade to direct water toward sump pumps or to perimeter or interior ditches may be required to help drain remaining surface water. We anticipate that with a properly installed sheet pile and dewatering system, groundwater within the excavation can be effectively dewatered and controlled. The clayey sand and gravel soils above the clay encountered in our investigation are generally medium dense to dense; therefore, we anticipate that sheet piles could be driven through these materials. Localized areas of very dense clayey sand and gravel were also encountered across the site that could make driving sheet piles more difficult.

The performance of the shoring and dewatering systems are highly dependent on the construction methods and procedures employed. The design of the necessary shoring and dewatering systems, as well as the protection of existing facilities, site improvements and utilities should be the responsibility of the contractor. The project geotechnical and structural engineers should review the proposed shoring system prior to installation.

6.5.3 Earthwork

If earthwork is performed during the dry season, moisture conditioning will be required to raise the in-situ moisture contents to near optimum moisture content (per ASTM D1557). If earthwork is performed during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

6.5.4 Height Restrictions During Construction

We understand that the hospital may maintain operations during construction. It is possible that during construction, height restrictions for construction cranes or rigs may be imposed to maintain clearance for air traffic accessing the existing helistop/helipad structure. If pier installation rigs are anticipated to extend above the height restrictions, a possible solution could be to complete the excavation for the proposed subgrade floor, lower the rigs into the excavation, and complete pier installation from the subgrade level. If height restrictions will be imposed, they should be included in construction and bid documents.

6.5.5 Construction Quality Control

Careful quality control should be maintained during construction so that the recommendations presented in this report are achieved. Construction observation is necessary to check that the quality control measures are being properly implemented.

7.0 RECOMMENDATIONS

The recommendations presented in this report should be incorporated in the design and construction of the project to help minimize soil or foundation related problems. Detailed earthwork and foundation recommendations for use in design and construction of the project are presented below. We recommend that our firm review the final design and specifications to check that the earthwork and foundation recommendations presented in this report have been properly interpreted and implemented. Fugro can assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications.

7.1 SEISMIC DESIGN CRITERIA

7.1.1 Seismic Parameters

The proposed structure should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practice. This section presents seismic design parameters for use with the 2007 California Building Code (CBC). The 2007 CBC Seismic Parameters presented in Table 5, below, were determined by the simplified method for spectrum development in accordance with ASCE 7-05 Section 11.1. These parameters were used in the development of the final design response spectrum ordinates. The CBC refers to the design code by American Society of Civil Engineers (ASCE 7-05) for the development of site-specific response spectra for critical facilities within 10 km of an active fault. Site-specific response spectra were developed in accordance with Section 21.2 of ASCE 7-05. The values of S_{DS} and S_{D1}, **for use in design** of the New Patient Tower, from of our site-specific analyses, are included in **Table 6**, below.

Table 5. Seismic Parameters

Parameter/Coefficient Description	2007 CBC Reference	Parameter/ Coefficient	Value
MCE for 0.2 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(3)	S_s	1.73
MCE for 1.0 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(4)	S_1	0.64
Long-Period Transition Period	ASCE 7-05 Figure 22-15	T_L	8
Soil Profile Type	Table 1613A.5.2	Site Class	D
Site Coefficient	Table 1613A.5.3(1)	F_a	1.0
Site Coefficient	Table 1613A.5.3(2)	F_v	1.5
Spectral Acceleration Parameters **NOT FOR USE IN STRUCTURAL DESIGN**	Equation 16-39	S_{DS}	1.16
	Equation 16-40	S_{D1}	0.64

**Site Located at: Latitude: 37.8380, Longitude: -122.2665

7.1.2 Site Class, Seismic Design Parameters, and Seismic Design Category

The above seismic parameters for the simplified method are presented indicating a Soil Profile Type, Site Class D based on an average site shear wave velocity (v_{s_30}) of 270 meters per second (discussed in detail in Appendix C). Site Class has been determined based on the referenced 2007 CBC Table 1613A.5.2. We note that the discussion of liquefaction potential above indicates the probable effects of liquefaction are confined to an isolated area of the site. Based on the isolated area of potential liquefaction, its minimal impact on the support of the proposed structure, and the minimal impact of liquefaction on propagating ground motions affecting the structure, we have selected Site Class D to calculate the seismic parameters presented above. We note that the code-based simplified method presented in ASCE 7-05 Chapter 11 provides a lower-bound check for site-specific analysis conducted according to ASCE 7-05 Chapter 21.

Table 6. Site-Specific Seismic Design Parameters

Site-Specific Design Spectral Acceleration Parameters	Section 21, ASCE 7-05	**S_{DS}**	**1.178**
	Section 21, ASCE 7-05	**S_{D1}**	**0.968**

The New Patient Tower is assumed to be an Occupancy Category IV Structure according to the 2007 CBC, and should be assigned to Seismic Design Category "D", based on an S_1 value presented above of 0.64 and according to Section 1613A.5.6 of the 2007 CBC.

The SB is not necessarily considered part of the "critical facility" of the hospital but is governed by the site-specific S_{DS} and S_{D1} provided above. Based on the S_1 value, the results of the site-specific analysis, and Tables 1613.5.6(1) and 1613.5.6(2) of the 2007 CBC, structures of occupancy category I, II, and III should be assigned to Seismic Design Category "D".

7.2 SITE PREPARATION AND GRADING

7.2.1 Site Preparation

The site should be cleared of all obstructions, including concrete, asphalt pavement, buried foundations, slabs, utility lines, trees and associated root systems, and debris. Removed concrete, asphalt concrete, and baserock may be reused as fill, provided the material is broken up to meet the requirements in Section 7.2.4 - Engineered Fill Materials. Holes resulting from the removal of root systems of larger trees could extend to depths of 3 feet, and laterally to the drip line of each tree. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with suitable material compacted to the requirements in Section 7.2.5 - Fill Placement and Compaction. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

As previously discussed, existing structures will be demolished prior to the Patient Tower construction. We recommend all foundations and below grade walls be removed to a minimum depth of 5 feet below final planned grade, or a depth of 3 feet below new foundation elements such as pier caps and grade beams, whichever is deeper. The contractor should prepare detailed documentation of the location, size and depth of all foundations that are not completely removed.

Prior to backfilling any basement level, holes should be drilled in the floor slab of the basement to prevent water from becoming trapped in the buried concrete structure. If the basement level is not completely removed, the buried concrete will act as a "bathtub" and will collect water. In order to prevent this water buildup, we recommend that holes be created in the floor slab to allow water to pass through. We recommend the holes be installed on a grid pattern with a maximum center-to-center spacing of 10 feet. The holes should be a minimum of 6-inches in diameter.

After clearing, the portions of the site containing surface vegetation or organic laden topsoil should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by the Geotechnical Engineer at the time of construction. Stripped materials should be removed from the site, or stockpiled for later use in landscaping, if approved by the owner.

7.2.2 Subgrade Preparation

Following excavation to the required grades, soil subgrades in areas to receive engineered fill (as defined in Section 7.2.4) or slabs-on-grade should be scarified to a depth of at least 6 inches, moisture conditioned to at least slightly above optimum moisture content, and compacted to at least 95 percent of the soil's maximum dry density. The top 6 inches of subgrade in areas to receive pavements or shallow spread footings should be moisture conditioned and compacted to at least 95 percent of the soil's maximum dry density. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. If the subgrade is allowed to become dry, it should be moisture conditioned to eliminate shrinkage cracks.

In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry. Conversely, it may require that scarification and aeration be performed in any soils that are too wet. There was no indication of weak or unstable soils in our borings; however, after the removal of existing building and pavements, the exposed subgrade materials may be above their optimum moisture content, and may be locally unstable. If areas of unstable soils are encountered at the time of construction, the geotechnical engineer should review conditions in the field and provide recommendations for stabilization procedures.

7.2.3 Reworking Undocumented Fill

As previously discussed, about 4 to 8 feet of undocumented fill was observed in our borings at the site. In areas designated for support of exterior flatwork or pavements, where the fill is not completely removed by excavation, the undocumented fill material should be scarified

to a depth of 12 inches, moisture conditioned to slightly above optimum moisture content, and compacted to at least 95 percent of the soil's maximum dry density as determined by ASTM D1557 (modified proctor test). Excavated material may be used as engineered fill elsewhere on the site, provided the material is placed in accordance with the requirements of Section 7.2.5 – Fill Placement and Compaction.

7.2.4 Engineered Fill Materials

All fill placed at the site should consist of engineered fill meeting the requirements presented in this report, except for landscaping materials which are placed on level ground. On-site soil below the stripped layer and having an organic content of less than 3 percent by volume can be used as fill where needed except where "non-expansive" import is required beneath slabs-on-grade. All engineered fill placed at the site, including on-site soils, should not contain rocks or lumps larger than 4 inches in greatest dimension and contain no more than 15 percent larger than 2.5 inches.

"Non-expansive" fill should be predominantly granular, have an organic content of less than 3 percent by volume, should have a liquid limit less than 40 percent, have a plasticity index not exceeding 15, and should contain no environmental contaminants or debris. All imported fill should consist of "non-expansive" fill, and satisfy California Environmental Protection Agency's Department of Toxic Substances Control (Cal EPA DTSC) guidelines for clean, import fill material.

7.2.5 Fill Placement and Compaction

All engineered fill should be compacted to at least 95 percent of the soil's maximum dry density as determined by ASTM Designation D1557 (latest edition). The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent of the soil's maximum dry density. Fill material should be spread and compacted in lifts not exceeding 8 inches in pre-compacted thickness. The moisture content of the natural on-site expansive clayey soils reused as fill should be 2 to 3 percent above the optimum moisture content for the soil at the time of compaction. In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet.

7.2.6 Trench Backfill

Pipeline trenches should be backfilled with materials satisfying the criteria described above for fill, placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts may be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Onsite soil used for trench backfill should be compacted to at least 90 percent of the soil's maximum dry density by mechanical means only (jetting should not be permitted). Sand can be used for trench backfill if the sand is compacted to at least 95 percent of the soil's maximum dry density and sufficient water is added during backfilling operations to prevent the soil from "bulking"

during compaction. The upper 3 feet of trench backfill below slab and pavements should be compacted to at least 95 percent of the soil's maximum dry density.

Where utility trenches backfilled with sand enter building areas, they should be backfilled by an impermeable soil plug that is at least 2-feet thick. The plug should be located at the perimeter of the foundation. The plug should be keyed at least 12 inches into the sides and bottom of the trench and should extend to within 12 inches of the finished grade. This should help to minimize moisture change in the moderately to highly expansive clays beneath the slabs. Where sand backfilled utility trenches cross planter areas and pass below pavements or concrete sidewalks, they should be plugged as described above to minimize soil volume change below asphalt and concrete areas.

7.2.7 Surface Drainage

Positive surface gradients should be provided adjacent to the buildings to direct surface water away from foundations and slabs toward suitable discharge facilities. Water from roof downspouts should be carried away from the buildings in solid pipes and discharged into suitable drainage facilities. Surface water should not be allowed to pond adjacent to the structures or on pavements.

7.3 TEMPORARY SHORING AND CONSTRUCTION SLOPES

7.3.1 General

Construction may require either temporary slopes or shoring for excavations as needed. If site constraints are not present, temporary construction slopes may be used. However, where site constraints are present, or at the contractor's option, temporary shoring should be used. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary construction slopes and shoring system. In selecting the temporary slope, the contractor should evaluate the potential of shallow soil movement, which may potentially jeopardize the existing nearby facilities.

7.3.2 Temporary Construction Slopes

Fugro recommends that the contractor, or his specialty subcontractor, design temporary construction slopes to conform to the OSHA "Guidelines for Excavations and Temporary Sloping." The contractor, or responsible subcontractor, should determine temporary slope inclinations based on the subsurface conditions exposed at the time of construction.

For planning purposes, the onsite soils should be categorized as OSHA Type C.

If temporary slopes are left open for extended periods of time, exposure to weathering and rain could have detrimental effects such as sloughing and erosion. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes and that the temporary slopes are protected from excessive drying or saturation during construction. No temporary excavation slopes should extend within 10 feet of existing foundations.

7.3.3 Temporary Shoring

If open-excavation is not possible because of site constraint and proximity of existing or new buildings or streets, we recommend that a temporary shoring system be used during any needed excavations. Such shoring should be sufficiently rigid to minimize damage to the adjacent facilities (streets, utilities, neighboring buildings, etc.) caused by deflection of the sides of the excavation. The temporary shoring system must be designed to include loads from foundations of neighboring structures.

We recommend a relatively impervious shoring system, such as an interlocking steel sheet pile system, be installed as discussed in Section 6.5.2. The shoring system should penetrate deep enough into the relatively impermeable clay soils beneath the base of the excavation such that the shoring wall acts as a water cutoff and to provide adequate factor of safety against base heave.

Fugro recommends that the contractor, or a specialty subcontractor, design and install the temporary shoring. We recommend that the geotechnical and structural engineers review the final temporary shoring plan to ensure compliance with the anticipated soil conditions encountered at the site. In addition, we recommend that a geotechnical engineer's representative observe the installation of the temporary shoring systems. The contractor should incorporate all appropriate requirements of OSHA into the design of the temporary shoring system.

7.3.4 Construction Dewatering

The excavation for the basement will extend below the design groundwater level and will require dewatering to provide dry working conditions and help prevent instability of the bottom of the excavation during construction. We recommend that the groundwater be drawn down at least 2 feet below the bottom of the lowest excavation. We acknowledge that in areas underlain by more clayey soil, it may be difficult to draw the groundwater down to these depths. In these cases, other measures such as grading and directing the water to sump pumps and/or drainage ditches within the excavation may be more appropriate.

The dewatering system should be designed and installed by a specialty contractor who is familiar with similar subsurface conditions. The Geotechnical Engineer should review and evaluate if the proposed dewatering system and approach may negatively impact the adjacent structures. The main considerations that must be factored into the dewatering design are settlement and decreased stability of foundation for the surrounding existing structures that could be induced by lowering the groundwater table through dewatering.

7.4 FOUNDATIONS

7.4.1 Existing Foundations at Helistop

We understand the existing helistop structure will be enclosed to form the SB, a remodel to provide storage and some office space. According to conversations with the project structural engineer, we understand the helistop structure is supported on 24-inch diameter drilled pier foundations extending approximately 30 feet below site grades. The piers were designed with

an allowable (dead plus live) skin friction of 750 pounds per square feet and 5000 psf end bearing.

The end-bearing recommendation varies significantly from what Fugro has recommended in later sections of this report for similarly-sized piers for the New Patient Tower. The difference between the two sets of recommendations has its derivation in the constructability of piers for end-bearing, and the associated settlement. Since the helipad is a flexible structure, expected settlements (typically 0.5 to 5 percent of pier diameter to mobilize end bearing in addition to any slough compaction) may not have adversely affected the structure. Slough at the bottom of a drilled pier excavation can lead to significant settlements caused by structural loads on the pier.

Fugro recommends a review of the original construction documents for the helistop by the design team prior to adding additional loads to the structure by enclosing the existing structure. Additional loads could induce additional settlement (likely less than 1 inch total).

7.4.2 Support of Appurtenant Structures

Spread footings may be used to support appurtenant structures such as the possible pedestrian bridge. Conventional continuous and isolated spread footings should bear entirely on properly compacted engineered fills or on firm, undisturbed native soils. No footings should bear in the undocumented fill material, where the consistency and density can be highly variable. The undocumented fill should be reworked and placed as engineered fill according to the requirements in this report, or the footings should be deepened to extend through the undocumented fill layer. All footing excavations should be examined by our representative prior to placing reinforcement.

Spread footings should be founded at least 24 inches below lowest adjacent finished grade due to the highly expansive nature of the onsite surface soils. Footings located adjacent to other footings (i.e. existing structures) or buried utilities should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trench. Where possible, footings should not bear within the utility easement. Pedestrian bridge footings should not bear within the utility easement.

Footing dimensions should be based on structural requirements. However, we recommend continuous footings be a minimum of 18-inches wide and spread footings be a minimum of 24-inches wide for ease in construction.

The following allowable bearing values may be used for spread footings. These bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Table 7. Allowable Bearing Pressures for Spread Footings on Native Soil Subgrades

Load Condition	Factor of Safety	Allowable Bearing Pressure (psf)
Dead Load	3	2,000
Dead plus Live Loads	2	3.000
Total Loads (including wind or seismic)	1.5	4,000

Footings supported on a minimum of 18 inches of engineered fill may use the increased bearing capacities presented below.

Table 8. Allowable Bearing Pressures for Spread Footings on 18" Engineered Fill

Load Condition	Factor of Safety	Allowable Bearing Pressure (psf)
Dead Load	3	3,000
Dead plus Live Loads	2	4,500
Total Loads (including wind or seismic)	1.5	6,000

Footings designed with the above bearing pressures should have long-term settlement limited to about 1 inch, and differential settlement limited to about ½ inch over a typical column spacing of 30 feet. We reiterate that structures supported by shallow spread footings be structurally isolated from structures supported by deep foundations.

Long-term resistance to lateral loads may be provided by friction along the base of spread footings. A friction coefficient of 0.30 times the dead load may be used to evaluate the allowable frictional resistance along the bottom of footings bearing on subgrades prepared as discussed above. Friction resistance should not be used on the bottom of grade beams.

In addition to friction, a long-term passive pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for lateral load resistance against the face of footings and grade beams perpendicular to the direction of loading where the foundation is poured neat against engineered fill or undisturbed native material. However, a reduced passive pressure of 150 pcf should be used on any portion of the footings, or grade beams within the undocumented fill layer (upper 8 feet below existing grade). Unless confined by concrete or pavement, the passive resistance of the uppermost one foot of soil should be neglected.

7.4.3 Conventional Drilled Pier Foundations

Conventional drilled pier foundations may be used to support the New Patient Tower. For the purposes of this report, "conventional drilled pier foundations" are designed to derive support through skin friction, and should be between 18 inches and 48 inches in diameter. The drilled piers should be cast-in-place, straight piers that develop their load carrying capacity

through skin friction in the stiff clay soils and interlayered clayey sands and sandy clays. Piers located in a group should have a minimum center-to-center spacing of three times the shaft diameter. Pier reinforcing should be based on structural requirements.

Piers supporting at-grade structures should be designed neglecting the upper 8 feet of the pier below existing ground surface because of the undocumented consistency of the fill material and to account for seasonal moisture variations in the highly expansive material that could result in shrinkage of the material away from the piers. Pier capacity should be determined using Table 10, below. The reference elevations provided were calculated assuming a ground surface at Elevation +103 feet.

Piers beneath the basement level may be designed according to the table below. The design depth of the piers should be determined using an ultimate skin friction of 1,400 pounds per square foot (psf) in the upper stiff clay and the interlayered clayey sand and sandy clay to a depth of 50 feet. An ultimate skin friction value of 2,000 psf may be used in the very stiff to hard clays and dense gravels encountered below a depth of 50 feet (approximate Elevation 53 feet). We recommend a minimum factor of safety of two be used for design of friction piers for dead plus live loads.

Table 9. Drilled Pier Design Skin Friction

Elevation (feet)	Soil Type	Ultimate Skin Friction (psf)	Allowable Skin Friction (psf)
+95 to +103	Undocumented Fill	0	0
+53 to +95	Stiff Clay and Interlayered Clayey Sand and Sandy Clay	1,400	700
Below Elevation +53	Very Stiff to Hard Clay and Dense Granular Lenses	2,000	1,000

Our recommended allowable skin friction values may be increased by one-third for all load combinations that include wind or seismic loads. Uplift loads may be resisted using 80 percent of the allowable skin friction values.

Total and differential settlements of drilled pier supported structures should be minimal, less than ½ inch.

Drilled Pier Construction

Pier excavations extending below a depth of about 8 feet will be below the groundwater table and will require special considerations during construction. Excavations may need to be cased to protect the holes from caving. All pier excavations adjacent to existing structures should be cased to prevent caving of the hole caused by nearby footing loads. In addition, we recommend that concrete be placed in the bottom of the drilled holes by the tremie method. Concrete should be designed with a high slump, equal to or greater than 6 inches, to facilitate construction and reduce the potential for free water to be trapped in the pier excavation. Concrete should be placed in all piers the same day that the excavations are completed. We

recommend that the excavation of all piers be performed under the direct observation of the geotechnical engineer to confirm that the pier foundation is constructed in general accordance with the requirements presented above.

Pier Caps and Grade Beams

Pier caps and grade beams should be used to tie together the foundation piers. Pier caps and grade beams within 3 feet of the ground surface should be designed to resist an uplift pressure of 2,500 pounds per square foot (from highly expansive soils), with uplift resistance provided by the drilled piers. Pier caps and grade beams beneath the proposed basement do not need to be designed for expansion pressures, provided the basement floor elevation is at least 5 feet below surrounding site grades (a zone of constant moisture and reduced potential for expansion). Grade beam reinforcement should be designed to meet structural requirements.

7.4.4 Large-Diameter Drilled Piers

Based on conversations with the project structural engineer, we understand that a system of large-diameter drilled piers installed on a grid pattern is being considered. Large-diameter piers have the advantage of better construction quality control, and the pier toe (bottom) can be inspected for slough prior to pouring of concrete. Installation quality control can allow the use of pier end bearing in calculations of pier capacity without settlement due to compaction of slough materials. Piers up to 60 inches in diameter are under consideration. The OSHA minimum diameter for conducting inspection of piers is 30 inches. Holes should be cased during installation, and dewatered as necessary to afford inspection for clean and flat end bearing surface.

Large-diameter piers may be used for support of the New Patient Tower and appurtenant structures. We recommend that one foundation support system (either large-diameter or conventional drilled piers) be used for connected structures. Structures supported by different systems should be structurally isolated, with relatively flexible connections to account for differences in potential settlement.

In addition to the skin friction values presented in Table 10, large-diameter piers may employ end-bearing of up to 18 kips per square foot ultimate capacity. We recommend a factor of safety of two on dead plus live loads. Large-diameter piers should extend a minimum of 30 feet below lowest adjacent soil grade (e.g., for a New Patient Tower pile cap elevation of approximately +80 feet, piers should extend to Elevation +50 feet).

End-bearing pier elements require settlement to mobilize the full resistance. Typically this settlement is approximately 0.5 percent to 5.0 percent of the pier diameter (Reese and O'Neil, 1988, O'Neil and Reese, 1999). The percentage of settlement varies by soil type and fraction of ultimate resistance required for design. We expect that piers founded in the stiff to hard clays or dense granular layers indicated by the site exploration will require settlements on the order of 1.5 percent of the pile diameter to mobilize the full end bearing.

Large-Diameter Drilled Pier Construction

As discussed above, pier excavations extending below a depth of about 8 feet will be below the groundwater table and will require special considerations during construction. Large-diameter piers must be cased and dewatered to prevent caving of the hole, and to allow for inspection of the pier toe. Pier excavation bottoms must be relatively level, firm, and free of slough.

7.5 LATERAL LOADS

7.5.1 Long-Term/Static Lateral Loads

Long-term resistance to lateral loads may be provided by friction along the base of spread footings. A friction coefficient of 0.30 times the dead load may be used to evaluate the allowable frictional resistance along the bottom of foundations in clay. Friction resistance should not be used on the bottom of grade beams or pier caps.

In addition to friction, a long-term passive pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for lateral load resistance against the face of footings, grade beams and pier caps perpendicular to the direction of loading where the foundation is poured neat against engineered fill or undisturbed native material. However, a reduced passive pressure of 150 pcf should be used on any portion of the footings, grade beams or pier caps within the undocumented fill layer (upper 8 feet below existing grade).

The equivalent fluid pressure of 300 pcf can also be assumed to act against twice the project diameter of the drilled pier shaft on the upper 10 feet of the piers. *However, a reduced passive pressure of 150 pcf should be used on any portion of the drilled pier within the undocumented fill layer (upper 8 feet below existing grade).* Additionally, any portion of pier caps and grade beams within the upper 3 feet below grade should be neglected in the passive resistance design because of the expansive nature of the soils. If piers are spaced closer than six diameters on center, the recommended passive pressure should be reduced to account for group effects. We can provide group reduction factors during design once the estimated pier spacing and load requirements are better known.

Note that lateral deflections corresponding to a Δ/H of 0.04 (where Δ equals horizontal deflection and H equals the height of the footing, pier cap or grade beam) are required to mobilize full passive resistance. Compatible deflection-resistance relationships must be considered when evaluating the relative contributions of passive resistance from foundation elements. The table below presents the recommended load-deflection curves to mobilize the passive resistances for different lateral deflections.

Table 10. Passive Soil Resistance (P) versus Displacement (Δ) Relationship

Deflection Ratio (Δ/H)	Passive Soil Resistance Ratio (P/P_{max})
0.00	0.0
0.01	0.5
0.04	1.0
1.00	1.0

7.5.2 Temporary Seismic Lateral Loads – Shallow Foundations

For temporary dynamic loads, a maximum passive resistance of 1,500 psf can be used instead of the passive pressures given above. Passive resistance in the upper 1 foot should be ignored unless the soil is confined by a pavement or slab.

For lateral resistance to seismic loads, friction along the sides of the footing, pier caps and grade beams parallel to the direction of the imposed lateral force may be combined with passive resistance. An allowable uniform frictional resistance of 300 psf, corresponding to a lateral displacement of about ½ inch, may be used to resist lateral seismic loads. *However, a reduced frictional resistance of 150 psf should be used on any portion of the footings, grade beams or pier caps within the undocumented fill layer (upper 8 feet).* Frictional resistance should only be considered applicable to the sides of the footing, pier cap and grade beam parallel to the direction of the imposed lateral seismic force.

7.5.3 Temporary Seismic Lateral Loads – Drilled Piers

We have calculated the lateral load resistance for drilled piers using the liquefied soil cases. Our calculations were based on an assumed maximum lateral displacement of 0.5 inches at the pier head, modeled at the ground surface.

Temporary Seismic Lateral Loads – Conventional Drilled Piers

Displacement, Moment, and Shear diagrams for 24 inch diameter drilled piers are presented on Plates 10a, 10b, and 10c, respectively. Table 11 presents the maximum shear (lateral capacity) and moment values for the piers, assuming 500 kips axial compression load. The capacities provided assume a minimum spacing of 3 pile diameters for lateral load conditions, and account for the associated reduction factors. Friction resistance along the bottom of the pier caps and grade beams should be ignored in evaluating the lateral seismic resistance.

Table 11. Lateral Capacity of Conventional Drilled Piers

Load Case	Maximum Shear Capacity	Maximum Moment	Depth to Fixity
Free-Head, 0.5" Displacement	12.5 kips	955 kip-inches	15 feet
Fixed-Head, 0.5" Displacement	28.0 kips	2,570 kip-inches	24 feet

Temporary Seismic Lateral Loads – Large Diameter Drilled Piers

Displacement, Moment, and Shear diagrams for 60 inch diameter drilled piers are presented on Plates 11a, 11b, and 11c, respectively. Table 12 presents the maximum shear (lateral capacity) and moment values for the piers, assuming 600 kips axial compression load. Based on the displacement results, piers should be reinforced throughout their installed length. Friction resistance along the bottom of the pier caps and grade beams should be ignored in evaluating the lateral seismic resistance.

For the lateral capacities provided, piles spaced less than six pile diameters, center to center, should employ a reduced lateral capacity. Piers spaced three diameters apart should employ reduction factors on lateral capacities presented below of 0.8 for the leading row, 0.4 for the second row, and 0.3 for subsequent rows. Linear interpolation may be used to approximate reduction factors for pier spacing between three and six diameters.

Table 12. Lateral Capacity of Large-Diameter Drilled Piers

Load Case	Maximum Shear Capacity	Maximum Moment	Depth to Fixity
Free-Head, 0.5" Displacement	155 kips	15,000 kip-inches	--
Fixed-Head, 0.5" Displacement	295 kips	40,000 kip-inches	--

7.6 CONCRETE SLABS

7.6.1 Patient Tower Structural Slab Foundation

The below-grade walls and floor of the New Patient Tower and any appurtenant, below-grade structure's lowest floor level should be waterproofed and designed to resist hydrostatic uplift pressures. Appropriate slab reinforcement should be provided in accordance with the anticipated loading of the slab, including hydrostatic uplift pressures. Hydrostatic pressure should be calculated assuming a design groundwater elevation of +93 feet. Hydrostatic uplift pressure can be resisted by the dead load of the structure or by drilled piers. Drilled piers may be used in combination with a structural slab, if additional uplift resistance is needed.

The structural slab and waterproofing may be placed directly on native subgrade soil. The subgrade should be proof-rolled (i.e., compacted) to provide a firm surface for constructing steel reinforcement and placement of slab concrete.

7.6.2 Interior Slabs-on-Grade – Services Building

We recommend that the non-structural, interior slab-on-grade for the SB remodel be supported on a minimum of 18 inches of select, predominantly granular "non-expansive" fill meeting the requirements discussed above in Section 7.2.4. The non-structural slab should not be located deeper than 6½ feet below existing site grades (approximate Elevation +94½ feet) to avoid intersecting design groundwater elevation. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. However, the slab should be reinforced with a minimum of No. 4 bars on 18-inch centers, both ways, because of the expansive soils at the site. Additionally, interior slabs should be separated from the pier foundations and grade beams, where used, so that expansion pressure on the foundations does not cause additional damage to the slab. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

If migration of moisture through the floor slab is undesirable, a moisture retarder system should be provided between the slab and subgrade. A high quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., .012 perms) per ASTM E 96 (e.g., 15-mil thick "Stego Wrap Class A") may be used to reduce the potential for moisture migration through the slab. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing. The vapor retarder or barrier should be placed directly under the slab, or at the structural engineer's option, the retarder may be covered with 2 inches of sand. Sand, if used, should be lightly moistened just prior to placing the concrete.

7.6.3 Exterior Slabs-on-Grade

As previously discussed, the onsite moderate to highly expansive surface soils could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs, sidewalks, and pavements should be anticipated. This movement could result in damage to the slabs, sidewalks, and pavements that might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

Exterior slabs such as sidewalks could be reinforced with steel reinforcing bars in lieu of wire mesh to minimize the impact of expansion pressures.

Walkways and pavement curbs and gutters should be supported directly on properly prepared native soils. Eliminating rock base beneath slabs will reduce the potential for migration of landscape irrigation water into pavement and walkway subgrade. Curbs should extend to the bottom of the pavement and baserock layer. One to two days prior to placing

concrete, subgrade soils should be soaked to increase their moisture content to 3 to 5 percent above laboratory optimum moisture (ASTM D1557). The water content of subgrade soils should be verified by field-testing by the Geotechnical Engineer prior to placing concrete.

To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and/or a "drip" irrigation watering system be used. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the tree from slabs or pavements to reduce the effects of tree roots on these improvements.

7.7 RETAINING/BELOW GRADE WALLS

Below grade walls must be designed to resist lateral earth pressures and any additional lateral loads caused by surcharging or seismic pressure. If the walls are designed to resist lateral loads through passive pressure, we recommend they be designed for the equivalent fluid pressures discussed in Section 6.3.4 – *Lateral Loads*. If below grade walls are not designed to resist passive lateral loads, they should be designed as discussed below.

The following recommendations were developed based on the clayey soils encountered at the New Patient Tower location. Retaining wall backfill less than 5 feet deep should be compacted using light compaction equipment to at least 90 percent of the soil's maximum dry density. Backfill greater than 5 feet deep should be entirely compacted to at least 95 percent of the soil's maximum dry density. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

7.7.1 Unrestrained Walls

Unrestrained walls (walls that are free to move/rotate) should be designed for active earth pressure. We recommend unrestrained walls be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot (pcf). Schematics of static and seismic lateral pressures on unrestrained retaining walls can be found on Plates 12a and 12b, respectively. Equivalent fluid pressures are based on:

- Walls backfilled with onsite, reworked lean clay fill compacted to the requirements in this report;
- The walls are fully back drained to prevent the development of hydrostatic pressure;
- The walls are capable of moving/rotating sufficiently for active pressure conditions to develop; and
- Level backfill. Walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.

In addition to soil pressure, a uniform seismic pressure of 30H (psf) should be applied to the entire wall height, where H is the height of backfill above the top of the wall footing, in feet.

Unrestrained walls should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load. The design surcharge should include the anticipated surcharge caused by vehicular traffic and construction equipment. Surcharge loads from adjacent structures need to be considered if the proposed walls extend below the zone of influence of adjacent foundations. The zone of influence of adjacent foundations can be defined as the area below an imaginary 1.5:1 (horizontal to vertical) line extending downward from the bottom of footings near the wall.

Wall Drainage

The recommended lateral pressures for unrestrained walls herein assume walls are fully back drained to prevent the buildup of hydrostatic pressures. Adequate drainage could be provided by means of either weep holes with permeable material installed behind the walls or by means of a system of subdrains.

For the subdrain system, the top of the perforated pipe should be below the bottom of the adjacent slab or grade at the toe of the wall. Drains should consist of a drain rock layer at least 12 inches thick that extends to within 2 feet of the ground surface. Four-inch diameter perforated plastic pipe should be installed (with perforations down) along the base of the walls on a two-inch-thick bed of drain rock. The pipe should be sloped to drain by gravity to a suitable drainage facility. Drain rock should conform to Caltrans specifications for Class 2 permeable material. A more open-graded material, such as ¾-inch crushed rock, could be used provided the rock is wrapped in a geotextile filter fabric (Mirafi 140 N or equivalent) to reduce the migration of fine-grained soils into the drain rock. Paving or a 2-foot thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration. Drainpipes should outlet to an appropriate drainage facility.

Alternatively, wall back-drainage can be provided by prefabricated drainage material (such as Miradrain 6000 or an approved alternative). The drainage material can be installed on the back (soil) face of the basement wall and should terminate at a 4-inch diameter perforated plastic pipe surrounded by at least 6 inches of drain rock as defined above.

7.7.2 Restrained Walls (Basement Walls)

Restrained walls (walls that are prevented from moving/rotating) should be designed for at-rest earth pressure as well as hydrostatic pressure. We recommend restrained walls be designed to resist an equivalent fluid pressure of 60 pounds per cubic foot (pcf). Recommended static design lateral pressures for restrained walls are presented on Plate 13a. Recommended seismic design lateral pressures for restrained walls are presented on Plate 13b.

These equivalent fluid pressures are based on:

- Walls backfilled with onsite, reworked lean clay fill compacted to the requirements in this report;
- The walls are prevented from moving/rotating such that at-rest pressure conditions develop; and

- Level backfill. Walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.

For seismic loading conditions only, we recommend that soil pressures be reduced to the active (unrestrained) condition plus the seismic increment. For the seismic case, restrained walls should be designed to resist an equivalent fluid pressure of 40 pcf for soil loading, plus a uniform seismic pressure of 38H psf, where H is the height of backfill above the top of the wall footing, in feet.

Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.50 times the anticipated surcharge load. The design surcharge should include the anticipated surcharge caused by vehicular traffic and construction equipment. Surcharge loads from adjacent structures need to be considered if the proposed walls extend below the zone of influence of adjacent foundations. The zone of influence of adjacent foundations can be defined as the area below an imaginary 1.5:1 (horizontal to vertical) line extending downward from the bottom of footings near the wall.

Hydrostatic Pressure

Below-grade walls should be waterproofed and designed to resist hydrostatic pressures. The pressures recommended above can be used for the portion of the wall above the design groundwater level (8 feet below grade); below the designed groundwater level, restrained walls should be designed for an equivalent fluid pressure of 95 pcf.

We recommend all basement walls be waterproofed. If the permanent wall will be constructed immediately against a temporary wall, the waterproofing must be applied on the face of the temporary wall.

Hydrostatic Pressure Reduction for Retaining Walls

Documentation of seasonal fluctuations of groundwater levels have forced a design groundwater recommendation of 8 feet below site grades (approximate Elevation 95 feet). We expect groundwater levels will be elevated to those levels only during the wettest times of the year, and recommend that a system of sumps and drains can be used to decrease the hydrostatic pressures on the below-grade retaining structures. At the discretion of the project structural engineer, a drainage system could be installed as deep as Elevation +88 feet (approximately 15 feet below site grades). This system should be designed with sumps, perforated pipe, drainage material to collect water during elevated groundwater conditions, effectively reducing the hydrostatic loads on the below grade retaining structures. It should be noted that the below-grade walls should still be adequately waterproofed as discussed above.

7.8 PAVEMENTS

We have developed the following pavement design recommendations based on an R-value of 5, based on the critical expansion potential of the clayey soils at the site and R-value tests performed on bulk samples collected during our investigation. We developed the following alternative preliminary pavement sections based on Chapter 630 of the State of California

Department of Transportation Highway Design Manual and assumed traffic indices. Pavement designs for pavement life of 11 to 20 years are presented below.

Table 13. Recommended Pavement Design Alternatives

Location	Anticipated Pavement Life (years)	Pavement Components		Total Thickness (inches)
		Asphalt Concrete (inches)	*Caltrans Class 2 Aggregate Base (inches)*	
Automobile Parking (T.I. = 4.5)	20	2.5	9.0	11.5
Automobile Access (T.I. = 5.5)	20	3.0	12.0	15.0
Heavy Truck Access (T.I. = 6.5)	20	3.5	15.0	18.5

The traffic indices used in our design were established assuming a typical mix of automobile and "delivery or garbage" truck type of use in the proposed development once construction has been completed. However, if the pavements are planned to be placed prior to, or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. Therefore, if the pavement sections will be used for construction access, our firm should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use. If requested, we could provide recommendations for a phased placement of the asphalt concrete to minimize the potential for mechanical scars caused by construction traffic on the finished grade.

The traffic indices should provide the indicated pavement lives with only a normal amount of pavement maintenance. Selection of the design traffic parameters, however, was based on engineering judgment, and not on an equivalent wheel load analysis developed from a traffic study or furnished to us.

In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils and trench backfill should be protected against saturation. Planned concrete sidewalks, driveways, and curb and gutters should be supported directly on the properly compacted native soils. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas and the pavement section. In addition, a compacted impermeable soil plug should be constructed within any lateral or other trench backfill that passes beneath the curb and gutter and under the adjoining pavement. In addition, water should never be allowed to pond behind the curb and gutter during or after the completion of construction.

The Aggregate Base for use in flexible pavements should conform to Caltrans Standard Specification Section 26-1.02A for Class 2 Aggregate Base. The Aggregate Base used in the pavement sections should be compacted to 95 percent of the soil's maximum dry density (ASTM D1557) and should be firm and unyielding.

7.9 ADDITIONAL GEOTECHNICAL SERVICES

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations. The analyses, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from the subsurface explorations conducted for the project, and upon the conditions existing when services were conducted. Variations of subsurface conditions from those analyzed or characterized in the report are possible, as may become evident during construction. In that event, it may be advisable to revisit certain analyses or assumptions.

We recommend that Fugro be retained to provide geotechnical services during site grading and foundation installation to observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered. During construction, our field engineer should observe and/or test the following:

- Soil conditions exposed by site grading and foundation excavations, to check that they are consistent with those encountered during the field exploration,
- Installation of temporary shoring,
- Pier foundation excavations,
- Pavement subgrade preparation, and
- Fill placement and compaction, including backfill of utilities and compaction of aggregate base.

8.0 LIMITATIONS

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted, local and current geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The analyses and recommendations contained in this report are based on the data obtained from the subsurface explorations conducted for this study and relevant previous explorations at the site. These explorations indicate subsurface conditions only at specific locations and times, and only to the depths explored. Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our analyses of the observed conditions. If conditions other than those described in this report are encountered, we should be notified so that we can provide additional recommendations, if warranted.

This report has been prepared for the exclusive use of Children's Hospital and Research Center Oakland and their consultants for specific application to the Children's Hospital development as described herein. In the event that there are any changes in the ownership, nature, design, or location of the proposed project, or if any future additions are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) the project changes are reviewed by Fugro, and 2) conclusions and recommendations presented in this report are modified or verified in writing. Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

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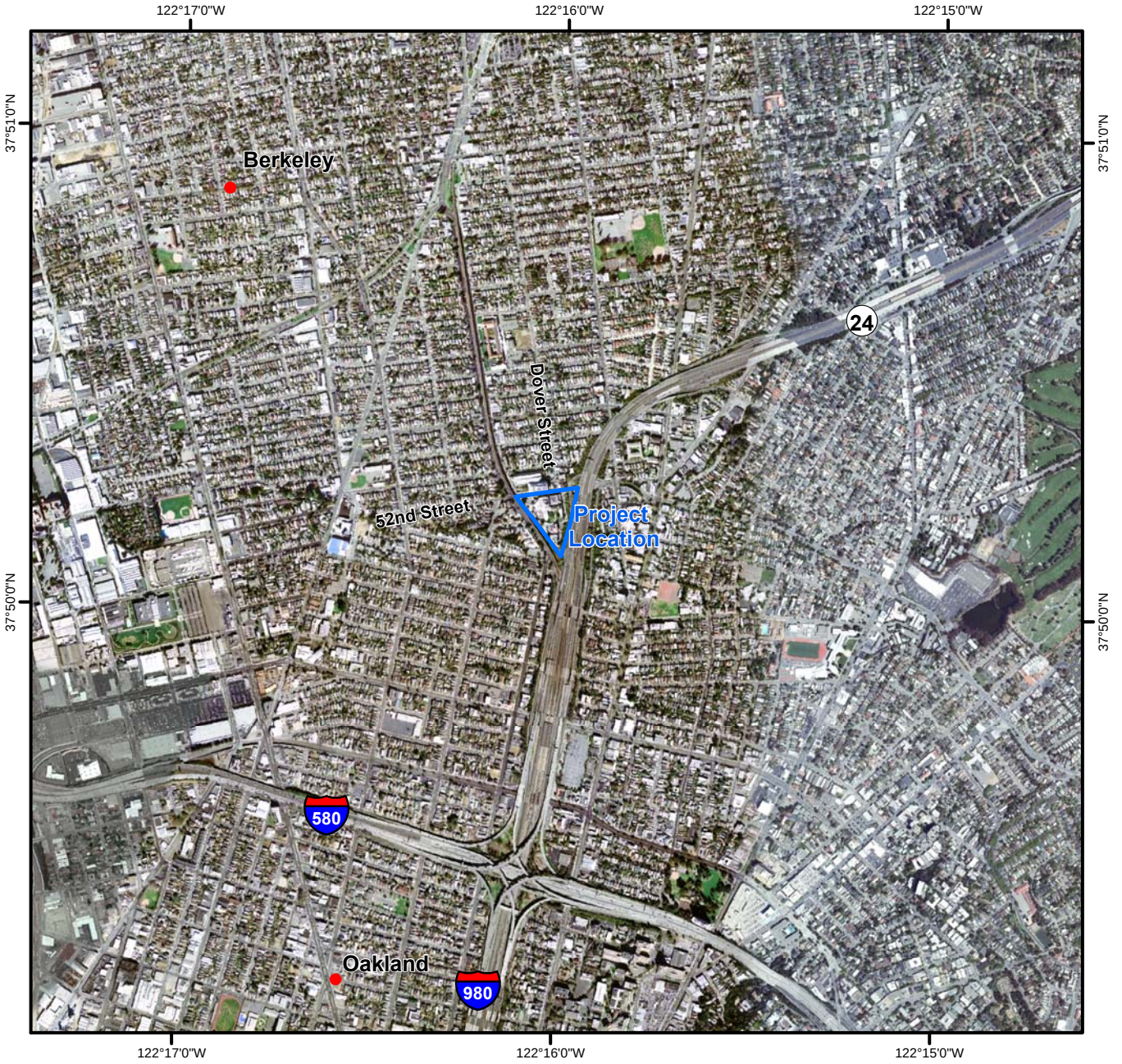
PLATES



Source: Aerial photograph obtained from Google Earth

Site Location:
Latitude: 37.8380°
Longitude: -122.2665°

N

1:24,000

0 1,000 2,000 4,000 Feet

VICINITY MAP
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 1

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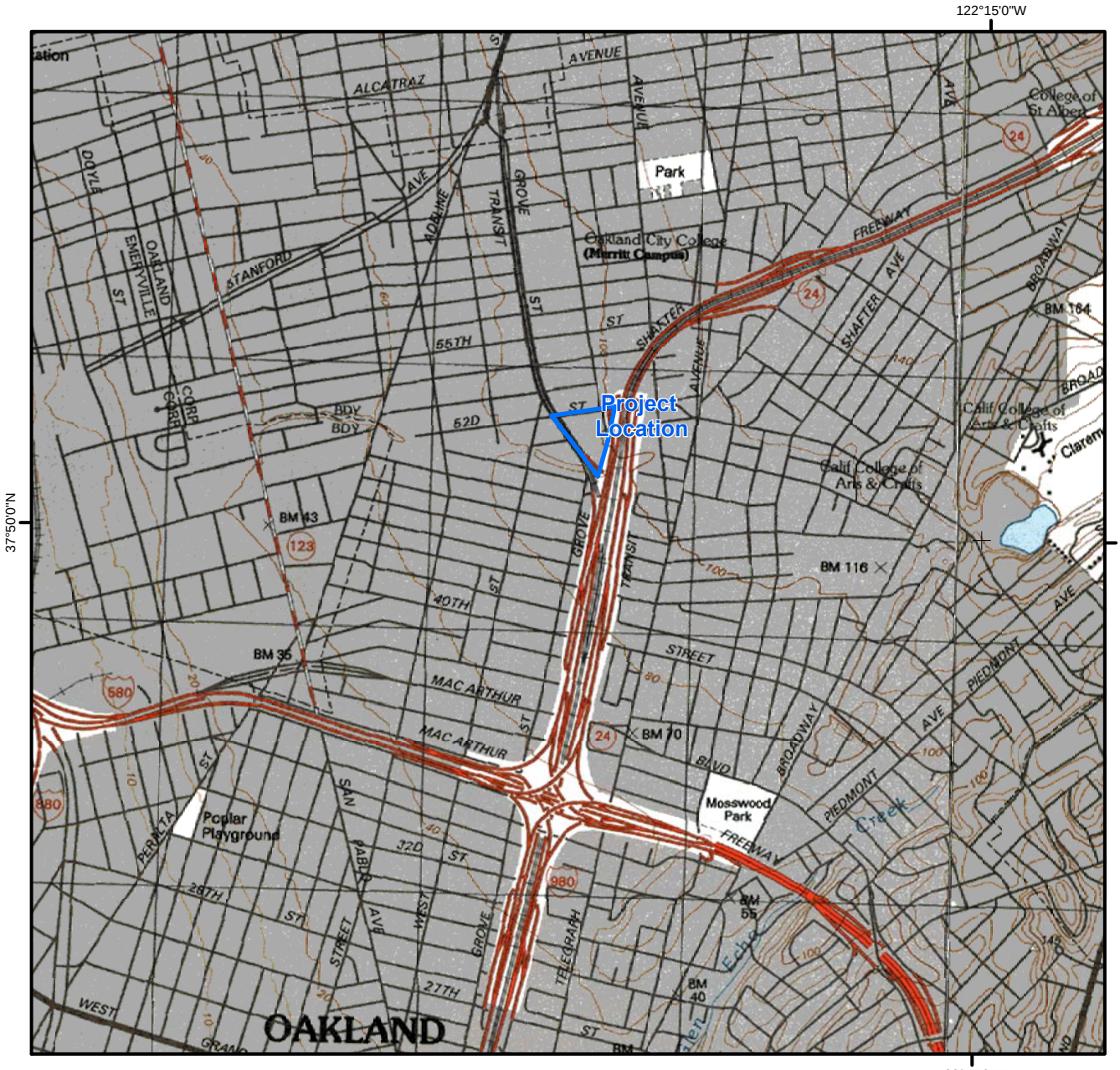

Source: USGS, Oakland West Quadrangle, California, 7.5-Minute Series (Topographic), 1993, Scale 1:24,000

Site Location:
Latitude: 37.8380°
Longitude: -122.2665°

N

1:24,000

0 1,000 2,000 4,000 Feet

TOPOGRAPHIC SITE MAP
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 2

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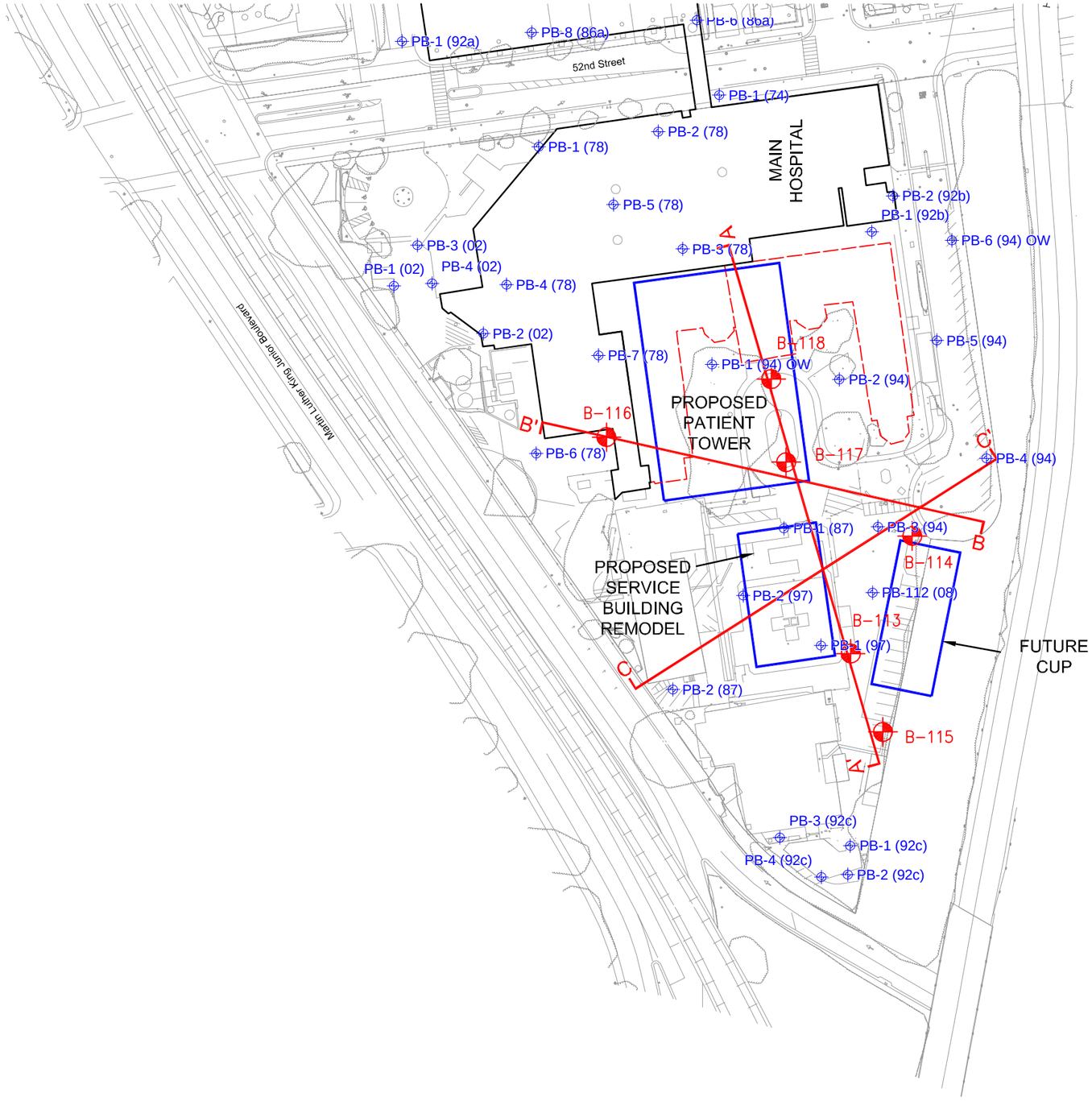

LEGEND

B–118 APPROXIMATE LOCATION OF BORING (2008)

PB-6 (78) APPROXIMATE LOCATION OF PREVIOUS BORING

APPROXIMATE LOCATION OF PROPOSED BUILDING

DEMOLISHED STRUCTURE

APPROXIMATE LOCATION OF CROSS SECTION

NOTES:

(78) Kaldveer "Addition to Children's Hopsital" Project No. K480-J

(87) Kaldveer "Nursing Tower Investigation" Project No. K339-7

(92a) Kaldveer "Ambulatory Services Center" Project No. K938-5-112

(92b) Kaldveer Harza "Proposed Cath Laboratory" Project No. K938-7-1313

(92c) Kaldveer "Chori Laboratory Addition" Project No. K938-6-1125

(94) Harza Kaldveer "Patient Services Pavilion - Phase I" Project No. K938-8-1421

(97) Harza "Helistop Facility" Project No. L998-G

(02) Fugro "Western Expansion" Project No. 21110-GI

(08) Fugro West, Inc. 2008

NORTH

0 100 200

FEET

SITE PLAN
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

G:\jobdocs\1595\1595.002 CHRCO\July 08 Hospital Replacement Project\Drawings\B1595_002_02.dwg 12-17-08 12:15:51 PM vtong

Children's Hospital and Research Center Oakland
Project No. 1595.002

FUGRO

122°18'0"W 122°17'0"W 122°16'0"W 122°15'0"W 122°14'0"W

37°51'0"N
37°50'0"N
37°49'0"N

Project Location

Derby Creek
Temescal Creek
Glen Echo Creek
Glen Echo Creek Northern Branch
Glen Echo Creek (Cemetery Creek)
Pleasant Valley Creek
Bushy Dell Creek
Wildwood Creek
Indian Creek
Broadway Branch
Rockridge Branch

EMERYVILLE
OAKLAND
SAN LEANDRO
BERKELEY
OAKLAND ARMY BASE
Oakland City College
Technical High
Piedmont

Legend

- Creeks
- Former creeks, buried or drained - dashed where channel was ephemeral
- Underground culverts & storm drains
- Engineered channels
- Flood control channels >100 feet wide
- Water spreads over the ground
- Water seeps out of the ground
- Present watersheds
- Artificial bodies of water
- Bay or slough
- Tidal marsh, circa 1850
  - Now water
  - Now filled or drained land
- Tidal marsh formed after 1850
- Fresh water marsh, modern red=artificial, blue=natural
- Willow groves, circa 1850

N

1:24,000

0 1,000 2,000 4,000 Feet

CREEK LOCATION MAP
New Hospital Replacement Project
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 4

Source: Creek and Watershed Map of Oakland and Berkeley; Sowers; 2000.

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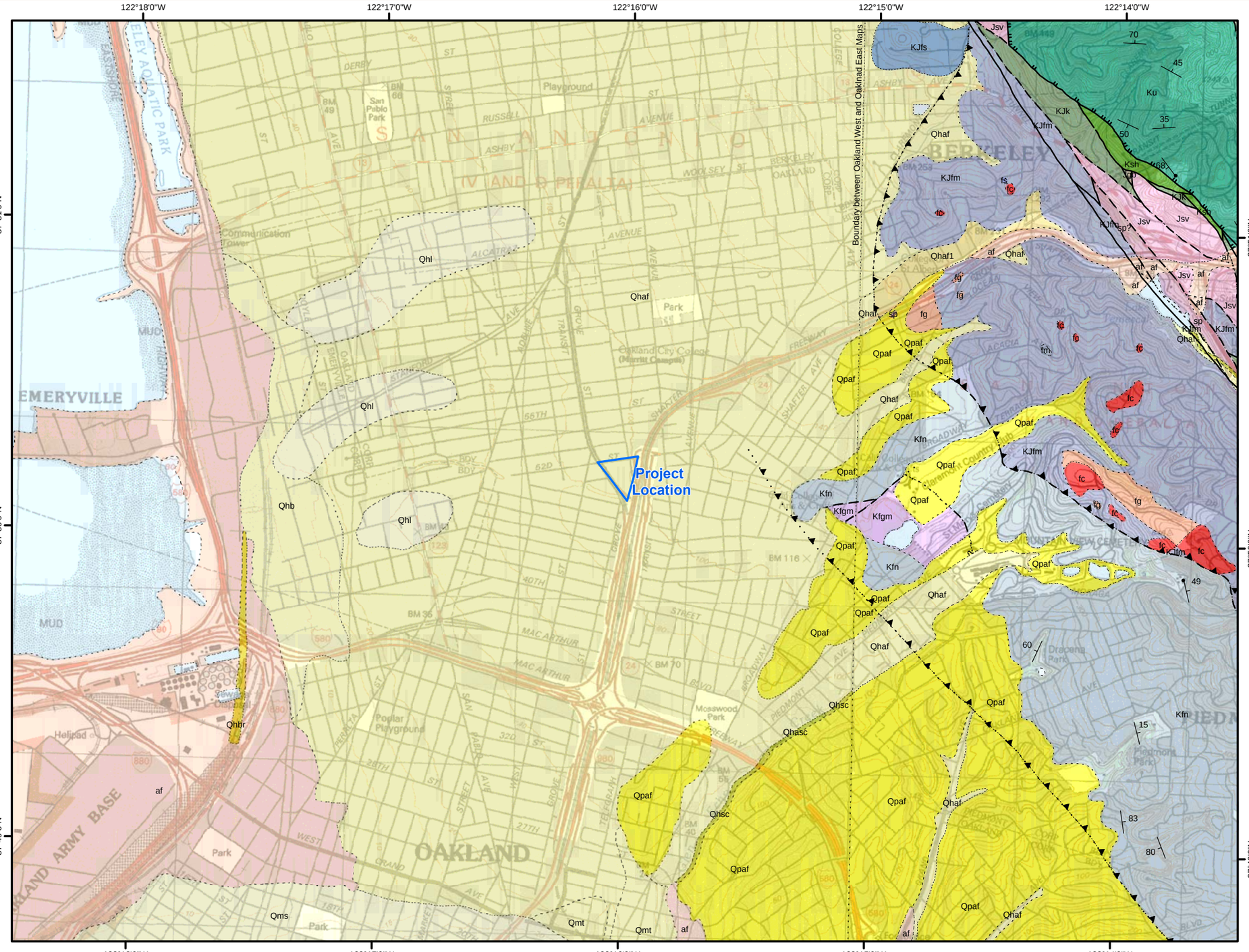

Legend

Contact

Faults (dashed where approx. located, small dashes where inferred, dotted where concealed, queried where location is uncertain)

Fault

Reverse or Thrust Fault

Normal Fault

70 — Strike and Dip of Bedding

49 — Strike and Dip of Bedding, top indicator observed

Geology

water

Surficial Deposits

af	Artificial Fill (Historic)	Qhl	Natural Levee Deposits (Holocene)
Qhasc	Artificial Stream Channels (Historic)	Qhsc	Stream Channel Deposits (Holocene)
Qhaf1	Younger Alluvial Fan Deposits (Holocene)	Qms	Merrit Sand (Holocene and Pleistocene)
Qhaf	Alluvial Fan and Fluvial Deposits (Holocene)	Qmt	Marine Terrace Deposits (Pleistocene)
Qhb	Basin Deposits (Holocene)	Qpaf	Alluvial Fan and Fluvial Deposits (Pleistocene)
Qhbr	Beach Ridge Deposits (Holocene)		

Great Valley Complex

Ku	Undivided Great Valley Complex Rocks (Cretaceous)
Ksh	Siltstone and Shale
KJk	Knoxville Formation (Early Cretaceous and Late Jurassic)
Jsv	Keratophyre and Quartz Keratophyre (Late Jurassic)

Coast Range Ophiolite

Jgb	Gabbro

Franciscan Complex

Kfn	Sandstone of the Novato Quarry Terrane of Blake and Others (1984) (Late Cretaceous)	fc	Serpentinite Blocks
Kfgm	Fine-grained Quartz Diorite (Late Cretaceous (?))	sp	Chert Blocks
KJfs	Franciscan Complex Sandstone, Undivided (Late Cretaceous to Late Jurassic)	fg	Greenstone Blocks
KJfm	Franciscan Complex, Melange (Cretaceous Late Jussaric)	fm	Schist Blocks
fs	Graywacke and Meta-Graywacke Blocks		

N

1:24,000

0 1,000 2,000 4,000 Feet

Source: USGS Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California; Graymer; 2000.

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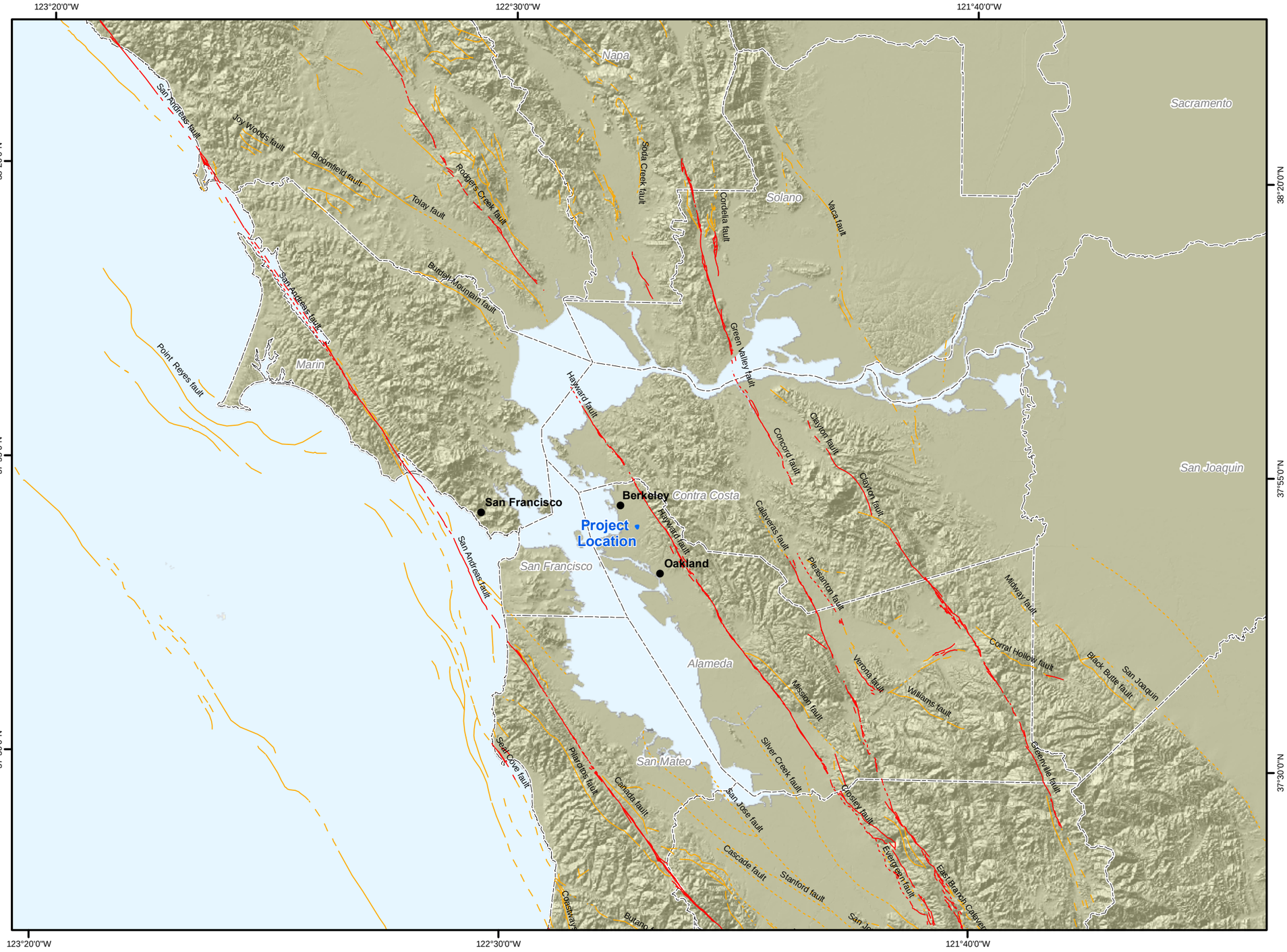
GEOLOGIC MAP

New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 5



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Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Sources:

a) Bryant, 2005

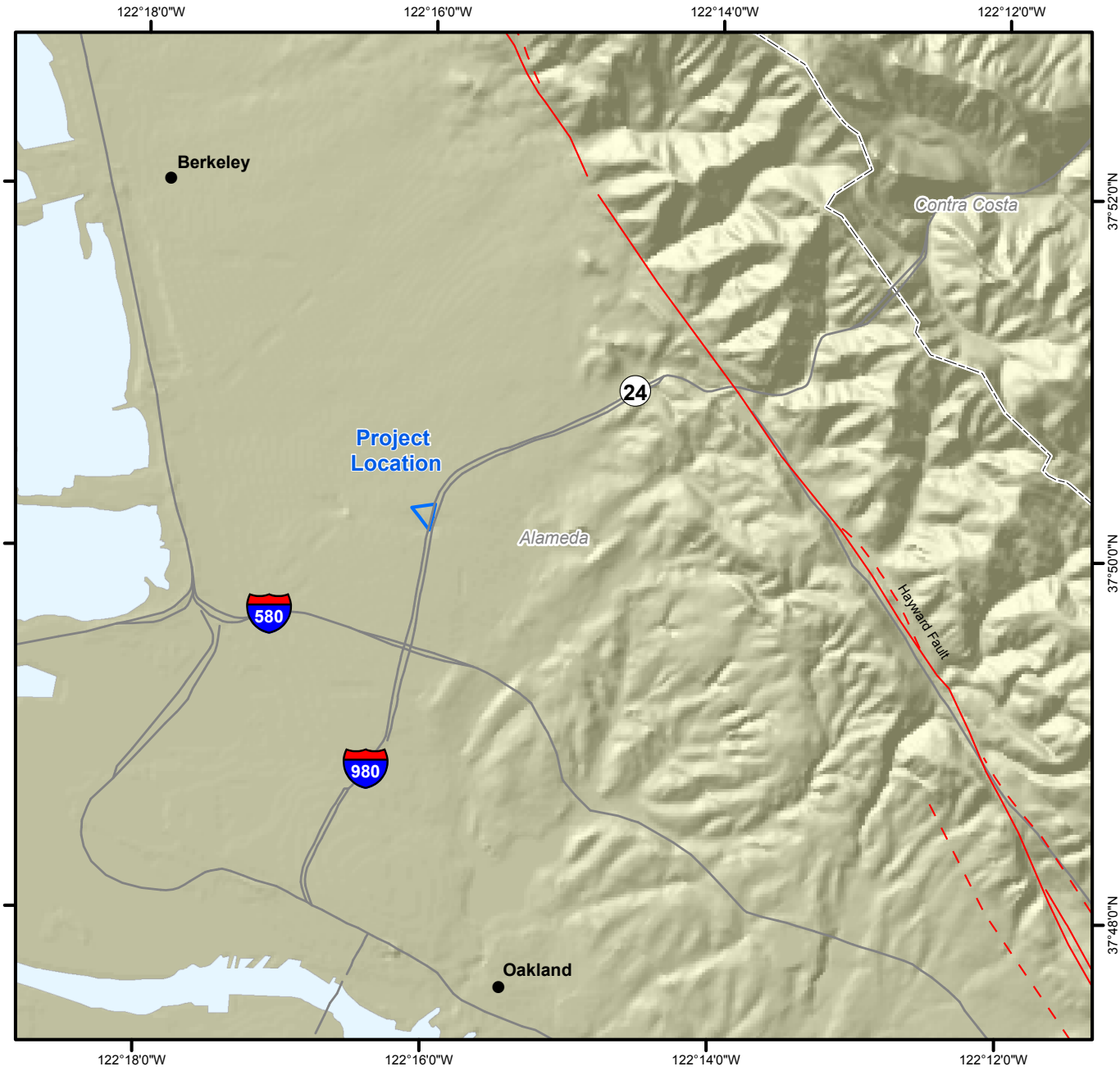
b) Jennings, 1994

N

1:633,600

0 5 10 20 Miles

REGIONAL FAULT MAP
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California
PLATE 6a



Source: Bryant, 2005; Jennings, 1994; Fisher et al., 2005

Legend

- Major Roads
- County Limits

Faults

- Active Fault, dashed where inferred
- Potentially Active Fault, dashed where inferred

N

1:63,360

0 0.5 1 2 Miles

VICINITY FAULT MAP

New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 6b

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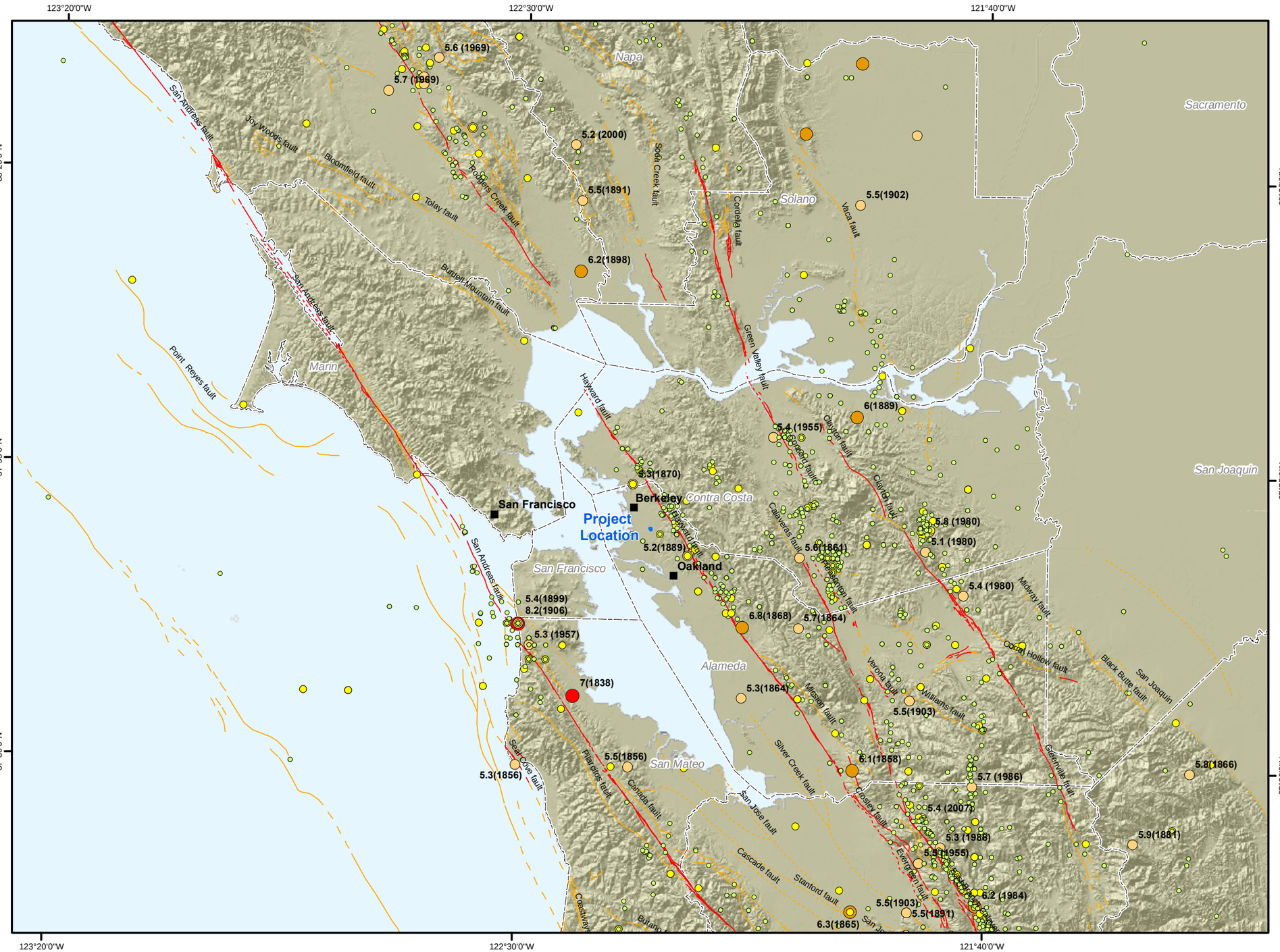

Legend

County Limits

Faults

Active Fault, dashed where inferred

Potentially Active Fault, dashed where inferred

Historical Earthquake Magnitude (1836 - March 2008)
Magnitudes equal to and greater than 5 are labeled.

- 3.0 - 3.9
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7+

Sources:

1) Earthquake Data:
   a) Earthquake epicenters from ANSS Composite Catalog Search, 1898 to 2008, <www.ncedc.org/anss/> (downloaded March 2008)

   b) USGS Earthquake Hazards Program Search, 1735 to 1974, <http://neic.usgs.gov/neis/epic/epic_circ.html> (downloaded March 2008)

2) Faults:
   a) Bryant, 2005

   b) Jennings, 1994

N

1:633,600

0 5 10 20 Miles

FAULTING AND HISTORICAL SEISMICITY
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 7

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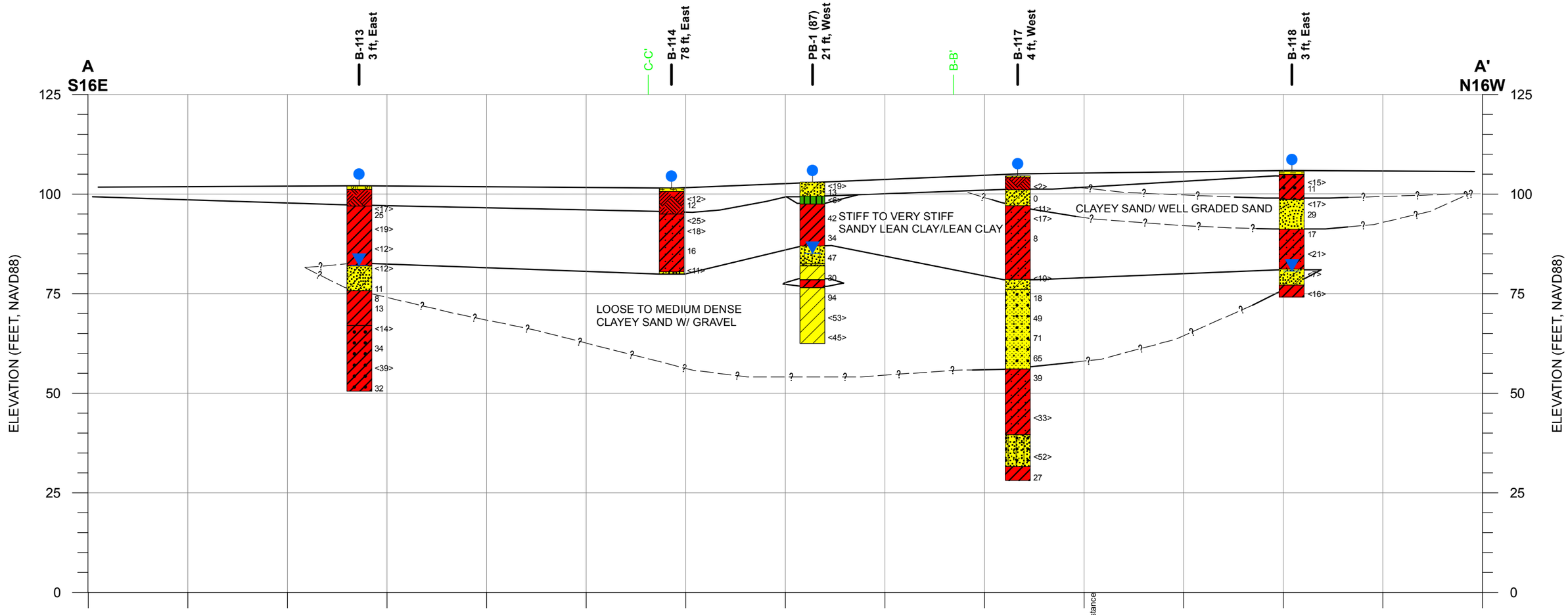
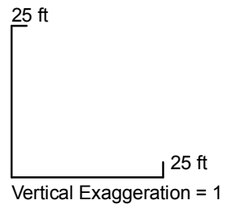


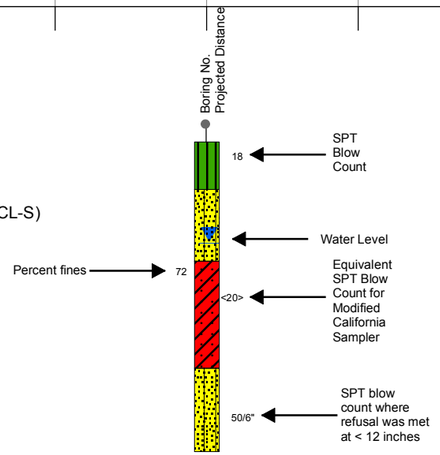
Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.



SOIL TYPES

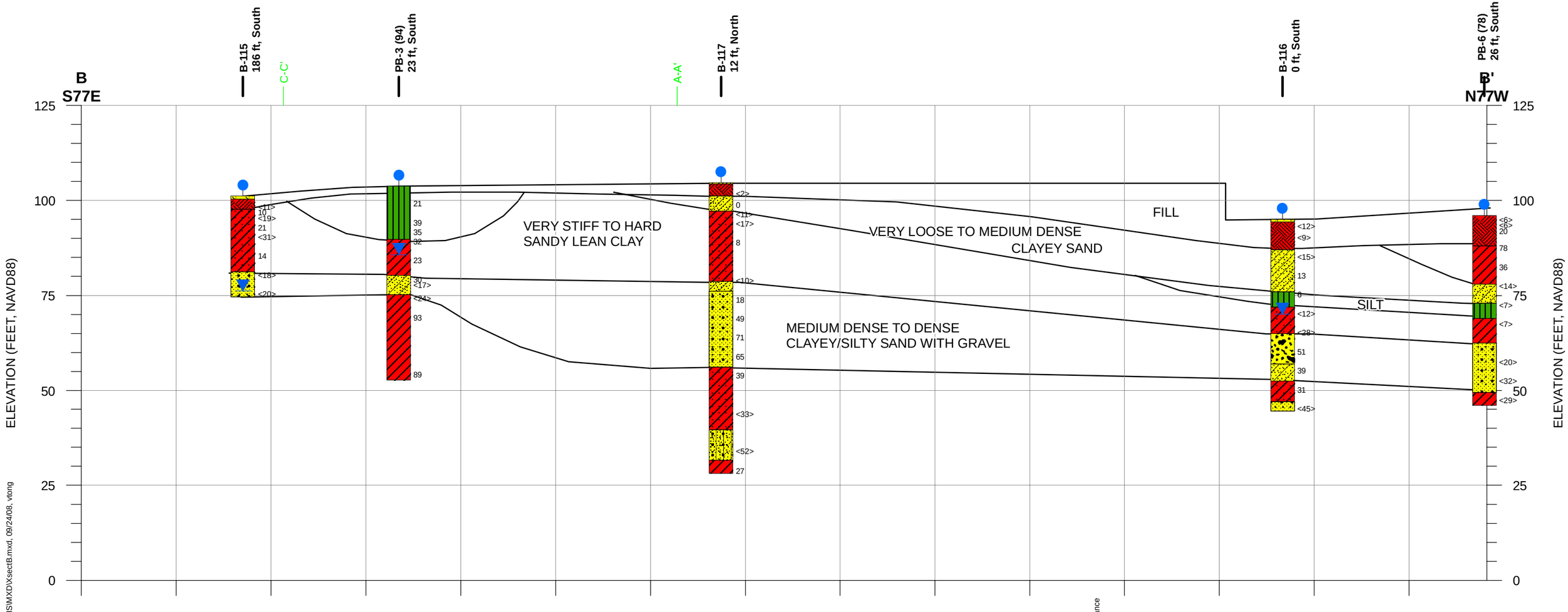
- Fill
- GRAVEL (GP or GW)
- SAND (SP or SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL-S)
- SILT (ML)



SUBSURFACE CROSS SECTION A-A'
New Hospital Replacement Project
New Patient Tower
Children's Hospital Research Center
Oakland,California

PLATE 8a

G:\jobdocs\1595\1595.002 CHRCO\July 08 Hospital Replacement Project\GIS\MXD\XsectA.mxd, 09/24/08, vtong



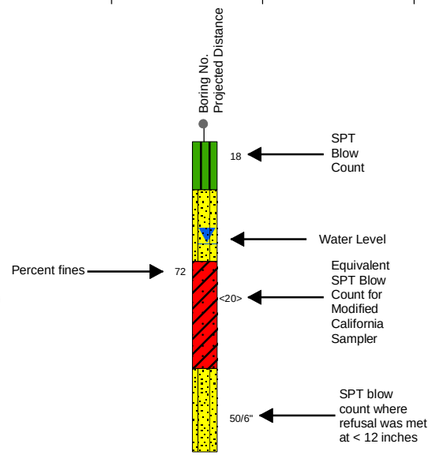
Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.

25 ft
25 ft
Vertical Exaggeration = 1

SOIL TYPES

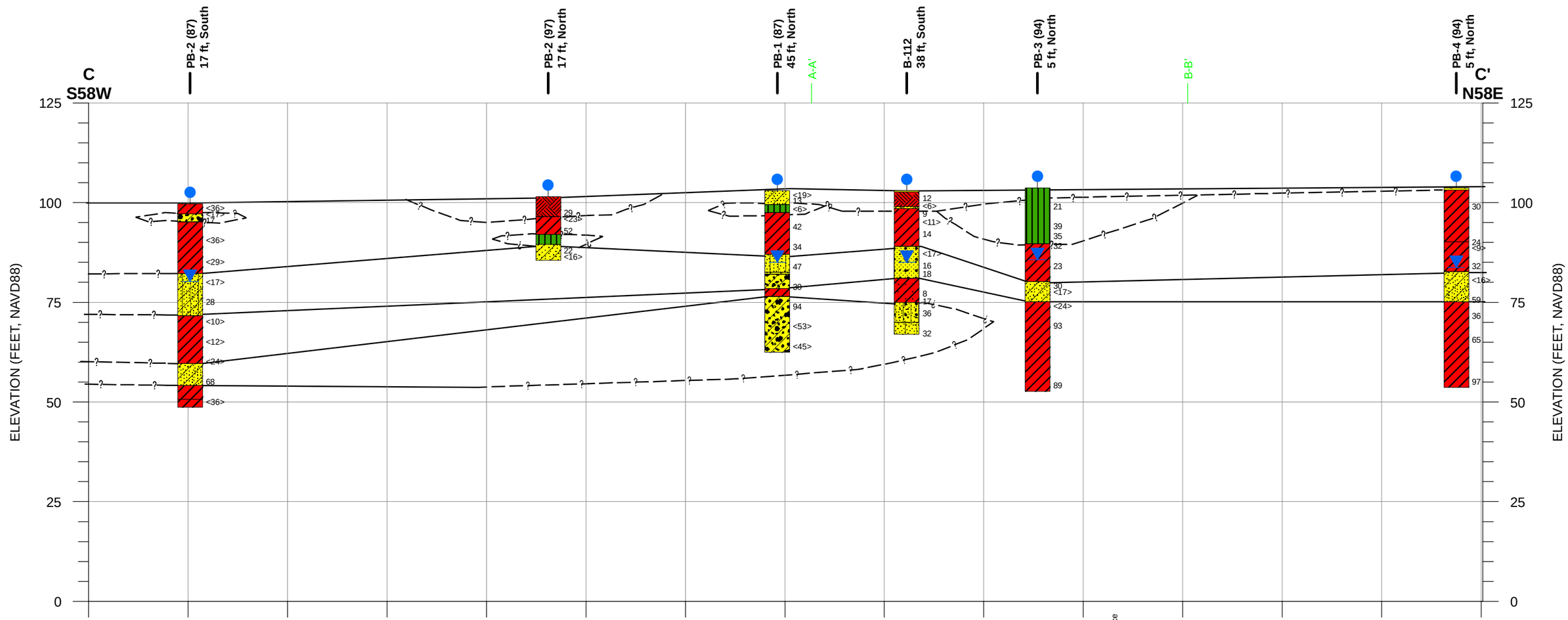
- Fill
- GRAVEL (GP or GW)
- SAND (SP or SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL-S)
- SILT (ML)



SUBSURFACE CROSS SECTION B-B'
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland,California

PLATE 8b

G:\jobdocs\1595\1595.002 CHRCO\July 08 Hospital Replacement Project\GIS\MXD\SectB.mxd, 09/24/08, vtong



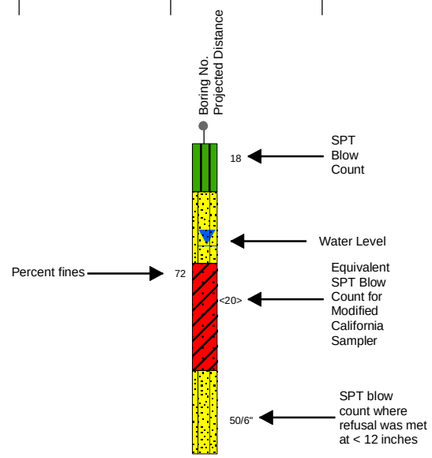
Notes:

1. Refer to Plate 3 for locations of cross section.
2. Topographic elevation, feet, created from contours from CAD drawing provided by Chaudhary & Associates, Inc, 2008.
3. Exploration logs were projected onto the section line. Therefore topographic and stratigraphic contacts of exploration shown on section line above may not match topographic surface or stratigraphic contacts along section line.
4. Subsurface conditions shown represent a limited number of borings. Actual conditions at the site may differ from those shown here.

25 ft
25 ft
Vertical Exaggeration = 1

SOIL TYPES

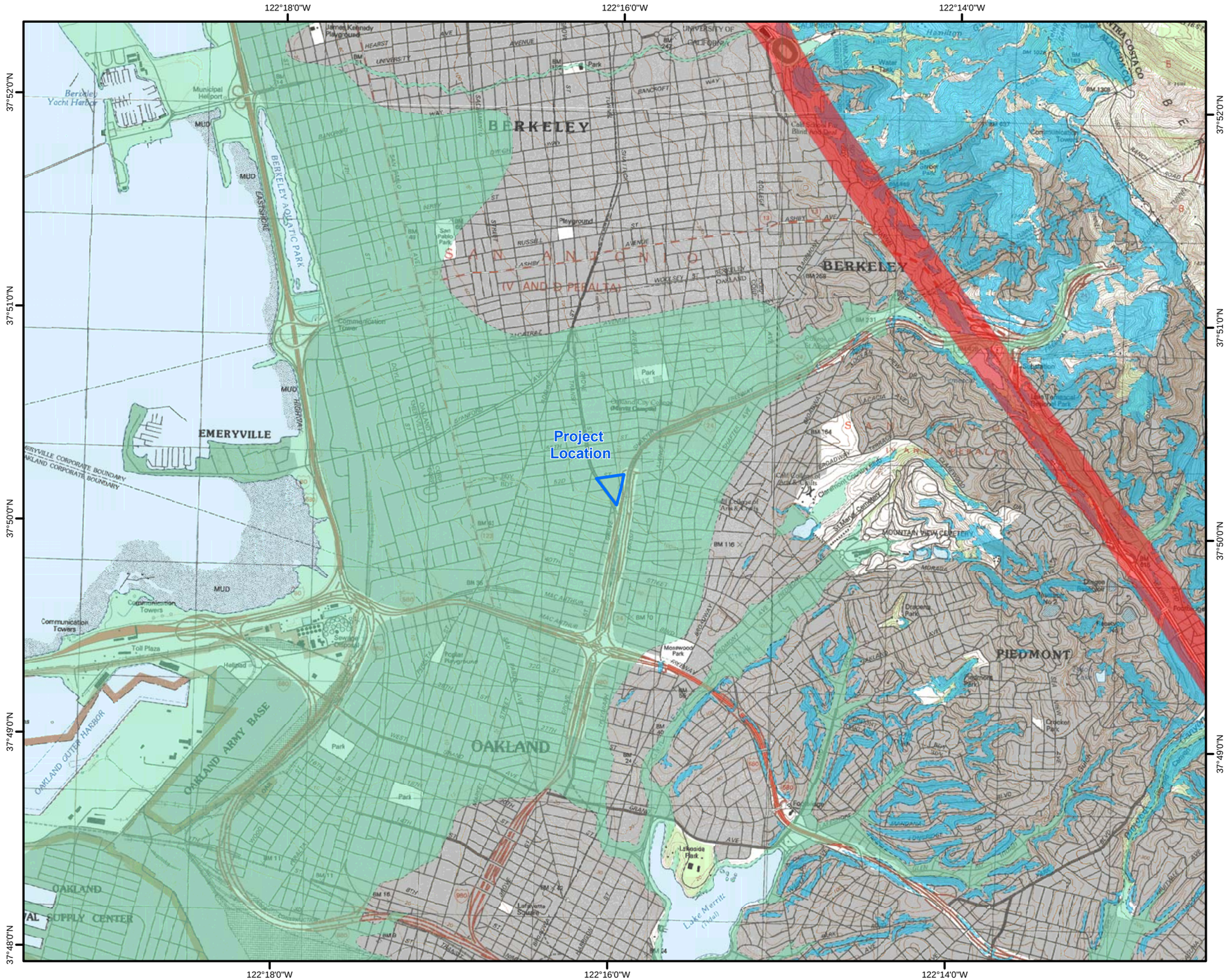
- Fill
- GRAVEL (GP or GW)
- SAND (SP or SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL-S)
- SILT (ML)



SUBSURFACE CROSS SECTION C-C'
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland,California

PLATE 8c

G:\jobdocs\1595\1595.002 CHRCO\July 08 Hospital Replacement Project\GIS\MXD\XsectC.mxd, 08/25/08, vtong



Legend

Liquefaction Hazard Zone
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacements.

Earthquake-Induced Landslide Hazard Zone
Areas where previous occurrence of landslide movement, or local topographic, and geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements.

Alquist-Priolo Earthquake Fault Zone

Sources:
1) Alquist-Priolo (AP) Special Studies Zone Boundaries: California Geologic Survey (2002)
2) Liquefaction and Landslide Special Studies Zones: California Geologic Survey <http://www.conservation.ca.gov/cgs/shzp/>

N

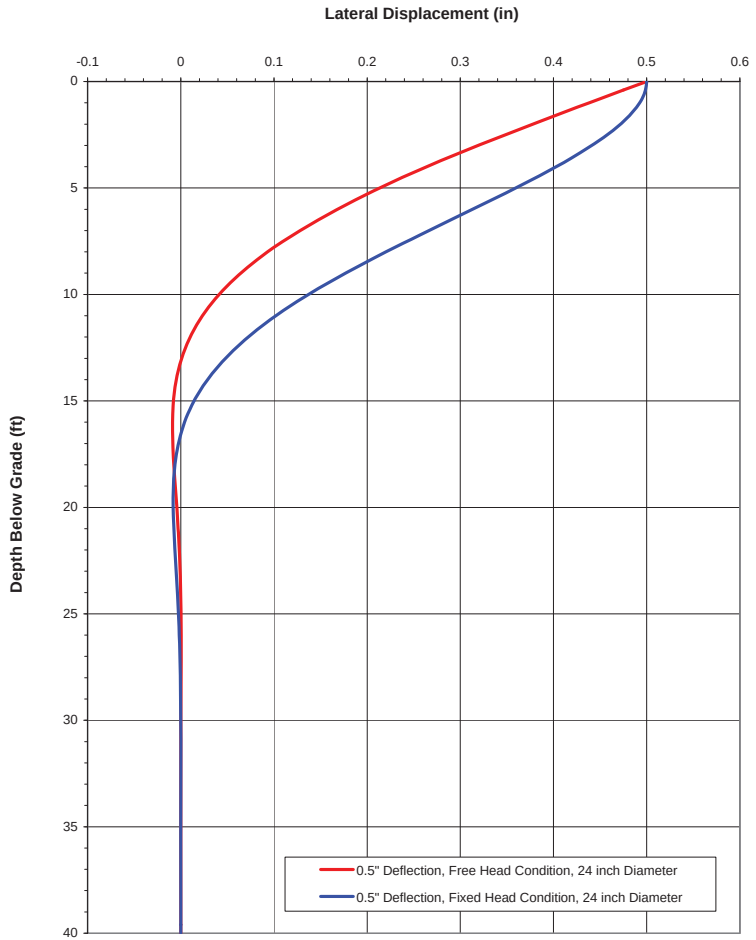
1:36,000

0 1,500 3,000 6,000 Feet

CGS SEISMIC HAZARD ZONE MAP
WITH ALQUIST-PRIOLO EARTHQUAKE ZONES
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 9

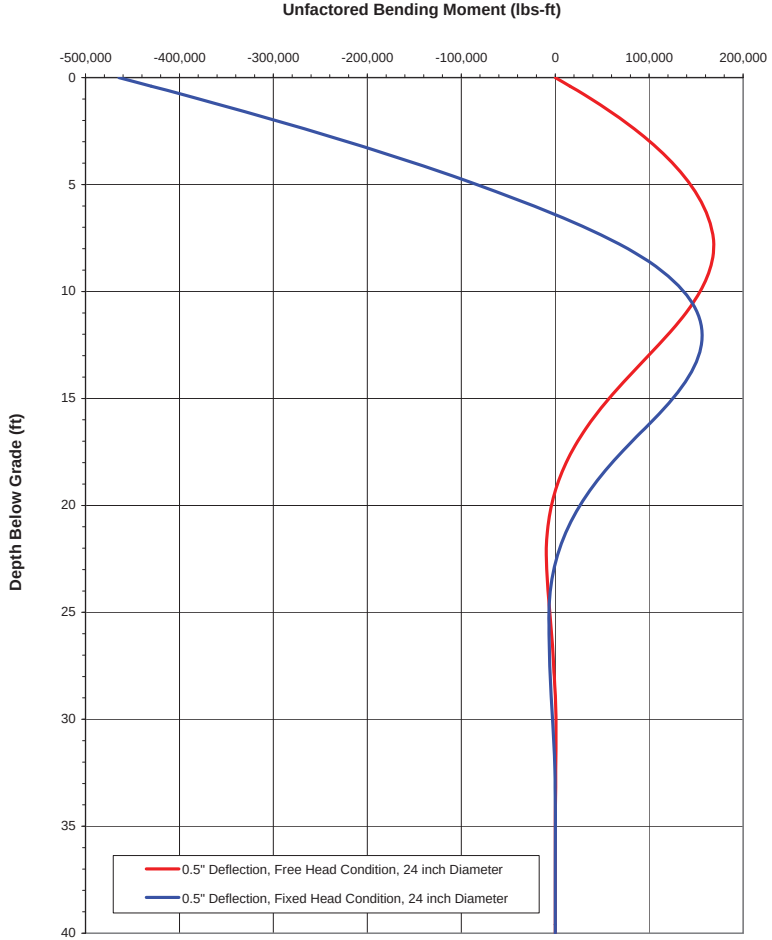
N:\Projects\1595_CHRCO\1595-002_New_Hospital_Replacement_Project\Outputs\March_2008\mxd\Plate9-Seismic_Hazard.mxd, 04/22/08, ksheil



LATERAL PIER ANALYSIS
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

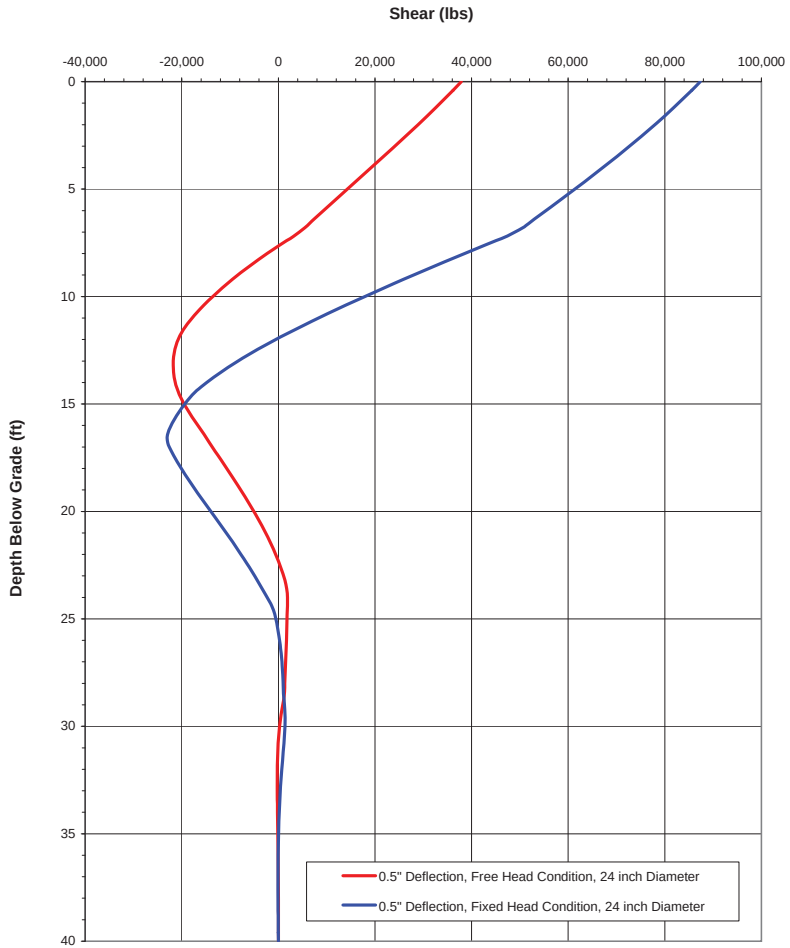
PLATE 10a

Children's Hospital and Research Center Oakland
Project No. 1595.002



LATERAL PIER ANALYSIS
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

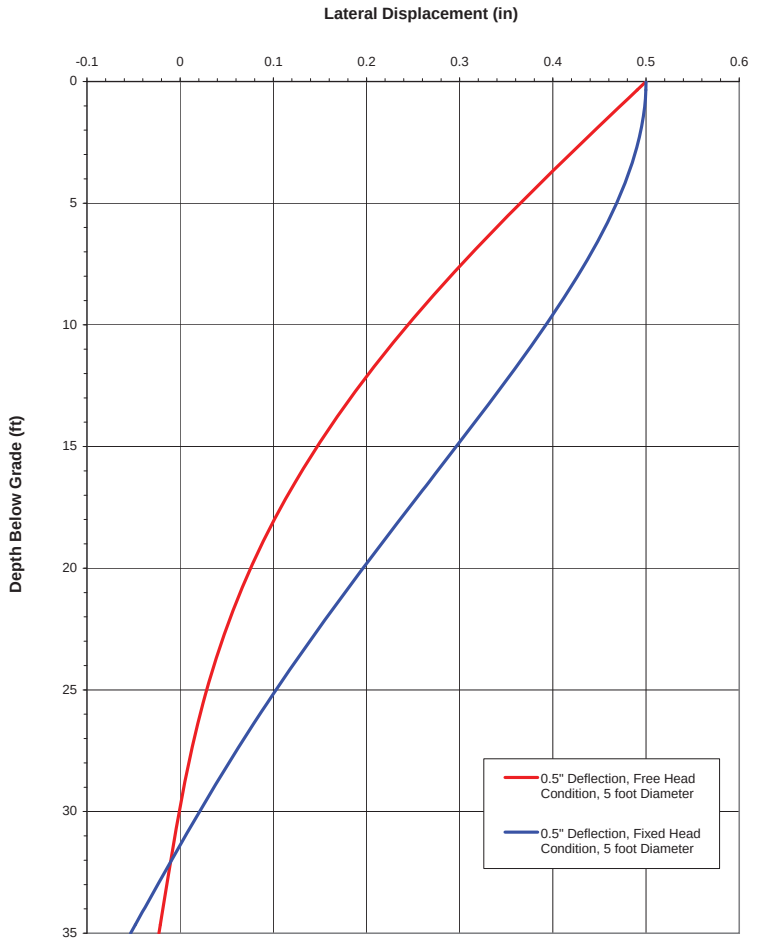
PLATE 10b



LATERAL PIER ANALYSIS
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

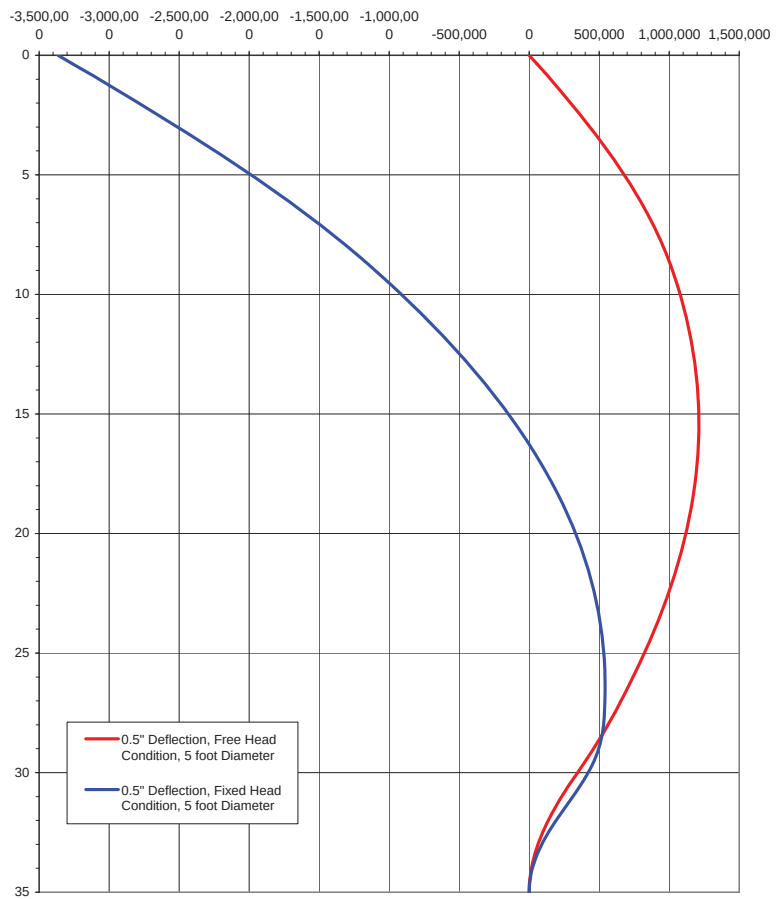
PLATE10c

Children's Hospital and Research Center Oakland
Project No. 1595.002



LATERAL PIER ANALYSIS
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

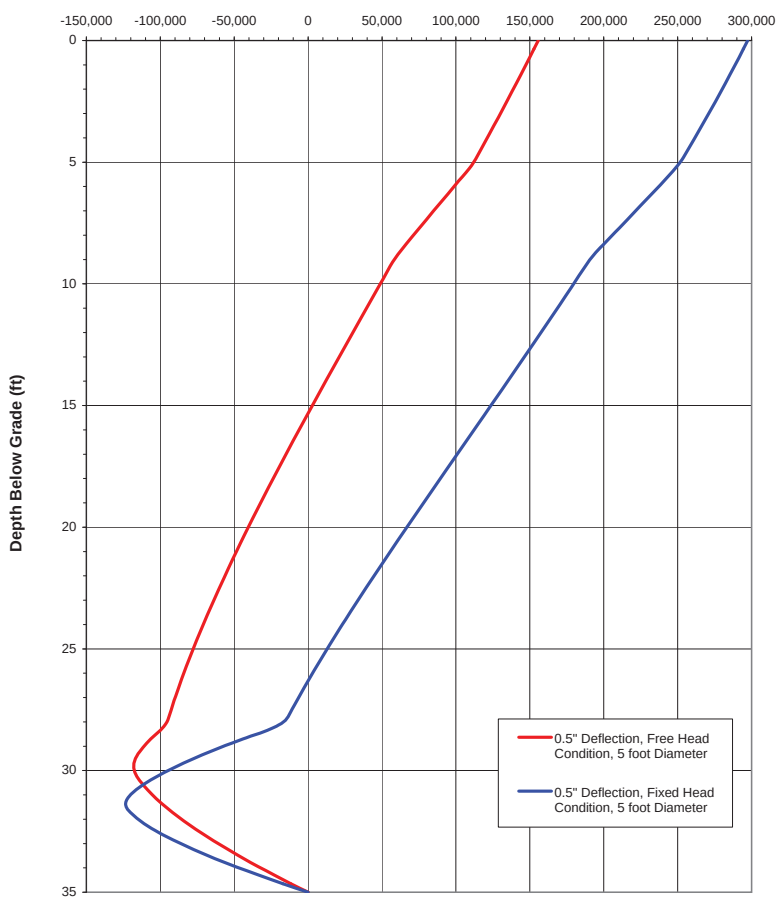
PLATE 11a



LATERAL PIER ANALYSIS
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 11b

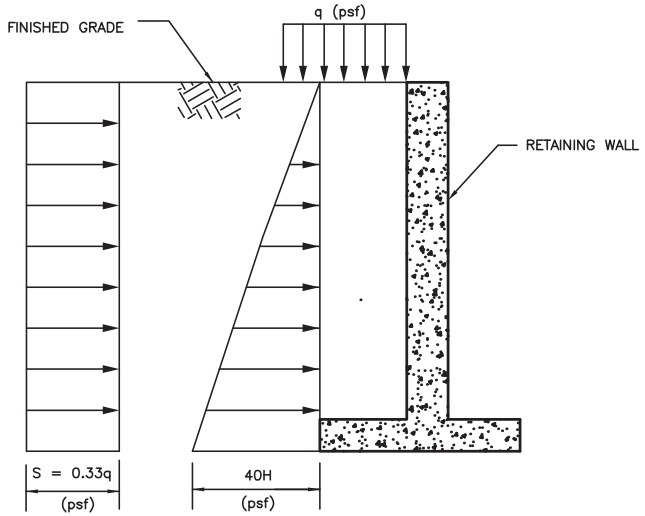
Children's Hospital and Research Center Oakland
Project No. 1595.002



LATERAL PIER ANALYSIS
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE11c

G:\jobdocs\1595\1595.002 CHRCO\July 08 Hospital Replacement Project\Drawings\Plate 11 A Ret Wall Pressures A1595_002_05.dwg 10-01-08 01:44:11 PM oakcad



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.33q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

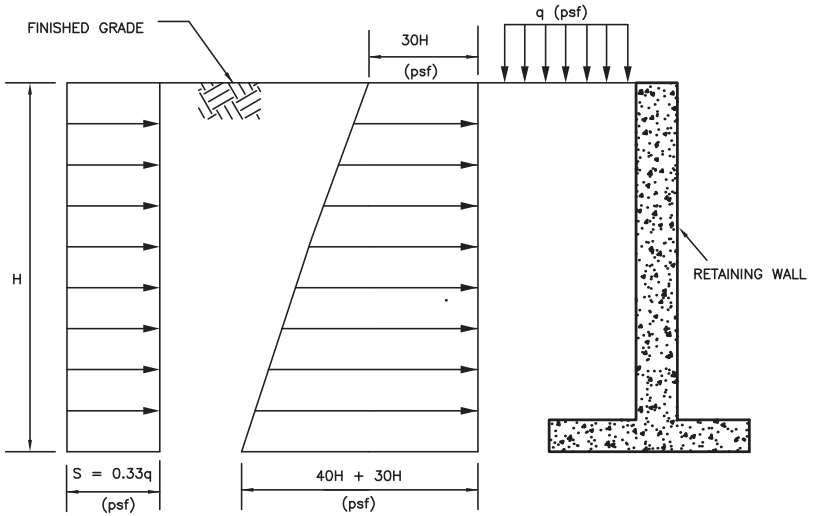
NOTES:

1) UNRESTRAINED WALLS ARE FREE TO MOVE/ROTATE.
2) WALLS ASSUMED TO BE FULLY BACK DRAINED
3) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL
4) ASSUMES LEVEL BACKFILL.

STATIC LATERAL PRESSURES ON UNRESTRAINED WALLS
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 12a

G:\jobdocs\1595\1595.002 CHRCO\May 08 Hospital Expansion Project\Drawings\A1595_002_02.dwg 10-01-08 01:49:26 PM oakcad



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.33q THE SURCHARGE LOAD (psf).

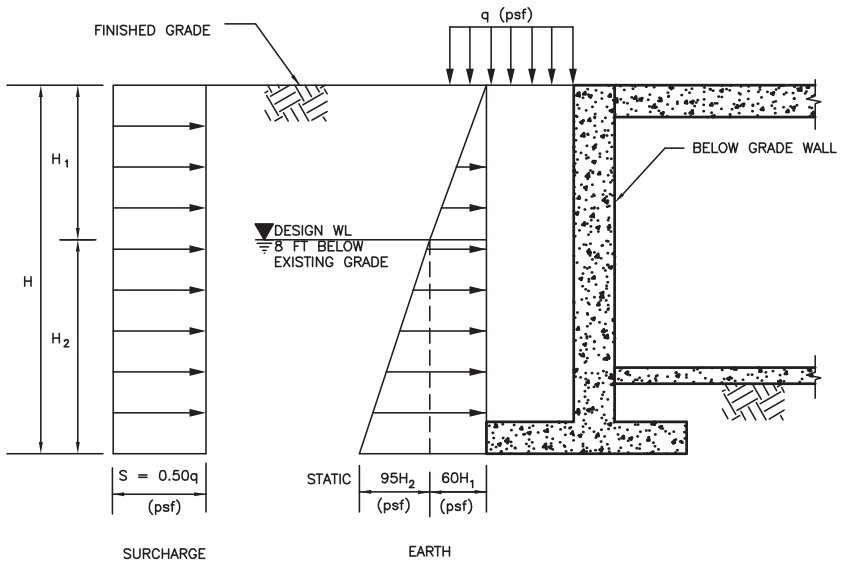
q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

1) UNRESTRAINED WALLS ARE FREE TO MOVE/ROTATE.
2) WALLS ASSUMED TO BE FULLY BACK DRAINED
3) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL
4) ASSUMES LEVEL BACKFILL.

SEISMIC LATERAL PRESSURES ON UNRESTRAINED WALLS
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

PLATE 12b



LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.50q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

1) RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE.
2) DESIGN GROUND WATER LEVEL IS 8 FEET BELOW EXISITNG GRADE.
3) USE $60H_1$ ABOVE AND $95H_2$ BELOW THE DESIGN WATER LEVEL FOR BOTH STATIC AND SEISMIC LOADING CONDITIONS.
4) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL.
5) ASSUMES LEVEL BACKFILL.

STATIC LATERAL PRESSURES ON RESTRAINED WALLS
New Hospital Replacement Project-
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 13a

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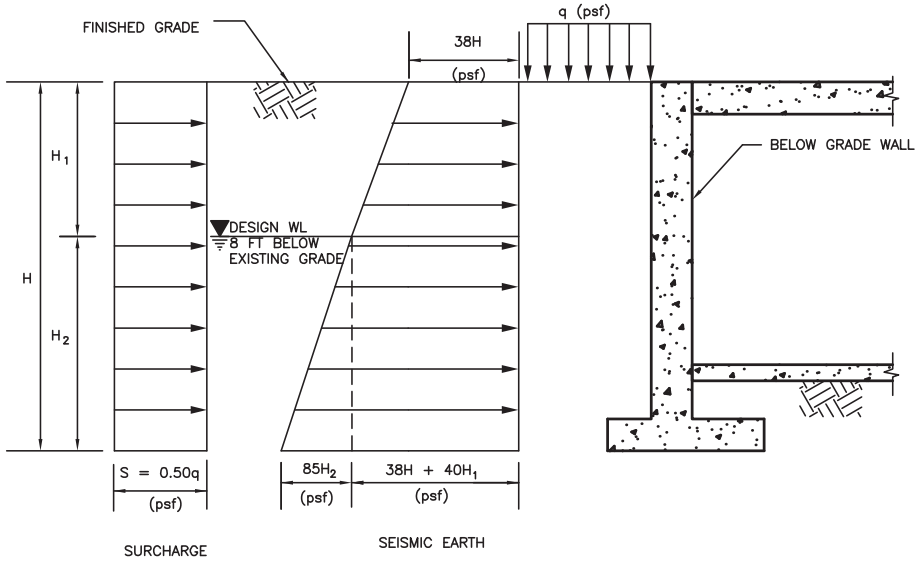

LEGEND:

H = HEIGHT OF BELOW GRADE WALL ABOVE BOTTOM OF FOOTING (FEET).

S = 0.50q THE SURCHARGE LOAD (psf).

q = UNIFORM SURCHARGE PRESSURE ADJACENT TO WALL.

NOTES:

1) RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE.
2) DESIGN GROUND WATER LEVEL IS 8 FEET BELOW EXISITNG GRADE.
3) USE $40H_1$ ABOVE AND $85H_2$ BELOW THE DESIGN WATER LEVEL FOR BOTH STATIC AND SEISMIC LOADING CONDITIONS.
4) BASED ON COMPACTED LEAN CLAY (CL) BACKFILL.
5) ASSUMES LEVEL BACKFILL.

SEISMIC LATERAL PRESSURES ON RESTRAINED WALLS
New Hospital Replacement Project -
New Patient Tower
Children's Hospital and Research Center
Oakland, California

PLATE 13b

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APPENDIX A
FIELD EXPLORATIONS

APPENDIX A
FIELD EXPLORATIONS

The field exploration consisted of a surface reconnaissance and a subsurface exploration program. Our subsurface investigation included drilling six conventional geotechnical borings. The following sections contain a discussion regarding the equipment and procedures to perform the exploration. The logs of the borings, as well as a key for the classification of the soil (Plate A-1), are included as part of this appendix.

V&W Drilling, of Lodi, California, drilled the four borings for this project between August 4 and 8, 2008. The borings were drilled using a truck-mounted CME-75 drill rig equipped with 8-inch diameter hollow-stem augers, 5-inch diameter solid-flight augers, and 4 3/8-inch diameter mud rotary drilling capabilities. The borings were advanced to a maximum depth of about 26 feet below grade. The approximate locations of the borings are shown on Plate 3 – Site Plan.

Soil samples were obtained using either a standard split spoon sampler (2-inch outer diameter, 1 3/8-inch inner diameter) or Modified California Liner sampler (3-inch outer diameter, 2 3/8-inch inner diameter). The sampler type is indicated on the boring logs as designated on the Terms and Symbols, Plate A-1. Soil samples were taken semi continuously to a depth of about 10 feet and at 5-foot intervals thereafter. The Standard Penetration Test (SPT) was performed as part of the sampling procedure (in accordance with ASTM D-1586). The Standard Penetration Test is a measure of soil density and consistency. The SPT-N value (blow count) is defined as the number of blows required to drive a 2-inch diameter split-barrel sampler 12-inches, after an initial penetration of 6-inches, using a 140-pound hammer falling freely for 30-inches. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the sampler the last 12 inches. We note that samples obtained using the Modified California Liner sampler are not considered standard N-values because of the larger diameter of the samplers.

A Fugro engineer observed the drilling operation and logged the soil and rock encountered. Our inspecting engineer recorded SPT N-values. Recovered soil samples were visually examined and classified in accordance with the Unified Soil Classification System (USCS) and ASTM D-2488 (visual method). Our engineer estimated soil undrained shear strength in the field with a Pocket Penetrometer. Soil samples were transported to our office and laboratory for further examination, confirmation of classification and laboratory testing.

The surface elevations on the boring logs were estimated by interpreting the topographic contours on the project topographic map prepared by Chaudhary & Associates, Inc.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated. We do not warrant they are representative of subsurface conditions at other locations and times.

MAJOR DIVISIONS			GROUP NAMES	
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW	Well-Graded Gravel
			GP	Poorly Graded Gravel
		Gravels with more than 12% fines	GM	Silty Gravel
			GC	Clayey Gravel
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	Clean sand less than 5% fines	SW	Well-Graded Sand
			SP	Poorly Graded Sand
		Sands with more than 12% fines	SM	Silty Sand
			SC	Clayey Sand
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML	Silt
			CL	Lean Clay
			OL	Organic Silt
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH	Elastic Silt
			CH	Fat Clay
			OH	Organic Clay
HIGHLY ORGANIC SOILS			PT	Peat or Highly Organic Soils
			FILL	Debris or Mixed Fill
			AC	Asphalt Concrete Pavement with Aggregate Base

GENERAL NOTES

Classification of Soils per ASTM D2487 or D2488

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

Sloped line in break column indicates transitional boundary

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

SAMPLER DRIVING RESISTANCE

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

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
50/7"	50 blows drove sampler 7" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

STRENGTH TEST METHOD

U = Unconfined Compression
Q = Unconsolidated Undrained Triaxial
T = Torvane
P = Pocket Penetrometer
M = Miniature Vane
F = Field Vane

OTHER TESTS

k = Permeability
Consol = Consolidation
Gs = Specific Gravity
MA = Particle Size Analysis
EI = Expansion Index
OVM = Organic Vapor Measurement

WATER LEVEL SYMBOLS

- Initial or perched water level
- Final ground water level
- Seepages encountered

SAMPLER TYPE

1 SPT, 2 MC, 3 CA, 4 SH, 5 BB, 6 HA, 7 LS, 8 PS, 9 VS, 10 NR, 11 RC

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

1. SPT Sampler, driven 1 3/8" ID, 2" OD
2. MOD CA Liner Sampler 2 3/8" ID, 3" OD
3. CA Liner Sampler 1 7/8" ID, 2.5" OD
4. Thin-walled Tube, pushed 2 7/8" ID, 3" OD
5. Bulk Bag Sample (from cuttings)
6. Hand Auger Sample
7. Lexan Sample
8. Pitcher Sample
9. Vibracore Sample
10. No Sample Recovered
11. Rock Core

SOIL STRUCTURE

Fissured: Containing shrinkage or relief cracks, often filled with fine sand or silt, usually more or less vertical.

Pocket: Inclusion of material of different texture that is smaller than the diameter of the sample.

Parting: Inclusion less than 1/8 inch thick extending through the sample.

Seam: Inclusion 1/8 inch to 3 inches thick extending through the sample.

Layer: Inclusion greater than 3 inches thick extending through the sample.

Laminated: Soil sample composed of alternating partings or seams of different soil types.

Interlayered: Soil sample composed of alternating layers of different soil type.

Intermixed: Soil sample composed of pockets of different soil type, and layered or laminated structure is not evident.

CONSISTENCY

Clays	Blows/Foot SPT	Undrained Shear Strength (ksf)
Very Soft	0 - 2	0 - 0.25
Soft	2 - 4	0.25 - 0.5
Firm	4 - 8	0.5 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

RELATIVE DENSITY

Sands and Gravels	Blows/Foot SPT
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

INCREASING VISUAL MOISTURE CONTENT

Dry → Moist → Wet

Information on each boring log is a compilation of subsurface conditions and soil or rock classifications obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the time and places indicated, and can vary with time, geologic condition, or construction activity.

TERMS AND SYMBOLS USED ON BORING LOGS PLATE A-1

LOCATION: SEE SITE PLAN, PLATE 3
N 6051529.36 E 2131714.14
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 102.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 0		1			4 inches of AC and 6 inches of AB							
					ARTIFICIAL FILL (af) Sandy Lean CLAY (CL): reddish brown, dry to moist, low to medium plasticity - bulk sample taken from auger cuttings		6		22	10		
95 / 5		2		(28)	**ALLUVIUM (Qal)** Lean CLAY with sand (CL): very stiff, light yellowish brown, moist, medium plasticity							
		3		25							3.5 M	
90 / 10		4		(30)	- very stiff, low to medium plasticity, fine sand	107	19				3.1 M	
85 / 15		5		(19)	- stiff, moist, low to medium plasticity						2.8 P	
80 / 20		6		(19)	Clayey Sand (SC): loose to medium dense, yellowish brown, wet, grades into some rectangular gravel up to 1/2"			29				
75 / 25		7		11	- medium dense, fine to medium grained sand, grades to silty clay at the bottom of sampler			40				
		8		8	Lean CLAY (CL): stiff, grayish brown, moist, medium to high plasticity							
70 / 30		9		13								
65 / 35		10		(23)	Sandy Lean CLAY with gravel (CL): very stiff, light brown, moist, low plasticity, fine sand with gravel up to 1/2"							

Continued

BORING DEPTH: 51.5 ft
BACKFILL: Grout
DEPTH TO WATER: 20.0 ft
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-113

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 6051529.36 E 2131714.14
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 102.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
60 / 45	11		34	- medium grained sand							
55 / 50	12		(63)	hard, moist, yellowish brown, fine sand, low to medium plasticity, trace gravel						3.5 M	
50 / 55	13		32	- very stiff, light brown, with medium grained sand							
				End of Boring at 51-1/2 ft.							

BORING DEPTH: 51.5 ft
BACKFILL: Grout
DEPTH TO WATER: 20.0 ft
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-113

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 6051579.8 E 2131810.51
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 101.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
				4 inches of AC and 6 inches of AB							
100 / 5	1		(20)	**ARTIFICIAL FILL (af)** Sandy Lean CLAY (CL): stiff, dark brown, moist, low to medium plasticity	96	20				3.8 P	
	2		12								
95 / 10	3		(40)	**ALLUVIUM (Qal)** Sandy Lean CLAY (CL): very stiff, light yellowish brown, moist, fine sand, low to medium plasticity				41	26	4.5 P	
	4		(29)	- yellowish brown, low to medium grained sand, medium plasticity	113	16				1.9 M 1.8 P	
90 / 15	5		16	- stiff, gray				34	19		
85 / 20	6		(17)	Clayey SAND with silt (SC): loose, yellowish brown, moist, fine grained sand at the bottom of the sampler						0.9 M	
				End of Boring at 21-1/2 ft.							

BORING DEPTH: 21.5 ft
BACKFILL: Grout
DEPTH TO WATER:
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-114

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 6051555.79 E 2131649.93
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 101.1 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 0				4 inches of AC and 6 inches of AB							
	1		(17)	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, dark brown, dry, low to plasticity		9				2.5 P	
95 / 5	2		10	**ALLUVIUM (Qal)** Sandy Lean CLAY (CL): firm, light to dark brown, dry to moist, low plasticity							
	3		(31)	- very stiff, dark brown							
	4		21	- very stiff, light brown	100	23					
90 / 10	5		(49)	- very stiff, light yellowish brown, moist, low to medium plasticity	114	17				4.5 P	
85 / 15	6		14	- stiff, tan and light brown, low to medium plasticity							
80 / 20	7		(29)	Clayey SAND with gravel (SC): medium dense, light brown, moist, fine grained sand, angular gravel up to 1"			35				
							22				
75 / 25	8		(32)	- light grayish brown, moist, fine sand with gravel			27			1.8 M	
				End of Boring at 26-1/2 ft.							
70 / 30											
65 / 35											

BORING DEPTH: 26.5 ft
BACKFILL: Grout
DEPTH TO WATER: 25.0 ft
COMPLETION DATE: August 4, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-115

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

LOCATION: SEE SITE PLAN, PLATE 3
N 6051329.33 E 2131891.48
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 95.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
				Concrete Pavement - 8 inches							
	1		(19)	**ARTIFICIAL FILL (af)** Lean CLAY (CL): stiff, black to dark brown, moist, low plasticity	107	17		43	26	2.8 P 3.5 M	
90 / 5	2		(14)	- stiff, light yellowish brown, low plasticity	111	18				1207.9 U	
	3		(24)	**ALLUVIUM (Qal)** Clayey SAND, Sandy CLAY (CL/SC): medium dense, yellowish brown, moist, coarse grained sand			50				
85 / 10	4		13	- gray and yellowish brown, moist, fine sand			46				
80 / 15	5		6	SILT (ML): firm, grayish brown, moist, very fine sand, low plasticity							
75 / 20	6		(19)	Lean CLAY with sand (CL): stiff, dark grayish brown, moist, low to medium plasticity, fine-to coarse-grained sand	94	29				2.1 M	
70 / 25	7		(45)	Well-graded GRAVEL with clay and sand (GW-GC): medium dense, dark to light brown, moist to wet, coarse grained sand, angular gravel up to 1"			15				
65 / 30	8		51								
60 / 35	9		39								

Continued

BORING DEPTH: 50.5 ft
BACKFILL: Grout
DEPTH TO WATER: 25.0 ft
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-116

Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

LOCATION: SEE SITE PLAN, PLATE 3
N 6051329.33 E 2131891.48
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 95.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
50 / 45	10		31	Clayey SAND (SC): medium dense, tan and yellowish brown, moist, subangular to angular, medium to coarse sand with trace gravel Sandy Lean CLAY (CL): very stiff, yellowish brown, moist to wet, low plasticity, fine-grained sand							
45 / 50	11		(72)	Clayey SAND (SC): dense, yellowish brown, moist to wet, coarse sands and increased gravel content up to 1"							
				End of Boring at 50-1/2 ft.							
40 / 55											
35 / 60											
30 / 65											
25 / 70											
20 / 75											

BORING DEPTH: 50.5 ft
BACKFILL: Grout
DEPTH TO WATER: 25.0 ft
COMPLETION DATE: August 5, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-116
Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

LOCATION: SEE SITE PLAN, PLATE 3
N 6051476.49 E 2131870.96
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 104.6 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
				grass and topsoil approximately 4 inches							
	1		(4)	**ARTIFICIAL FILL (af)** Sandy Lean CLAY (CL): soft, light brown, moist, low plasticity, trace rootlets						1.8 P	
100 / 5	2		0	**ALLUVIUM (Qal)** Clayey SAND (SC): very loose, light brown, moist, medium grained sand		24	48			2.8 P	
	3		(17)	Sandy Lean CLAY (CL): stiff, dark brown, moist, dark brown, moist, low to medium plasiticty	98	26				1.6 M	
95 / 10	4		(28)	- very stiff, light yellowish brown, moist to wet, low to medium plasticity, some sand							
90 / 15	5		8	- firm, wet, trace gravel							
85 / 20				- very stiff, gray and yellowish brown						2.7 M	
80 / 25	7		(16)			25	21				
				Clayey SAND (SC): loose to medium dense, yellowish brown, wet, medium to coarse grained sand							
75 / 30	8		18	Clayey SAND with gravel (SC): medium dense, yellowish brown, moist to wet, subangular gravel up to 1/2-inch in size			22				
70 / 35	9		49	- dense, mottled brown, fine to medium grained sand, trace subangular to angular gravel							
65											

Continued

BORING DEPTH: 76.5 ft
BACKFILL: Grout
DEPTH TO WATER:
COMPLETION DATE: August 4, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Rotary Wash
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-117
Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

Sheet 2 of 2

LOCATION: SEE SITE PLAN, PLATE 3
N 6051476.49 E 2131870.96
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 104.6 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
60 / 45	10		71								
	11		65	- very dense, coarse sand, gravel lens with 1" clasts							
55 / 50	12		39	Sandy Lean CLAY (CL): hard, grayish brown, moist to wet, medium plasticity, fine grained sand							
50 / 55											
45 / 60	13		(53)	- hard, light brown, low to medium plasticity	100	26				3.5 M	
40 / 65				Silty SAND with gravel (SM): very dense, light brown, moist to wet, gravel lens with 1" clasts							
35 / 70	14		(84)								
30 / 75				Lean CLAY with sand (CL): very stiff, light yellowish brown, moist, low to medium plasticity							
	15		27								
25				End of Boring at 76-1/2 ft.							

BORING DEPTH: 76.5 ft
BACKFILL: Grout
DEPTH TO WATER:
COMPLETION DATE: August 4, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Rotary Wash
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-117
Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

Project No. 1595.002

Sheet 1 of 1

LOCATION: SEE SITE PLAN, PLATE 3
N 6051464.29 E 2131939.03
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 105.7 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
				4 inches of AC and 6 inches of AB							
105 / 0	1		(24)	Sandy Lean CLAY (CL): very stiff, dark brown, dry, low to medium plasticity, with gravel lenses		8				4.0 P 3.5 M	
	2		11								
100 / 5	3		(28)	Well-graded SAND (SW): medium dense, light yellowish brown, dry, medium grained sand, fine to coarse gravel up to 1" clasts		6	4				
95 / 10	4		29	- medium dense - angular gravel up to 1 1/2" from cuttings at 12 ft.							
90 / 15	5		17	Sandy Lean CLAY (CL): stiff, dark to light yellowish brown, dry to moist, some sand, low to medium plasticity							
85 / 20	6		(34)	- stiff to very stiff, gravel lenses						2.0 P	
80 / 25	7		(12)	Clayey SAND (SC): loose, yellowish brown, moist to wet, fine grained sand		26	40				
75 / 30	8		(26)	Lean CLAY (CL): very stiff, grayish brown, moist to wet, medium plasticity	102	23				3.3 P 2.2 M	
70 / 35				End of Boring at 31-1/2 ft.							

BORING DEPTH: 31.5 ft
BACKFILL: Grout
DEPTH TO WATER: 25.0 ft
COMPLETION DATE: August 4, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME-75
DRILLED BY: V & W Drilling, Inc., Juan Franco
LOGGED BY: R. Vedantham
CHECKED BY: M. Walker

LOG OF BORING NO. B-118
Phase-1 New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

APPENDIX A (CONTINUED)
HISTORICAL FIELD EXPLORATIONS

Project No. 1595.002

Sheet 1 of 1

LOCATION: SEE SITE PLAN, PLATE 3
N 2131764 E 6051547
California State Plane, NAD83, Zone 3, ft
SURFACE EL: 103.0 ft +/- (rel. NAVD88 datum)

ELEVATION, ft / DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
100 / 5		1		12	Asphalt Concrete Pavement **ARTIFICIAL FILL (af)** Lean CLAY (CL): firm to stiff, black, moist, trace fine gravel							
		2 3		(9)		92	22				1.8 M 0.5 R	Corrosion
		4		9	- 6" clayey sand layer, trace gravel, from 4 to 4.5'							
					ALLUVIUM (Qal)						2.0 P	
95 / 10		5 6		(18)	Lean CLAY with sand (CL): stiff to very stiff, brown, moist, fine-grained sand, low plasticity fines, trace subangular to subrounded fine gravel	102	23				3.0 M 1.0 R 2.5 P	
		7		14	- low to medium plasticity, trace fine-grained sand						3.5 P	
90 / 15		8 9		(28)	Clayey SAND with gravel (SC): medium dense, brown, moist to wet, fine- to coarse-grained sand, well graded, subangular to subrounded fine to coarse gravel up to 1.25-inch in size;			14				
85 / 20		10		16	▽ medium plasticity - with red iron staining, at 18'		17	15	31	17		
		11		18	- increased sand content, at 20'							
80 / 25		12		8	Sandy Lean CLAY (CL): firm, gray, wet, very fine-grained sand, low plasticity			66				
75 / 30		13		17								
		14		36	Silty SAND with gravel (SM): dense, brown, moist, fine-to coarse-grained sand, medium plasticity, subangular to subrounded fine gravel							
70 / 35		15		32	Poorly-graded SAND with silt (SP-SM): dense, brown, moist, medium-to coarse-grained, low plasticity							
65					End of Boring at 36'							

BORING DEPTH: 36.0 ft
BACKFILL: Grout
DEPTH TO WATER: 18.0 ft
COMPLETION DATE: February 14, 2008
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 4-in. dia. Solid Stem Auger
HAMMER TYPE: Automatic Trip
RIG TYPE: CME 300
DRILLED BY: Taber Consultants
LOGGED BY: K. Gupta
CHECKED BY: M. Paquette

LOG OF BORING NO. B-112
New Hospital Replacement Project
Children's Hospital and Research Center
Oakland, California

BORING LOG OAK G:\JOBDOCS\1595\1595.002 CHRCO\GINT\1595.002.GPJ LIBRARY_092906OAK.GLB 4/22/08 12:55 p

Sept. 2008 Report ID: PB-112 (08)

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	ERT
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	9/7/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 2" AC/4" AB **AGGREGATE BASE,** 4 inches	Firm				13	23	96		LL=37, PI=21, Passing #200 Sieve = 77%
CLAY (CL), brown, silty, sandy (fine- to medium-grained), damp (with silt, some sand (fine- to medium-grained below 2½ feet)	Stiff		5		9 26	21	99		
(trace gravel (subangular, fine) below 8½ feet)			10		14	19	109	2.3	
SAND (SC), motled reddish-brown to borwn, fine- to coares-grained sand, with gravel (fine and coarse, subangular to angular), some clay, some interbedded gravel, damp	Medium Dense		15		40	14	120	1.0	
(some clay, moist to wet below 18½ feet)			20		31	▽ 17	115		Triaxial Test, Deviator Stress=0.79 ksf
			25		17				
CLAY (CL), mottled light rust brown and grey, silty trace sand (fine-grained), damp	Very Stiff		30		20				
SAND (SC), light orange brown, fine- to coarse-grained, gravelly (fine and coarse, subangular to angular), with clay, moist to wet	Medium Dense		35		31				
					48				

File Name: G:\ENGINEER\GINTW\PROJECTS\17979.GPJ Report Template: FUGRO Output Date: 3/14/02

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL WESTERN EXPANSION
Oakland, California

PROJECT NO.	DATE	BORING NO.
17979-CA	September 2000	EB-1

Sept. 2008 Report ID: PB-1 (02)

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	ERT
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	9/7/00

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SAND (SC), Continued,	Medium Dense								
	Dense		45		53				

Bottom of Boring = 45 Feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of penetration resistance values, see Appendix A.
3. Ground water level was measured at 22½ feet at the time of drilling.
4. The boring was backfilled with neat cement in accordance with Alameda County Public Works Department requirements.
5. LL = Liquid Limit, PI = Plasticity Index

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL WESTERN EXPANSION
Oakland, California

PROJECT NO.	DATE	BORING NO.
17979-CA	September 2000	EB-1

File Name: G:\ENGINEER\GINTW\PROJECTS\17979.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/12" AB									
FILL: CLAY, (CL/CH), dark brown, some gravel (subangular), trace sand, damp	Very Stiff				47				
					12				
FILL: SAND (SP), light brown, damp, (trench backfill)	Medium Dense		5		21				
CLAY (CL), brown, silty, some gravel (subangular to subrounded), trace sand (fine- grained), damp	Stiff								
			10		17				Passing #200 Sieve = 58%
(grading mottled brown and gray, some sand (fine- to coarse- grained), no gravel at 14 feet)			15		26	26	98	1.9	
			20		22	25	102		Passing #200 Sieve = 68%, LL=33, PI=19
SAND (SC), brown, fine- to coarse-grained, with gravel (subrounded to subangular), some clay, wet	Medium Dense				35	17			Passing #4 Sieve = 66%, #200 Sieve = 18%
CLAY (CL), reddish-brown mottled gray, silty, moist	Very Stiff		25						
(interbed of gravel between 27 and 28 feet					45	23	106	14.8	Passing #200 Sieve = 89% LL=43, PI=27

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-2

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), moist	Very Stiff								
(grading trace gravel (coarse, subrounded) at 34½ feet)			35		37				
SAND (SC), brown, fine- to coarse-grained, with gravel (coarse, subangular to subrounded), some clay and silt, wet	Dense								Passing #4 Sieve = 62%, #200 Sieve = 18%
CLAY (CL), brown mottled gray, sandy (fine- to coarse- grained), some gravel (coarse, subangular to subrounded), trace silt, damp	Hard		40		54	16			Passing #200 Sieve = 57%, LL=39, PI=23
SAND (SC), brown, fine- to coarse-grained, some clay and silt, trace gravel (fine to coarse, subangular to subrounded), moist	Dense		45		52	21			Passing #200 Sieve = 26%
CLAY (CL), brown, silty, some sand (fine- to coarse- grained), damp	Hard		50		61	25	102	18.3	

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid Limit, PI=Plasticity Index

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FUGRO
425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-2

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Enc.	BORING DIAMETER	5-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
3" Asphaltic Concrete									
FILL: CLAY (CL), brown mottled rust and blue, silty, with sand (fine- to coarse-grained), some gravel (fine- to coarse-grained), trace organics	Firm				14*				PP = 1.5 tsf
	Stiff				13				
(less gravel and organics)			5						
(light brown, no organics)			10		25*	18	108	2.9	PP = 3.5 tsf
CLAY (CL), light brown mottled rust, with sand (fine- to coarse-grained), some silt, some gravel (fine-grained)	Stiff		15		16*	18	109	1.5	PP = 1.7 tsf
SAND (SM), light brown mottled gray, fine- to coarse-grained, gravelly (fine-grained), some silt and clay	Medium dense		20		29*				

Bottom of Boring = 20-1/2 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not encountered at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

Kaldveer Associates
Geoscience Consultants
A California Corporation

EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	1

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/12" AB underlain by geosynthetic									
CLAY (CL), brown, with sand (fine-grained), some gravel (fine, weak subangular sandstone fragments), trace silt, damp	Very Stiff				47				
					24				
			5		54	22	102		
(grading gravelly at 8½ feet)	Stiff				19				
			10						
SAND (SC), brown, fine- to coarse-grained, with gravel (coarse, subangular to subrounded), some clay and silt, damp	Medium Dense				43				Passing #4 Sieve = 63%, #200 Sieve = 20%
			15						
(grading gray-brown, fine- grained, clayey, some silt, moist to wet at 18 feet)					26	24	101	1.5	Passing #200 Sieve = 48%, LL=28, PI=7
			20						
(grading reddish-brown, fine- to coarse-grained, some gravel (fine to coarse, subrounded to subangular), some clay, wet					45	20			Passing #4 Sieve = 71%, #200 Sieve = 15%
			25						Passing #200 Sieve = 75%, LL=38, PI=24
CLAY (CL), brown mottled gray, silty, some sand (fine- to coarse- grained), damp	Very Stiff				48	21	111	9.7	

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-3

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), brown mottled gray, silty, some sand (fine- to coarse- grained), damp	Very Stiff								
GRAVEL (GC), brown, fine to coarse, subrounded to subangular, some sand (fine- to coarse- grained), some clay, wet	Dense		35		58	13			Passing #4 Sieve = 45%, #200 Sieve = 16%
SAND (SC), brown, fine- to coarse-grained, silty, trace gravel (fine to coarse, subrounded to subangular), moist	Dense		40		52	22			Passing #4 Sieve = 97%, #200 Sieve = 41%, LL=46, PI=28
CLAY (CL), rust brown, sandy (fine- to coarse- grained), some silt, wet	Very Stiff		45		43	21	112	11.6	Passing #200 Sieve = 54%, LL=44, PI=28
(grading silty, with sand (fine- grained), damp to moist at 48½ feet)	Hard		50		51	22	108		Passing #200 Sieve = 63%, LL=36, PI=19

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid Limit, PI=Plasticity Index

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-3

File Name: G:\ENGINEER\GINTW\PROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT SECTION: 4" AC/18" AB									
CLAY (CL/CH), black, with silt, trace gravel (subrounded), damp	Very Stiff				30				
					23				
GRAVEL (GC), brown, coarse, subrounded to subangular, clayey, with sand (fine- grained), damp	Dense		5		31				
CLAY (CL), brown, silty, some gravel (fine to coarse, subrounded to subangular), trace sand (fine- grained), damp	Stiff				25	18	109		
			10						
GRAVEL (GW), brown, fine to coarse, subrounded to subangular, sandy (fine- to coarse- grained), damp	Medium Dense				26				
			15						
SAND (SM), brown, fine- to coarse-grained, with gravel (fine to coarse, subrounded to subangular), some silt, moist	Dense				43	▼			Passing #4 Sieve = 61%, #200 Sieve = 17%
			20						
(grading clayey sand at 23 feet)	Medium Dense					▽			Passing #4 Sieve = 66%, #200 Sieve = 15% at 23½ ft
(lens of sandy clay at 24½ feet)					35	23			Passing #200 Sieve = 68%, LL=38, PI=22 at 24½ ft
(grading sandy gravel (fine- grained), moist at 25 feet)			25			30			
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), damp	Very Stiff				38	23	106	10.2	Passing #200 Sieve = 94%, LL=43, PI=28

FUGRO
425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-4

File Name: G:\ENGINEER\GINTWPROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	91 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	18 feet	BORING DIAMETER	8-inch	DATE DRILLED	3/4/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), reddish-brown mottled gray, silty, trace sand (fine- grained), damp	Very Stiff								
	Hard				40				
SAND (SC), brown, fine- to coarse-grained, some gravel (fine, subangular), trace silt, wet	Dense		35						
(grading some silt and clay at 39 feet)			40		54	16			Passing #4 Sieve = 71%, #200 Sieve = 27%, LL=40, PI=23
			45		59				Passing #4 Sieve = 75%, #200 Sieve = 22%
CLAY (CL), brown, with sand (coarse-grained), damp	Hard		50		31				

Bottom of Boring = 50 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lbdown-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with a neat cement grout immediately upon completion.
5. LL=Liquid limt, PI=Plasticity Index

FUGRO
425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

OAKLAND CHILDRENS HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
21110-GI	March 2002	EB-4

File Name: G:\ENGINEER\GINTWPROJECTS\21110-GI.GPJ Report Template: FUGRO Output Date: 3/14/02

DRILL RIG	CME 75 HSA	SURFACE ELEVATION	96 feet	LOGGED BY	MHL
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	11/24/97

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT: 2-1/2-inched A.C. over 9-1/2-inches A.B.					50/5"				
FILL: SAND (SW), yellowish-brown, fine-to coarse-grained, some gravel (fine to coarse, angular), trace to some clay, dry	Dense								
	Dense				44	21	94		LL=37%, PI=21%, Passing #200 Sieve=72%
FILL: SAND (SC), yellowish-brown, fine-to-coarse-grained, some clay, trace gravel (fine to coarse, angular), dry-damp	Very Dense		5		51				
FILL: CLAY (CL), dark brown, silty, some sand (fine grained), trace gravel (fine, angular), damp									
CLAY (CL), brown to dark brown, silty, some sand (fine grained), damp	Hard				50/5"	21	107		
(grey with yellowish-red iron staining, with sand (fine-to medium-grained), trace gravel (fine to medium, rounded) below 12 feet)			10		50/6"	18	111		
			15						
SILT (ML), light grey with yellowish-red iron staining, some sand (fine grained), trace pockets of wet clay, damp	Hard				75	20	108		
			20		37				

Bottom of Boring = 20 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. The boring was backfilled with native soil cuttings immediately upon completion.

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EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL HELISTOP
Oakland, California

PROJECT NO.	DATE	BORING NO.
L998G	December 1997	EB-1

DRILL RIG	gCME 75 HSA	SURFACE ELEVATION	96 feet	LOGGED BY	MHL
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	11/24/97

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
PAVEMENT: 2-inches A.C. over 8-inches A.B.									
FILL: SAND (SC), brown-dark brown, fine-grained, silty, some clay, trace gravel (fine to medium, subangular), trace wood and brick debris, damp	Med. Dense								LL=31%, PI=19%, Passing #200 Sieve=35%
CLAY (CL), dark brown with yellowish-red iron staining, silty, trace to some sand (fine grained), damp	Hard		5		29				
					37				
	Very Stiff				52	21	106		
SILT (ML), grey with yellowish-red iron staining, some sand (fine grained), some clay, damp	Very Stiff		10						
SAND (SC), yellowish-brown, fine-to coarse-grained, some clay, some gravel (fine to medium, subrouned), damp-moist (yellowish-brown with red iron and black staining below 13-1/2 feet)	Med. Dense				22	17	108		
			15		25				

Bottom of Boring = 16-1/2 Feet

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see the first page of Appendix A.
3. The boring was backfilled with native soil cuttings immediately upon completion.

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EXPLORATORY BORING LOG

CHILDREN'S HOSPITAL HELISTOP
Oakland, California

PROJECT NO.	DATE	BORING NO.
L998G	December 1997	EB-2

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	18 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
4 inches of asphaltic concrete, 5 inches of aggregate baserock									
SILT (ML), dark brown, clayey, some sand (fine-grained), trace rootlets, dry	Stiff		5		21	16	99		PI=19%, LL=36%, Passing #200 Sieve=72%
(coarse-grained sand apparent, trace gravel (fine, subangular) at 9-1/2 to 11 feet)	Very Stiff		10		39 35				PP=9.0+ ksf, TV=4.5+ ksf
CLAY (CL), olive gray with rust mottling, silty, moist	Very Stiff		15		32	22	103		PP=6.5 ksf, TV=2.8 ksf
	Stiff		20		23				PP=3.0 ksf, TV=2.3 ksf
SAND (SC), brown, fine- to coarse-grained, with clay, and gravel (fine, subangular), wet	Medium Dense		25		30 28				
CLAY (CL), rust brown to brown, with sand (fine- to coarse-grained), trace gravel (fine, angular to subrounded), moist	Hard		30		38				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-3

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	18 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), (continued) (sandy at 34-1/2 to 36 feet)	Hard		40		93/11"				
(sand appears cemented below 40 feet)			45						
			50		89				
Bottom of Boring = 51 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 24 feet at time of drilling. After completion of drilling, ground water was measured at 18 feet. 4. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot. 5. PI = Plasticity Index, LL = Liquid Limit.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-3

Sept. 2008 Report ID: PB-3 (94)

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	20 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
6 inches asphaltic concrete, 7 inches aggregate baserock									
CLAY (CL), dark brown to black, silty, damp	Very Stiff		5		30				PP=9.0 ksf, TV=4.3 ksf
(brown, plasticity decreases below 9-1/2 feet)			10		36				PP=9.0 ksf, TV=4.5+ ksf
CLAY (CL), olive gray with rust mottling, silty, moist	Very Stiff		15		24 15	23	102		PP=8.0 ksf, TV=4.3 ksf PP=8.0 ksf, TV=2.5 ksf
(trace sand (fine- to coarse-grained) and gravel (fine, subrounded) at 19-1/2 to 20 feet)			20		32				
SAND (SC), brown, fine- to coarse-grained, with gravel (fine, subangular to subrounded), trace clay, wet	Medium Dense		25		26				
(some clay at 28 to 28-1/2 feet)									
CLAY (CL), rust brown, with gray mottling, some sand (fine- to coarse-grained), moist	Hard		30		59				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-4

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	98 Feet	LOGGED BY	CDT
DEPTH TO GROUND WATER	20 Feet	BORING DIAMETER	8-inch	DATE DRILLED	7/31/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL) (continued)	Hard		40		65				
(cemented at 38 to 30-1/2 feet)			45						
(sand not apparent at 48-1/2 to 50 feet)			50		97/11"				
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 22 feet at time of drilling. After completion of drilling, ground water was measured at 20 feet. 4. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-4

Sept. 2008 Report ID: PB-4 (94)

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	101 Feet	LOGGED BY	CDT/MJ
DEPTH TO GROUND WATER	22 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/3/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL), dark brown, silty, trace rootlets	Stiff								
SAND (SM), brown, fine- to coarse-grained, with gravel (fine, subangular to subrounded), trace silt, damp	Loose		5		15				
	Medium Dense		10		29				
CLAY (CL), brown, with sand (fine- to coarse-grained), trace silt, moist	Firm		15		5				
	Stiff				14	19	107	1.1	TV=2.5 ksf
(tan-gray with rust mottling, damp at 18-1/2 to 20 feet)	Stiff to Very Stiff		20		26				PP=4.0 ksf, TV=3.0 ksf
(moist to wet at 23-1/2 to 25 feet)	Stiff		25		17				
SAND (SM), tan-gray, fine- to medium-grained, some silt, wet	Medium Dense		30		13				Passing #200 Sieve=25%
(trace gravel (fine, subangular) at 33-1/2 to 35 feet)	Dense				45				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-1

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	101 Feet	LOGGED BY	CDT/MJ
DEPTH TO GROUND WATER	22 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/3/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
SAND (SM) (continued)	Dense		40		14				
CLAY (CL), gray, silty, damp to moist	Stiff		45						
(with sand (fine- to medium-grained at 48-1/2 to 50 feet)	Hard		50		58				PP=9.0 ksf, TV=5.0 ksf
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 27 feet at time of drilling. 4. An observation well was installed in this boring to periodically measure the ground water depth. 5. Ground water was measured at 22 feet on 9/22/92. 6. PP = Compressive Strength by Pocket Penetrometer, TV = Torvane Shear Strength, ksf = kips per square foot.									

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-1

Sept. 2008 Report ID: PB-1 (94)OW

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	97 Feet	LOGGED BY	MJ
DEPTH TO GROUND WATER	16 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/4/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
8 inches asphaltic concrete, 4 inches aggregate baserock									
CLAY (CL), dark brown, silty, trace gravel (fine, subangular), damp	Very Stiff								
(red-brown with gray mottling, trace sand (fine-grained) at 4-1/2 to 6 feet)			5		24				
			10		19				
SAND (SC), red-brown, fine- to coarse-grained, with gravel (fine, subangular), moist	Medium Dense								
(gravelly, wet at 13 to 14-1/2 feet)	Dense				35				
			15			▼			
(fine-grained sand layer at 17-1/2 feet)	Medium Dense				20				Passing #200 Sieve = 36%
			20						
					24				
			25						
CLAY (CL), tan-brown with gray mottling, trace sand (fine-grained), moist	Very Stiff				29				
			30						
(with sand at 28 to 29-1/2 feet)	Hard				55				

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-6

DRILL RIG	CME-45, HSA	SURFACE ELEVATION	97 Feet	LOGGED BY	MJ
DEPTH TO GROUND WATER	16 Feet	BORING DIAMETER	8-inch	DATE DRILLED	8/4/92

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
CLAY (CL) (continued)	Hard								
SAND (SC), red-brown, fine- to coarse-grained, with clay, trace gravel (fine, angular), damp	Very Dense		40		84				Passing #200 Sieve = 21%
			45						
	Dense		50		74				
Bottom of Boring = 50 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values, see first page Appendix A. 3. Ground water level was measured at 17-1/2 feet at time of drilling. 4. An observation well was installed in this boring to periodically measure the ground water depth. 5. Ground water was measured at 16 feet on 9/22/92.									

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG

PATIENT SERVICES PAVILLION - PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO
K938-8-1421	February 1994	EB-6

Sept. 2008 Report ID: PB-6 (94)OW

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2" Asphaltic Concrete over 2" Gravel									
FILL: CLAY (CL), light brown mottled blue-green and black, silty, with sand (fine- to coarse-grained), trace gravel (fine-grained), trace organics	Firm				15*	18	102		PP = 1.5 tsf
	Stiff		5		10				
(dark brown mottled rust and gray, some gravel (fine-grained))									
(brown mottled rust and gray, with sand (fine- to coarse-grained), some gravel (fine- to coarse-grained))	Firm		10		9*	16	107	0.9	PP = 1.0 tsf
GRAVEL (GM), gray-brown, fine-grained, some sand (fine- to coarse-grained), trace silt and clay	Medium dense		15		21*				
CLAY (CL), tan and rust, some silt, trace sand (fine-grained)	Firm		20		7				
(some sand (fine- to coarse-grained), trace gravel (fine-grained))			25		17*	23	102	2.5	PP = 1.2 tsf
(silty, some gravel (fine-grained))	Stiff		30		27*				PP = 1.7 tsf
Bottom of Boring = 30-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not established at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	2

DRILL RIG	Continuous Flight Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Enc.	BORING DIAMETER	5-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2-1/2" Asphaltic Concrete over 4-1/2" Gravel									
FILL: CLAY (CL), brown, silty, trace sand (fine- to coarse-grained), trace gravel (fine-grained)	Firm				15*	20	90		PP = 2.0 tsf
(some sand (fine- to coarse-grained), more gravel)	Stiff		5		10				
(brown mottled rust and gray, some gravel (fine-grained))			10		38*	17	107	3.4	PP = 4.0 tsf
CLAY (CL), brown, silty, trace sand (fine- to medium-grained)	Stiff		15		248				
GRAVEL (GM), brown mottled rust and gray, fine- to coarse-grained, some sand (fine- to coarse-grained), trace silt and clay	Medium dense		20		37*				
Bottom of Boring = 20-1/2 Feet									

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A.
3. The groundwater level was not encountered at the time of drilling.
4. PP = Pocket Penetrometer
5. The boring was backfilled with cuttings and capped with asphaltic concrete.

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	3

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
2" Asphaltic Concrete									
FILL: CLAY (CL), black and blue-green, silty, some sand (fine- to coarse-grained), trace gravel (fine-grained)	Stiff				23*	19	102		PP = 3.2 tsf
(brown mottled rust and gray, some gravel (fine- to coarse-grained), trace organics)			5		9				
(with sand (fine- to coarse-grained), trace gravel (fine- to coarse- grained), no organics)			10		25*	13	110	2.0	PP > 4.5 tsf
(sandy (fine- to coarse-grained))			15		8				
GRAVEL (GM), gray, fine-grained, trace sand (fine- to coarse-grained), trace silt and clay	Loose		20		6				
SILT (ML), tan mottled rust, sandy (fine-grained), trace clay, trace sand (medium- to coarse-grained)	Stiff				19*	28	95	2.8	PP = 1.5 tsf
(less sand)			25						
			30						

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	4

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	95.5 ft.	LOGGED BY	MC
DEPTH TO GROUNDWATER	Not Est.	BORING DIAMETER	8-inch	DATE DRILLED	1/3/92

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SILT (ML), tan mottled rust, with sand (fine-grained), trace clay, trace sand (medium- to coarse-grained)	Stiff								
	Firm		35		14*	24	100		PP = 1.2 tsf
(same as above)			40						
(same as above)	Stiff								
CLAY (CL), orange-brown mottled black and rust, some sand (fine-grained), trace sand (medium- to coarse-grained)	Hard		45						
			50		62*				PP > 4.5 tsf
Bottom of Boring = 50-1/2 Feet Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*), see the first page of Appendix A. 3. The groundwater level was not established at the time of drilling. 4. PP = Pocket Penetrometer 5. The boring was backfilled with cuttings and capped with asphaltic concrete.									

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EXPLORATORY BORING LOG

CHORI LAB ADDITION / CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO
K 938-6-1125	January 1992	4

Sept. 2008 Report ID: PB-4 (92c)

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 97 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	18' (See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 4" Baserock									
SAND, (fine-grained), clayey, with silt, traces of gravel (fine-grained) Liquid Limit = 30% Plasticity Index = 12% Passing #200 Sieve = 49%	dark grey	medium dense	SC-CL	1 2 3		30 13*	17		
SILT, sandy, (fine-grained), trace of gravel, (fine grained) Passing #200 Sieve = 54%	tan	stiff	ML	4 5		9	24		
CLAY, silty, some sand, (fine to coarse-grained) (gravel fine grained more sand) Liquid Limit = 33% Plasticity Index = 33% Passing #200 Sieve = 51%	grey mottled with tan	stiff very stiff	CL CL	6 7 8 9 10		42*	14	114	10.5
(grading more gravel, fine-grianed)	light grey			11 12 13 14 15		34*	24	101	8.6
SAND, (fine-to coarse grained) silty, trace of gravel, (fine grained), some clay	light grey mottled with tan	medium dense	SM	16 17 18			▽		
Passing #200 Sieve = 51%			SM-ML	19 20		47*			

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	1

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 97 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	18' (See Note	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse-grained), some clay and gravel (fine-grained)	light grey	medium	SC	21		47*			
GRAVEL, (fine-grained), sandy, (fine-to coarse-grained), some clay and silt	brown	medium dense	GC	22 23 24					
CLAY, silty, sandy (fine-to coarse-grained), with gravel (fine grained)	grey mottled with tan	stiff	CL	25		30*	20		
GRAVEL (fine-grained), sandy, (fine-to coarse-grained), some silt and clay Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see frist page Appendix A. 3. Groundwater level was measured at a depth of 20 feet at time of drilling. 3½ hours later the groundwater level was measured at 18 feet.	tan	very dense	GC	26 27 28 29 30 31 32 33 34 35		94*/11	16	119	12.0
(sandy clay lense)				36 37 38 39 40		85 72	18		

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	1

Sept. 2008 Report ID: PB-1 (87)

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 94 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	19½'(See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 4" Baserock									
CLAY, silty gravelly (fine-grained), some sand (fine-to coarse-grained)	dark grey	very stiff	CL	1–2		57*	19	114	
GRAVEL (fine-grained) sandy, (fine-to coarse-grained), with clay Passing #200 Sieve = 17%	grey	medium dense	GC	3–4		28*			
CLAY, silty some sand (fine-grained) traces of gravel (occasional gravel lenses)	dark grey	very stiff	CL	5		17			
(grading gravelly, fine-grained)	mottled with tan			9–10		58*	17		
(grading with traces of gravel fine grained)				15–16		47*	21		
SAND, (fine-to coarse-grained), some silt, some clay	brown	medium dense	SC-SM	18					
(occasional clay lense)				20		28*	22	101	

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, Calfiornia

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	2

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 94 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	19½'(See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse grained), gravelly, (fine-grained), with silt, some clay (occasional clay lense) Passing #200 Sieve = 38%	brown	medium dense	SC-SM	21–25		28			
CLAY, sandy, (fine to coarse-grained), with gravel (fine grained), silty (occasional sand or gravel lense)	light grey	firm	CL-SC	28–30		16*	22	104	0.5
(grading without gravel, less sand) Liquid Limit = 30% Plasticity Index = 16% Passing #200 Sieve = 79%		stiff	CL	35–36		20*	27	96	0.6
SAND, (fine to coarse-grained) some clay and silt, trace of gravel (fine-grained)	brown	dense	SC	40		39*	29		

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EXPLORATORY BORING LOG
NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K339-7	December 1987	2

Sept. 2008 Report ID: PB-2 (87)

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	± 94 Feet	LOGGED BY	LG
DEPTH TO GROUNDWATER	19½' (See Note 3)	BORING DIAMETER	8 Inches	DATE DRILLED	12/11/87

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND, (fine-to coarse grained) some silt, trace of gravel (fine-grained) (occasional gravel lense)	brown	medium dense	SC	41		39*			
(grading gravelly)				45		68			
CLAY, silty, with sand (fine to coarse-grained) (trace of gravel)	light grey	hard	CL	46					
CLAY, silty, sandy (medium-grained)	rust	hard	CL	49		58*/6"	21		
Bottom of Boring = 51 Feet				51					
Notes: 1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual. 2. For an explanation of penetration resistance values marked with an asterisk (*) see first page, Appendix A. 3, Groundwater level was measured at a depth of 19½ feet at time of drilling.				60					

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EXPLORATORY BORING LOG

NURSING TOWER
Oakland, California

PROJECT NO.	DATE	BORING NO.
K 339-7	December 1987	2

DRILL RIG	Hollow Stem Auger	SURFACE ELEVATION	--	LOGGED BY	F.M.
DEPTH TO GROUNDWATER	23' (SeeNote3)	BORING DIAMETER	8 Inches	DATE DRILLED	6/22/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
4" AC over 7" BASEROCK				1					
CLAY, silty with some sand (fine-to coarse grained)	mottled brown-black	stiff	CL	2		10	12		
CLAY, silty, sandy (fine-grained) with chunks of masonry pipe	mottled browns-blacks	stiff	CL-ML	3		10	9		
Dry Density = 104 pcf				4					
				5		20*			>5.0
(Possible Fill)				6					
				7					
CLAY, very sandy (fine to coarse grained) with gravel to ½"	mottled browns-black	hard	CL	8					
				9		78*		1.8	3.5
Liquid Limit = 35% Plasticity Index = 21% Passing #200 Sieve = 55% Dry Density = 115 pcf				10					
				11					
				12					
				13					
				14		36*			
				15					
				16					
				17					
				18					
SAND (fine to coarse grained) with clay and gravel to ½"	mottled brown-grey-black	medium dense	SC	19		23	23		
Passing #200 Sieve = 16%				20					

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EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION -- | LOGGED BY F.M.

DEPTH TO GROUNDWATER 23' (SeeNote3) | BORING DIAMETER 8 Inches | DATE DRILLED 6/21/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (fine to coarse grained) with clay and gravel to ½" (continued)	mottled brown-grey-black	medium dense	SC	21–23			▽		
SILT, clayey with sand (fine-grained)	mottled tan-rust	stiff	ML	24–25		12	25		
CLAY, silty, sandy (fine to medium grained)	blue-green	stiff	CL	26–30		12	19		
SAND (fine to coarse grained) gravelly to 1", with some clay (increasing clay content) Passing #200 Sieve = 19%	mottled brown-grey-black	dense	SC-GC	31–34		push 500-1000 psf	13		
				35–40		32	13		

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EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

DRILL RIG Hollow Stem Auger | SURFACE ELEVATION -- | LOGGED BY F.M.

DEPTH TO GROUNDWATER 23' (SeeNote3) | BORING DIAMETER 8 Inches | DATE DRILLED 6/22/78

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (fine to medium grained) gravelly to 1" with some clay (continued)	mottled browns-black-red	dense	GC	41–45		52	16		
CLAY, silty with some sand (fine to medium grained)	mottled red-brown-black	hard	CL	46–49		46	20		
Bottom of Boring = 50 Feet				50					

Notes:
1. The stratification lines represent the approximate boundaries between soil types and the transitions may be gradual.
2. For an explanation of penetration resistance values marked with an asterisk (*), see page A-1.
3. Groundwater level measured at time of drilling.

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EXPLORATORY BORING LOG

ADDITION TO CHILDREN'S HOSPITAL
Oakland, California

PROJECT NO.	DATE	BORING NO.
K480-5	July 1978	6

Sept. 2008 Report ID: PB-6 (78)

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE BLOWS/FT.	WATER CONTENT %	SHEAR STRENGTH BY TORVANE KSF	UNCONFINED COMPRESSIVE STRENGTH KSF
CLAY, silty, sandy Liquid Limit = 22% Plasticity Index = 19% Passing #200 Sieve = 62% (grading sandier with depth)		dark brown	firm	CL	1–5		5	26		
Liquid Limit = 36% Plasticity Index = 17%			stiff		5–10		12	19		
CLAY, sandy, gravelly Dry Density = 105 pcf		tan	stiff	CL	10–15			22		1.3
(grading less gravelly with depth)					15–20		11	21		
CLAY, silty		tan	stiff	CL						

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE BLOWS/FT.	WATER CONTENT %	SHEAR STRENGTH BY TORVANE KSF	UNCONFINED COMPRESSIVE STRENGTH KSF
CLAY, silty Dry Density = 102 pcf		tan	stiff	CL	21–23			24	0.6	1.2
SAND (well graded), gravelly, clayey Passing #4 Sieve = 70% Passing #50 Sieve = 29% Passing #200 Sieve = 15%		tan	medium dense	SC	23–29		19			
CLAY, silty Dry Density = 104 pcf		mottled tan & brown	very stiff	CL	29–33			29		6.1
SAND (well graded), gravelly		tan	medium dense	SW-SP	33–36		19	19		
CLAY, silty		tan	very stiff	CL	36–40					
SAND (well graded), clayey, gravelly		tan	dense	SC						

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/9/73

DESCRIPTION AND REMARKS	SYM BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (well graded), clayey, gravelly Passing #4 Sieve = 78% Passing #50 Sieve = 41% Passing #200 Sieve = 26%		tan	dense	SC	41–44		34			
CLAY, sandy, gravelly Passing #200 Sieve = 57% Dry Density = 111 pcf		tan	hard	CL	45–47			20		9.1
GRAVEL (well graded), sandy		brown	dense	GW	48–49					
CLAY, silty, sandy Liquid Limit = 35% Plasticity Index = 18%		tan	very stiff	CL	50–53		31	17		
SAND (well graded), clayey, gravelly Passing #4 Sieve = 78% Passing #50 Sieve = 42% Passing #200 Sieve = 29%		brown	very dense	SC	54–56		70			
CLAY, silty, sandy		tan	hard	CL	57–60					

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EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYM BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty, sandy Dry Density = 124 pcf		tan	hard	CL	61					
SAND (well graded), clayey, gravelly		tan	dense	SC–SW	62–63		15			
CLAY, silty, sandy Dry Density = 104 pcf		brown	hard	CL	64–67			24		4.7
Dry Density = 113 pcf (grading gravelly with depth)					68–80			19		7.7

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EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT %	SHEAR STRENGTH BY TORVANE KSF	UNCONFINED COMPRESSIVE STRENGTH KSF
CLAY, silty, sandy, gravelly		brown	hard	CL	81		73			
(grading less gravelly with depth)					82–84					
					85					
Dry Density = 105 pcf					86–87			23		5.7
					88–89					
(grading stiffer with depth)					90–94					
					95					
Dry Density = 107 pcf					96–97			21		6.9
					98–100					

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

DRILL RIG Rotary Wash | SURFACE ELEVATION --- | LOGGED BY A.K.

DEPTH TO GROUNDWATER See Note 2 | BORING DIAMETER 6 Inches | DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT %	SHEAR STRENGTH BY TORVANE KSF	UNCONFINED COMPRESSIVE STRENGTH KSF
CLAY, silty, sandy		brown	hard	CL	101–102					
SAND (well graded)		brown	very dense	SC	103–104					
Passing #200 Sieve = 40%					105–106		85/11"			
					107–108					
CLAY, silty, sandy		brown	hard	CL	109–110					
					111–114					
(grading sandier with depth)					115					
Dry Density = 109 pcf					116–117			20		7.3
SAND (well graded), clayey		brown	very dense	SC	118–119					
					120					

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EXPLORATORY BORING LOG

PATIENT SERVICES PAVILION–PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1994	PB74-1

Sept. 2008 Report ID: PB-1 (74)

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
SAND (well graded), clayey		brown	very dense	SC	121					
CLAY, silty		tan	hard	CL	122					
					123					
					124					
					125					
Dry Density = 112 pcf					126			19		14.0
					127					
					128					
(sandy between depths of 129 and 130 feet)					129					
					130					
					131					
					132					
					133					
					134					
					135					
Dry Density = 104 pcf					136			23		6.8
					137					
					138					
					139					
					140					

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

DRILL RIG Rotary Wash	SURFACE ELEVATION ---	LOGGED BY A.K.
DEPTH TO GROUNDWATER See Note 2	BORING DIAMETER 6 Inches	DATE DRILLED 11/10/73

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT (%)	SHEAR STRENGTH BY TORVANE (KSF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)
CLAY, silty		tan	hard	CL	141					
					142					
					143					
(grading sandier with depth)					144					
					145					
CLAY, silty, sandy		dark brown	hard	CL	146					
					147					
					148					
Dry Density = 114 pcf					149			19		12.8
					150					
Bottom of Boring = 150 Feet										

Notes:

(1) The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

(2) The groundwater level was not established because of the use of bentonite during drilling.

HARZA KALDVEER
Consulting Engineers

EXPLORATORY BORING LOG
PATIENT SERVICES PAVILION-PHASE I
Oakland, California

PROJECT NO.	DATE	BORING NO.
K938-8-1421	February 1974	PB74-1

Sept. 2008 Report ID: PB-1 (74)

APPENDIX B
LABORATORY TESTING PROGRAM

APPENDIX B
LABORATORY TESTING PROGRAM

The laboratory-testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. A summary of the laboratory tests performed is included as Figure B-1.

The natural water content was measured on 27 samples recovered from the borings in accordance with ASTM Test Designation D-2216. Water contents are recorded on the boring logs at the appropriate sample depths.

Dry density was measured on 16 samples of the subsurface soils in accordance with ASTM Test Designation D-2937. The results of these tests are shown on the boring logs at the appropriate sample depths.

Atterberg limits were performed on 11 samples of the subsurface soils to estimate the range of water content over which these materials exhibit plasticity. The Atterberg limits were performed in accordance with ASTM Test Designations D-428 and D-424. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of these tests are presented on Figure B-2a, and on the logs of the borings at the appropriate sample depths.

The percent passing the No. 200 sieve was measured on 14 samples of the subsurface soils to aid in the classification of these soils. These tests were performed in accordance with ASTM Test Designation D-1140. The results of these tests are shown on the boring logs at the appropriate sample depths.

Sieve analyses were performed on four samples of the subsurface soils to aid in the classification of these soils. This test was performed in accordance with ASTM Test Designation C136. The results of these tests are presented on Figure B-2b.

Miniature vane tests were performed on nine samples of the subsurface soils to estimate the undrained shear strength and residual strength of these layers to provide data for bearing capacity and foundation recommendations. These tests were performed in accordance with ASTM Test Designation D4648. The results of these tests are presented on the boring logs at the appropriate sample depths.

Undrained shear strength was determined by unconsolidated, undrained triaxial testing for one representative sample of the materials recovered from the borings in accordance with ASTM Test Designation D-2850. The strengths from the test are shown on the boring log at the appropriate sample depth.

Two Resistance (R-value) tests were performed on representative samples of the surface soils onsite to provide data for pavement design. The tests were performed in accordance with California Test Method 301-F and indicated a typical R-value of 4 and 9 at an exudation pressure of 300 pounds per square inch. The results of the tests are presented below:

R-value Test Results – Bulk Sample 1 (B-102)

Description of Material	Dry Density (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
Dark Brown fat CLAY with sand (CH)	92.1	27.7	223	0	2
	93.1	26.5	286	0	3
	94.9	25.4	342	0	8
R-Value = 4 at Exudation pressure of 300 psi					

R-value Test Results – Bulk Sample 2 (B-106)

Description of Material	Dry Density (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
Dark Brown lean to fat CLAY with sand (CL-CH)	100.6	21.8	231	0	5
	101.5	20.7	278	0	7
	105	19.5	334	0	14
R-Value = 9 at Exudation pressure of 300 psi					

California State Certified Laboratory No.2153

C E R C O
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

19 March 2008

Job No.0803031
Cust. No.11608

Mr. Kap Gupta
Fugro West, Inc.
1000 Broadway, Suite 440
Oakland, CA 94607

Subject: Project No.: 1595.002
Project Name: Oakland – Childrens Hospital
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. Gupta:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on March 05, 2008. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, all samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from none detected to 52 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration ranges from none detected to 110 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 5.6 to 7.5 which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Any soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potentials range from 410 to 450-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No.2153

CERCO analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

Client: Fugro West
Client's Project No.: 1595.002
Client's Project Name: Oakland - Childrens Hospital
Date Sampled: Not indicated
Date Received: 5-Mar-08
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 19-Mar-2008

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0803031-001	B-104/1 @ 2.5'	420	6.8	-	1,300	-	52	110
0803031-002	B-105/6 @ 15.5'	450	7.5	-	1,700	-	N.D.	N.D.
0803031-003	B-112/3 @ 4'	410	5.6	-	1,800	-	N.D.	33

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	11-Mar-2008	11-Mar-2008	-	13-Mar-2008	-	11-Mar-2008	11-Mar-2008

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel:925.462.2771
Fax:925.462.2775

CERCO analytical, inc.

Chain Of Custody

Page of

Job No. 080203 | Client No. 11608 | Client Project I.D. 1595.002

Sample Schedule: WK MO QT AN Resample New: | Date Sampled | Date Due

Contact Name: Kap Gupta

Company Name: Fugro West Oakland

Sample Source: Oakland - Childrens Hospital

Lab No.	Sample I.D.	Date	Time	Matrix	Container	Preserv.	Qty.	ASTM Test Methods* with Brief Evaluation	CalTrans Test Methods** with Brief Evaluation
001	B-104/1 @2.5	N					1	/	
002	B-105/6 @15.5						1	/	
003	B-112/3 @4.0						1	/	

MATRIX: DW - Drinking Water; GW - Ground Water; SW - Surface Water; WW - Waste Water; SL - Sludge; S - Soil

ABBREVIATIONS: HB - Hosebib; PV - Petcock Valve; PT - Pressure Tank; PH - Pump House; RR - Restroom; GL - Glass; PL - Plastic; ST - Sterile

SAMPLE RECEIPT: Total No. of Containers: 3; Rec'd Good Cond./Cold; Conforms to Record; Sampler

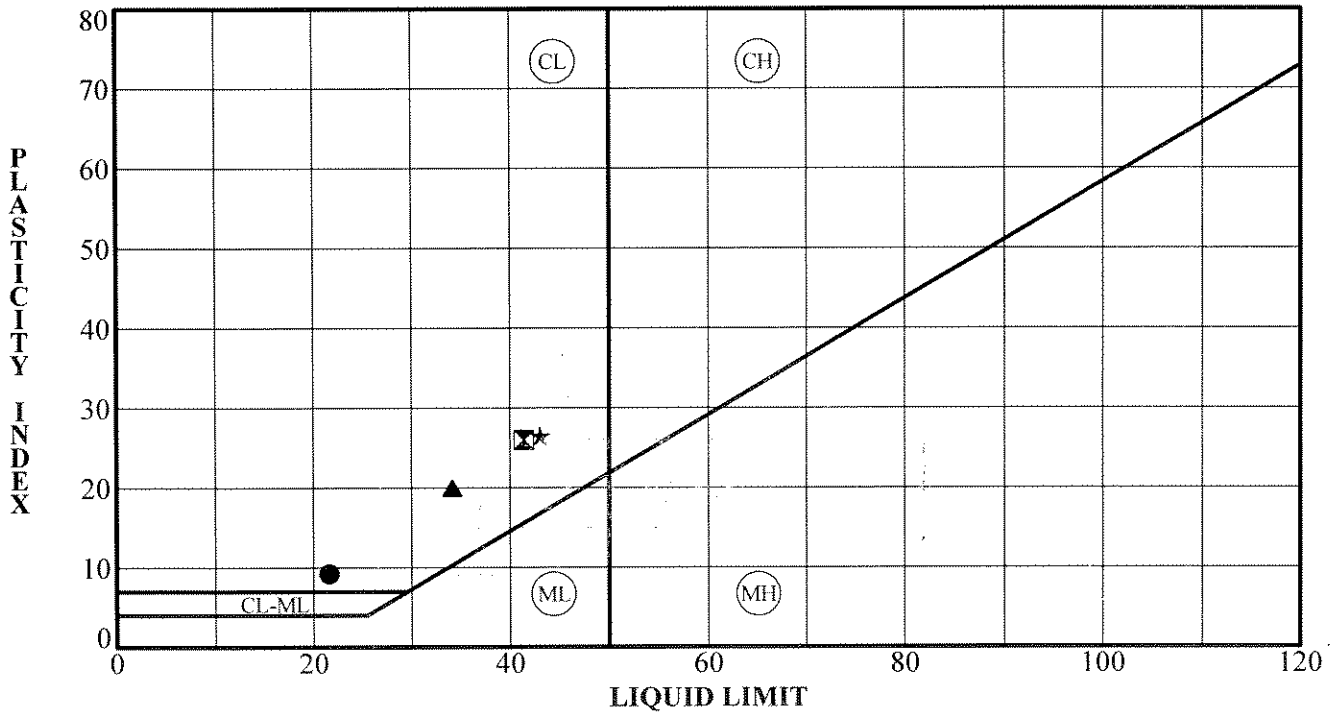
Relinquished By	Date	Time
	3/5/08	rd: 10
Received By	Date	Time
	3/5/08	10:08
Relinquished By	Date	Time
Received By	Date	Time
Relinquished By	Date	Time
Received By	Date	Time

Sample Location/Instructions

* Includes Redox, pH, Resistivity (100% Saturation), Chloride and Sulfate

** Includes pH, Minimum Resistivity, Chloride & Sulfate

There may be an additional charge for submitting soil in brass tubes.



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	B-113	2.5	22	9	-0.691	6		
⊠	B-114	8.5	41	26	---	---		
▲	B-114	15.0	34	19	---	---		CL
★	B-116	3.0	43	26	0.032	17		CL

FUGRO

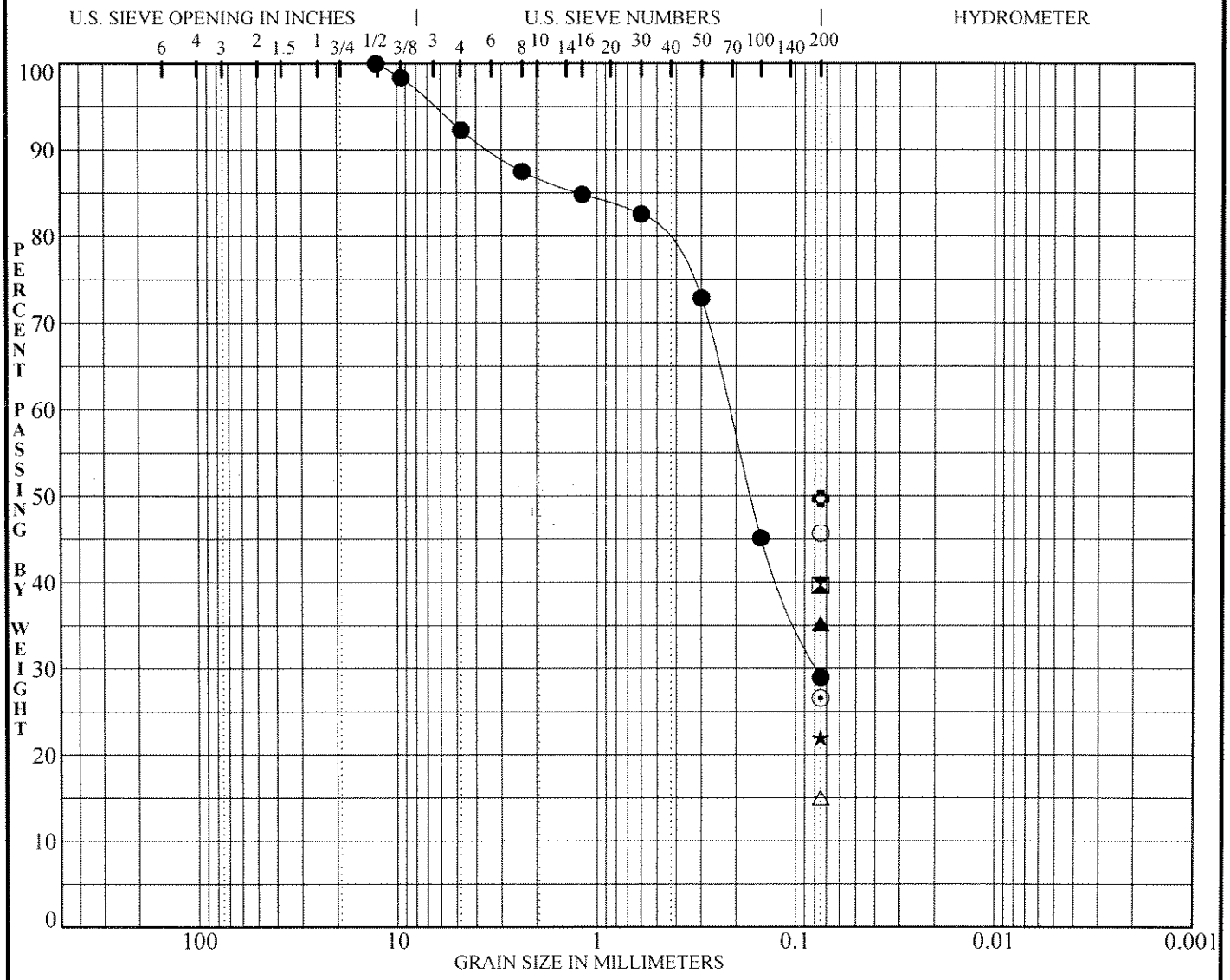
PREP'D BY: RV
APP'D BY: M. Walker
DATE: 8/21/08
DWG FILE:

PLASTICITY CHART AND DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-1
PROJECT No.
1595.002

ATTERBERG 1595.002.GPJ VERSION-012505.GDT 8/21/08



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	B-113	21.0	29	92	Brown silty SAND	SM
⊠	B-113	25.0	40			
▲	B-115	21.0	35		Clayey SAND with gravel	SC
★	B-115	23.0	22		Clayey SAND with gravel	SC
⊙	B-115	25.0	27		Clayey SAND with gravel	SC
✿	B-116	9.0	50			
○	B-116	15.5	46		Clayey SAND (SC)	SC
△	B-116	29.0	15			

FUGRO

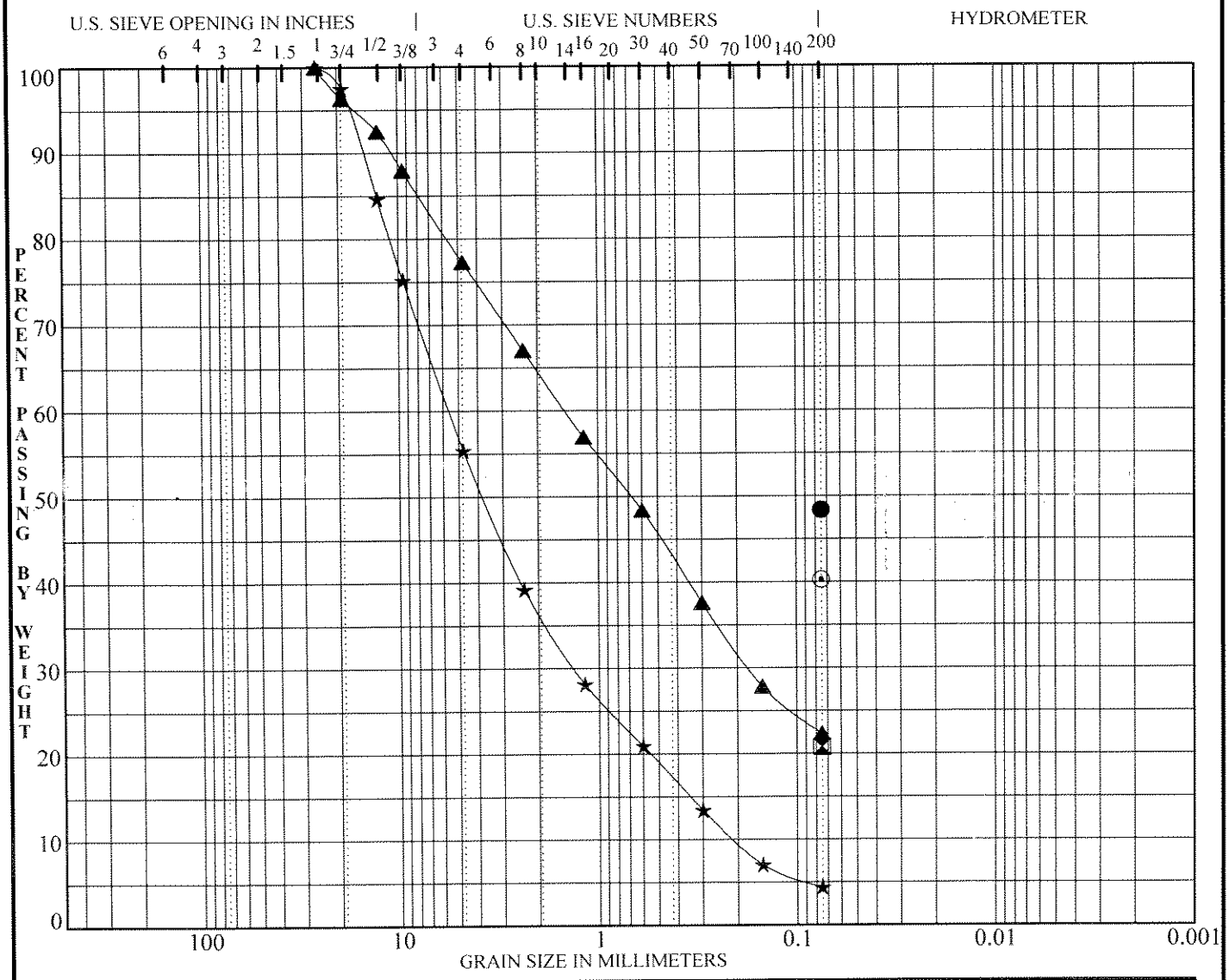
PREP'D BY: RV
APP'D BY: M. Walker
DATE: 8/21/08
DWG FILE:

GRADATION TEST DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-2a
PROJECT No.
1595.002

GRADATION_B 1595.002.GPJ STD.GDT 8/21/08



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	B-117	5.0	48		Brown clayey SAND	SC
⊠	B-117	26.0	21		Yellowish brown clayey SAND	SC
▲	B-117	30.0	22	77	Brown clayey SAND with gravel	SC
★	B-118	8.5	4	55	Brown well-graded SAND with gravel	SW
⊙	B-118	26.0	40		yellowish brown clayey SAND	SC

FUGRO

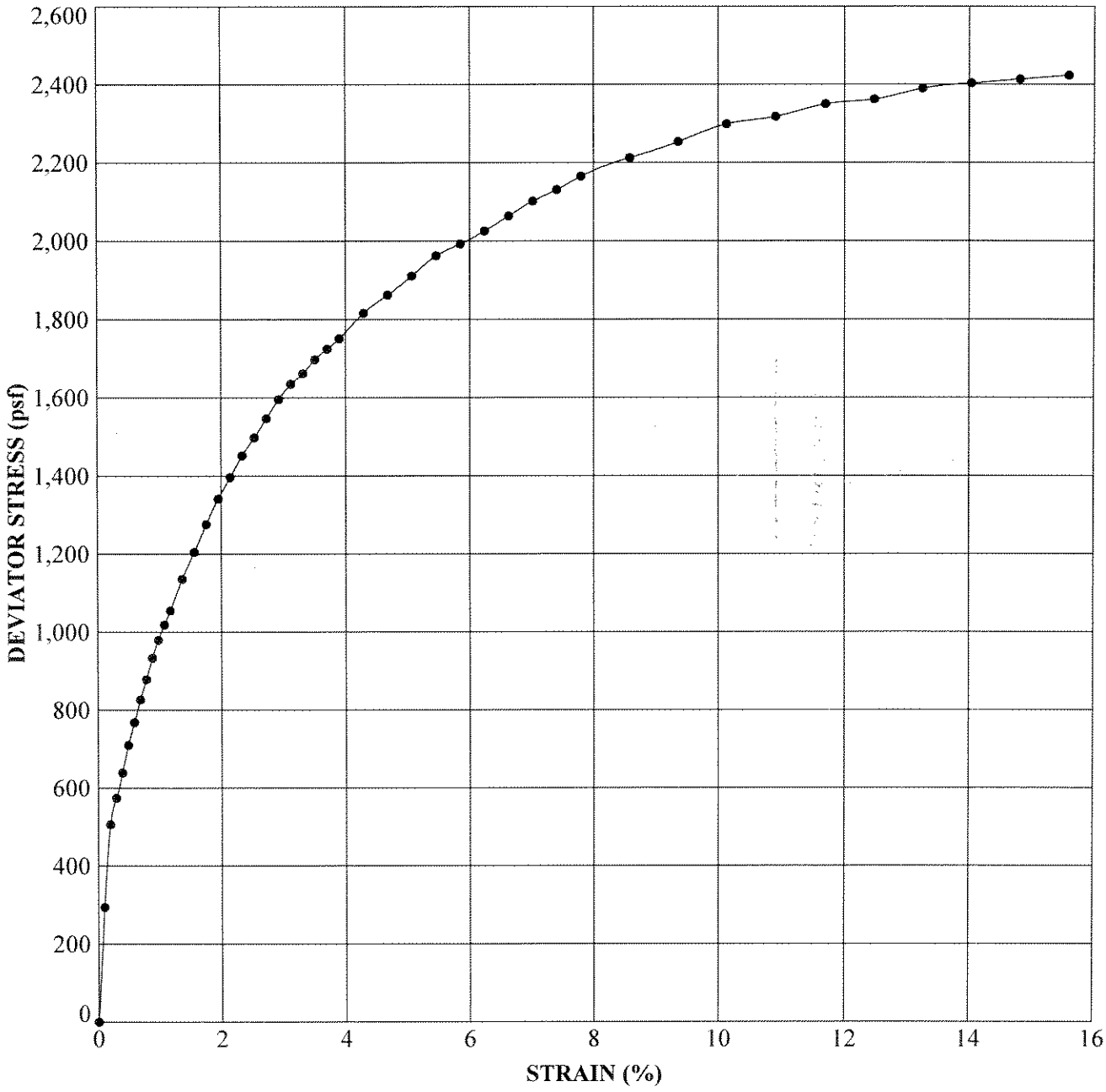
PREP'D BY: RV
APP'D BY: M. Walker
DATE: 8/21/08
DWG FILE:

GRADATION TEST DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-2b
PROJECT No.
1595.002

GRADATION_B_1595.002.GPJ_STD.GDT_8/21/08



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Peak Deviator Stress (psf)	Strain (%)
●	B-116	5.0	Dark yellowish brown sandy lean CLAY (CL)	110.5	17.6	2,423	15.0

FUGRO

PREP'D BY	RV
APP'D BY	M. Walker
DATE	8/21/08
DWG FILE	

TRIAXIAL COMPRESSION TEST DATA

PHASE-1 NEW HOSPITAL REPLACEMENT PROJECT
CHILDREN'S HOSPITAL AND RESEARCH CENTER
Oakland, California

FIGURE
B-3
PROJECT No.
1595.002

TRIAXIAL 1595.002.GPJ STD.GDT 8/21/08

APPENDIX C
SEISMIC HAZARD ANALYSIS AND GROUND MOTIONS

APPENDIX C
SEISMIC HAZARD ANALYSIS AND SITE-SPECIFIC GROUND MOTIONS

1.0 INTRODUCTION

This appendix documents the seismic hazard evaluation study consisting of probabilistic seismic hazard analysis (PSHA) and deterministic analysis conducted to update the design ground motion criteria per the 2007 California Building Code (2007 CBC) code for New Hospital Replacement Project – New Patient Tower for Children's Hospital & Research Center at Oakland (CHRCO). The CHRCO site is located at 52nd Street in Oakland, Alameda County, California.

The geotechnical study conducted by Harza Kaldveer (now Fugro) in 1994 included a seismic hazard evaluation. Since January 2008, California has adopted the 2007 CBC; therefore our seismic hazard evaluation study as summarized in this appendix has been revised to meet the requirements of the 2007 CBC.

Site-specific PSHA and deterministic seismic hazard analysis (DSHA) have been conducted in accordance with the 2007 CBC requirements to develop design ground motion criteria for a Maximum Considered Earthquake (MCE) and the calculation from the MCE of a design response spectrum.

The scope of work for this aspect of the project consisted of:

- Characterizing the subsurface conditions,;
- Identifying potential seismic sources and fault parameters;
- Performing PSHA to generate acceleration response spectra at the ground surface for horizontal component of motion;
- Comparing the PSHA and DSHA results with deterministic limit and code-based simplified spectra to develop the site-specific design spectrum for the project according to the 2007 CBC; and,
- Incorporating the provisions of the Office of Statewide Health Planning and Development (OSHPD) Code Application Notices (CANs) numbered 2-1802A.6.2, effective September 30, 2008; and 2-1614A.1.2, effective January 26, 2009.

2.0 SUBSURFACE CONDITIONS

The soils encountered across the site in our borings typically consisted of a layer of lean clay fill, over interlayered clayey sands to sandy clays with varying gravel content. The clay fill was observed to a typical depth of about 5 feet. Stiff lean clay was generally encountered between depths of 5 and 20 feet. Below this stiff clay, interlayered clayey sands and sandy clays, as well as some gravel layers, were observed to depths ranging from 24 to 46 feet. The previous deep boring, PB-1 (74), shows a lean clay layer below 50 feet to a depth of 150 feet with two clayey sand layers encountered from 102 to108.5 feet and 118 to 121 feet. The

subsurface conditions at the site of each proposed structure are presented in Section 4.3 of this report. The boring logs are presented in Appendix A. In general, the subsurface conditions across the site are fairly similar and consist of fill underlain by alluvial deposits.

Idealized Soil Profile

Idealized soil profile, design unit weight, and shear strength parameters were developed based on the results of the field investigations and laboratory testing conducted by Fugro. The profile is based on the field and laboratory test data from twelve exploratory borings conducted for this investigation.

We have assumed a V_{s30} value of 270 m/s for the site based on published correlations between shear wave velocity and SPT N-values (Ohta & Goto, 1976, Imai and Tonouchi, 1982). This is consistent with published values of V_{s30} for Quaternary alluvium in the San Francisco Bay Area (Wills and Clahan, 2006).

Based on the idealized shear wave velocities, the entire project site is considered to be Site Class "D" per CBC 2007. We note that this takes into consideration the potentially liquefiable zones encountered in borings B-112 (08) and B-113. Based on our experience, the thin and discontinuous soil deposits have no impact on overall site response analysis. For this site, the stiff clays and dense sands at this site will govern the seismic site response. We believe that the Site Class D is appropriate for design because the majority of the soil at these sites is stiff clay and dense sand material.

3.0 METHODOLOGY FOR DEVELOPMENT OF DESIGN RESPONSE SPECTRA

The procedure for development of a design response spectrum for this site consisted of the following four steps:

1. Conducting probabilistic seismic hazard analysis (PSHA) using the Next Generation Attenuation (NGA) relationships and in accordance with the 2007 CBC requirements.
2. Applying the factors required by the OSHPD CAN 2-1802A.6.2 to account for maximum rotated motion. Rotation factors used can be found in Campbell and Borzorgnia (2007) report, Table 4.3, Median of Maximum Rotated Component ratios.
3. Conducting deterministic seismic hazard analysis (DSHA) to determine the largest mean plus one sigma (M+σ) acceleration response spectrum from characteristic earthquakes on all known active faults in the region.
4. PSHA and DSHA results were used to develop design ground motion criteria for a Maximum Considered Earthquake (MCE) and a Design Response Spectrum according to 2007 CBC that is applicable at the foundation level. Design response spectrum ordinates are presented in accordance with ASCE 7-05 §21.2 and OSHPD CAN 2-1614A.1.2.

4.0 PROBABILISTIC SEISMIC HAZARD ANALYSES

Introduction

Probabilistic seismic hazard analyses were performed to evaluate the probability of occurrence of ground motion corresponding to the 2475-year return period event in order to develop the Maximum Considered Earthquake (MCE) per CBC 2007. The spectral analyses consider site-specific subsurface conditions and the regional seismic setting.

Tectonic/Seismic Setting

The Oakland Children's Hospital is located in the Coast Ranges geomorphic province of California, which is characterized by northwest-southeast trending valleys and ridges. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates and subsequent strike-slip faulting along the San Andreas Fault zone. The potential for surface fault rupture at the site is low. The site lies approximately 2 miles from the closest active fault zone (Hayward fault).

Identification and Characterization of Potential Seismogenic Sources

A potential seismogenic source is one that exhibits historic displacement, historic earthquake activity, or geologic evidence of activity during the latest Holocene (approximately the past 11,000 years). All significant seismogenic sources within 100 km of the site were identified and considered in our analysis. The key geometric, geologic and seismologic characteristics for each potential seismogenic source include:

- Style of Faulting (e.g. strike-slip, reverse, normal);
- Closest source-to-site distance;
- Total fault length and rupture length;
- Rupture width;
- Slip rate; and
- Maximum magnitude (M_{max}).

Table C-1 lists the key fault parameters for the sources found within a 25 km radius of the site. These parameters were subsequently used in the seismic hazard analyses and were adapted from the Working Group on California Earthquake Probabilities (WGCEP, 2003).

Table C-1: Geologic Characteristics of Significant Seismogenic Sources within 25 km of the Site

Potential Seismogenic Source[1]	Type[2,3]	Closest Distance in km (miles)	Slip Rate (mm/y)[5]	M_{max}[4,5]
Hayward Rogers Creek (HS)	RL-SS	3.3 (2.1)	9.0	6.67
Hayward Rogers Creek (HN)	RL-SS	3.3 (2.1)	9.0	6.49
Hayward Rogers Creek (HS+HN)	RL-SS	3.3 (2.1)	9.0	6.91
Hayward Rogers Creek (HN+RC)	RL-SS	3.3 (2.1)	9.0	7.11
Hayward Rogers Creek (HS+HN+RC)	RL-SS	3.3 (2.1)	9.0	7.26
Hayward Rogers Creek (FLOATING)	RL-SS	3.3 (2.1)	9.0	6.90
Background	-	-	Activity Rate (M>5.0) 0.45 events/yr	7.25
Calaveras (CN)	RL-SS	20.6 (12.8)	6.0	6.78
Calaveras (CC+CN)	RL-SS	20.6 (12.8)	6.0	6.90
Calaveras (CS+CC+CN)	RL-SS	20.7 (12.9)	6.0	6.93
Calaveras (Floating)	RL-SS	20.7 (12.9)	6.0	6.20
Mt Diablo (MTD)	RL-SS	25.2 (15.7)	2.0	6.65
Concord (CON)	RL-SS	25.2 (15.7)	4.0	6.25
Concord (CON+GVS)	RL-SS	25.2 (15.7)	5.0	6.58
Concord (CON+GVS+GVN)	RL-SS	25.2 (15.7)	5.0	6.71
Concord (FLOATING)	RL-SS	25.2 (15.7)	5.0	6.20
San Andreas (SAP)	RL-SS	25.7 (16.0)	17.0	7.15
San Andreas (SAS+SAP)	RL-SS	25.7 (16.0)	17.0	7.42
San Andreas (SAN+SAP+SAS)	RL-SS	25.7 (16.0)	24.0	7.76
San Andreas (SAP+SAN+SAO)	RL-SS	25.7 (16.0)	24.0	7.83
San Andreas (SAS+SAP+SAN+SAO)	RL-SS	25.7 (16.0)	24.0	7.90
San Andreas (FLOATING)	RL-SS	25.7 (16.0)	24.0	6.90

Notes:
1. Various fault segments and rupture scenarios were considered as described in WGCEP (2003)
2. Fault parameters were adapted from the CGS fault database (2003) and WGCEP (2003)
3. SS = Strike Slip Fault, RL = Right Lateral, R = Reverse (Dip in degrees), O = Oblique
4. M_{max}= Maximum moment magnitude with highest weightage in the seismotectonic model (per WGCEP 2003)
5. Only mean slip rate and M_{max} values are listed here. WGCEP (2003) provides upper- and lower-bound values that were also used with appropriate weighting factors. Those are not listed here for brevity.

Seismic Hazard Analysis Methodology

Probabilistic seismic hazard analyses (PSHA) were carried out using the computer program HAZ38 (Abrahamson, 2007). The program employs the analytical procedure to compute seismic hazard originally developed by Cornell (1968).

The activity of the seismic sources was modeled using the Youngs and Coppersmith (1985) composite model to simulate the relative distribution of magnitudes for each

fault. In implementing the model in the hazard code, it was considered that for each fault the mean characteristic magnitude (M_{char}) is the maximum magnitude (M_{max}) tabulated in Table C-1. The composite model includes a characteristic earthquake distribution for the large magnitude earthquakes ($M > M_{char} - 0.25$) and an exponential distribution for smaller magnitude earthquakes ($M \leq M_{char} - 0.25$). The slope of the exponential part (i.e. b-value) was assigned a value of 0.8 based on regional seismicity data.

The probability density function of the Youngs and Coppersmith model is given by:

$$f_m^{YC}(M) = \begin{cases} \frac{1}{1+c_2} \cdot \frac{\beta \cdot \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 1.25)\right)}{1 - \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)} & for \quad \overline{M}_{char} - 0.25 \leq M \leq \overline{M}_{char} + 0.25 \\ \frac{1}{1+c_2} \cdot \frac{\beta \cdot \exp\left(-\beta \cdot (M - M_{\min})\right)}{1 - \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)} & for \quad M_{\min} \leq M \leq \overline{M}_{char} - 0.25 \end{cases}$$

where

$$c_2 = \frac{0.5 \cdot \beta \cdot \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 1.25)\right)}{1 - \exp\left(-\beta \cdot (\overline{M}_{char} - M_{\min} - 0.25)\right)},$$

$\beta = b \cdot \ln(10)$, and

$\overline{M}_{char}$ is the mean characteristic magnitude of the fault here taken equal to M_{max}.

The seismic hazard analyses were conducted using the recently developed Next Generation of Attenuation (NGA) Relationships developed for shallow crustal earthquakes and considered the influence of near-source and rupture directivity effects. These relationships used were developed as part of the PEER NGA program (http://peer.berkeley.edu/). These are:

- Abrahamson and Silva (2008);
- Boore and Atkinson (2008);
- Chiou and Youngs (2008); and,
- Campbell and Bozorgnia (2008).

PSHA Results

Figure C-1 shows the horizontal component conducted for the idealized site average shear wave velocity of 270 m/s. The motion presented is adjusted for motion directivity by applying the factors presented in Campbell and Borzorgnia (2007). The resulting spectrum used for 2007 CBC comparison (not for use in design of hospital structures) is presented in Table C-2.

Table C-2: Probabilistic MCE Spectrum

Period (seconds)	Spectral Acceleration (g)
0.01	1.077
0.10	1.923
0.20	2.400
0.30	2.486
0.40	2.498
0.50	2.408
0.60	2.222
0.75	2.005
1.0	1.690
1.5	1.293
2.0	1.027
3.0	0.670
4.0	0.486

Deaggregation of Hazard

Deaggregation of the hazard shows that a majority of the hazard comes from events with magnitudes ranging between 6.5 and 7.5 with distances between 0 and 5 km. This hazard is associated with events on the Hayward-Rodgers Creek fault that lies approximately 3.3 km from the site.

5.0 DETERMINISTIC HAZARD ANALYSIS

The M+σ deterministic spectrum was obtained from an earthquake occurring approximately 3.3 km from the project site with a magnitude of 7.26, which is the maximum M_{max} value for the rupture of the entire Hayward-Rodgers Creek fault. The M+σ deterministic spectrum has then been increased per the OSHPD Code Application Notice directive by the factors presented in Campbell and Borzorgnia (2007) for median maximum rotated spectrum. The resulting spectrum is shown on Figure C-2 and tabulated below.

Table C-3: M = 7.26, r = 3.3km Deterministic Spectrum Rotation Calculation

Period (seconds)	Spectral Acceleration (g)
0.01	0.836
0.10	1.375
0.20	1.823
0.30	1.899
0.40	1.894
0.50	1.815
0.60	1.719
0.75	1.563
1.0	1.339
1.5	1.034
2.0	0.838
3.0	0.570
4.0	0.431

6.0 DEVELOPMENT OF SPECTRA PER CBC 2007

Site-specific ground motion procedures were undertaken to develop the design spectrum in accordance with CBC 2007, Chapter 21 of ASCE 7-05, and OSHPD CAN 2-1802A.6.2. ASCE 7-05 § 21.2.3 requires the calculation of the response spectrum for a Site-Specific MCE and the design spectrum is calculated as two-thirds of the Site-Specific MCE spectrum.

The Site-Specific MCE is defined probabilistically and deterministically as the lesser of:

1. the 2 percent probability of exceedance in 50 year (i.e. 2475 year return period MCE) probabilistic ground motion calculated with 5 percent damping; and,
2. The greater of:
    a. M+σ deterministic ground motion calculated for 5 percent damping, and,
    b. A deterministic lower bound spectrum calculated according to ASCE 7-05 §21.2.2.

The comparison of the MCEs using these spectra and the procedures mentioned in ASCE 7-05 §21.2 is shown on Figure C-3.

DESIGN RESPONSE SPECTRUM

The Site-Specific MCE spectrum described in this report takes into account the variation of near-surface stratigraphy at the project site. Therefore, the spectrum is applicable directly at the foundation level. The Design Response Spectrum defined in ASCE 7-05 §21.3 is calculated from the following comparisons of probabilistic MCE and deterministic MCE:

The greater of:

1. Two-thirds of the site-specific MCE calculated above, and,
2. 80 percent of the design response section calculated according to ASCE 7-05 §11.4.

This comparison and the final Design Response Spectrum are shown graphically on Figure C-4.

Per ASCE 7-05 §21.4 and the interpretation of OSHPD CAN No. 2-1614A.1.2, the parameters S_{DS} and S_{D1} can be obtained from the site-specific design response spectrum. The S_{D1} value is taken as the spectral acceleration at 1 second period. The S_{DS} value is calculated as the larger of: (1) spectra acceleration at 0.2 second, and (2) 90 percent of the largest spectral acceleration for any period greater than 0.2 second. Based on these criteria, the design parameters for the project are:

S_{DS} = 1.178 and S_{D1} = 0.968.

The calculated spectra are presented in the following table.

Table C-4: Horizontal Response Spectra

Period (seconds)	Site-Specific MCE Spectrum (g)	Design Response Spectrum (g)
0.01	1.140	0.760
0.10	1.500	1.000
0.20	1.767	1.178
0.30	1.888	1.259
0.40	1.932	1.288
0.50	1.927	1.285
0.60	1.826	1.217
0.75	1.689	1.126
1.0	1.453	0.968
1.5	1.085	0.723
2.0	0.855	0.570
3.0	0.556	0.371
4.0	0.410	0.273

7.0 SEISMIC DESIGN CATEGORY

From the mapped values presented in ASCE 7-05 and the 2007 CBC, the site S_S value is 1.73 and the S_1 value is 0.64. Our site-specific study indicates that the S_{DS} value is 1.178 and the S_{D1} value is 0.968. The 2007 CBC §1613A.5.6 indicates that the Seismic Design Category for Occupancy Category I, II, III, and IV structures is "D".

8.0 REFERENCES

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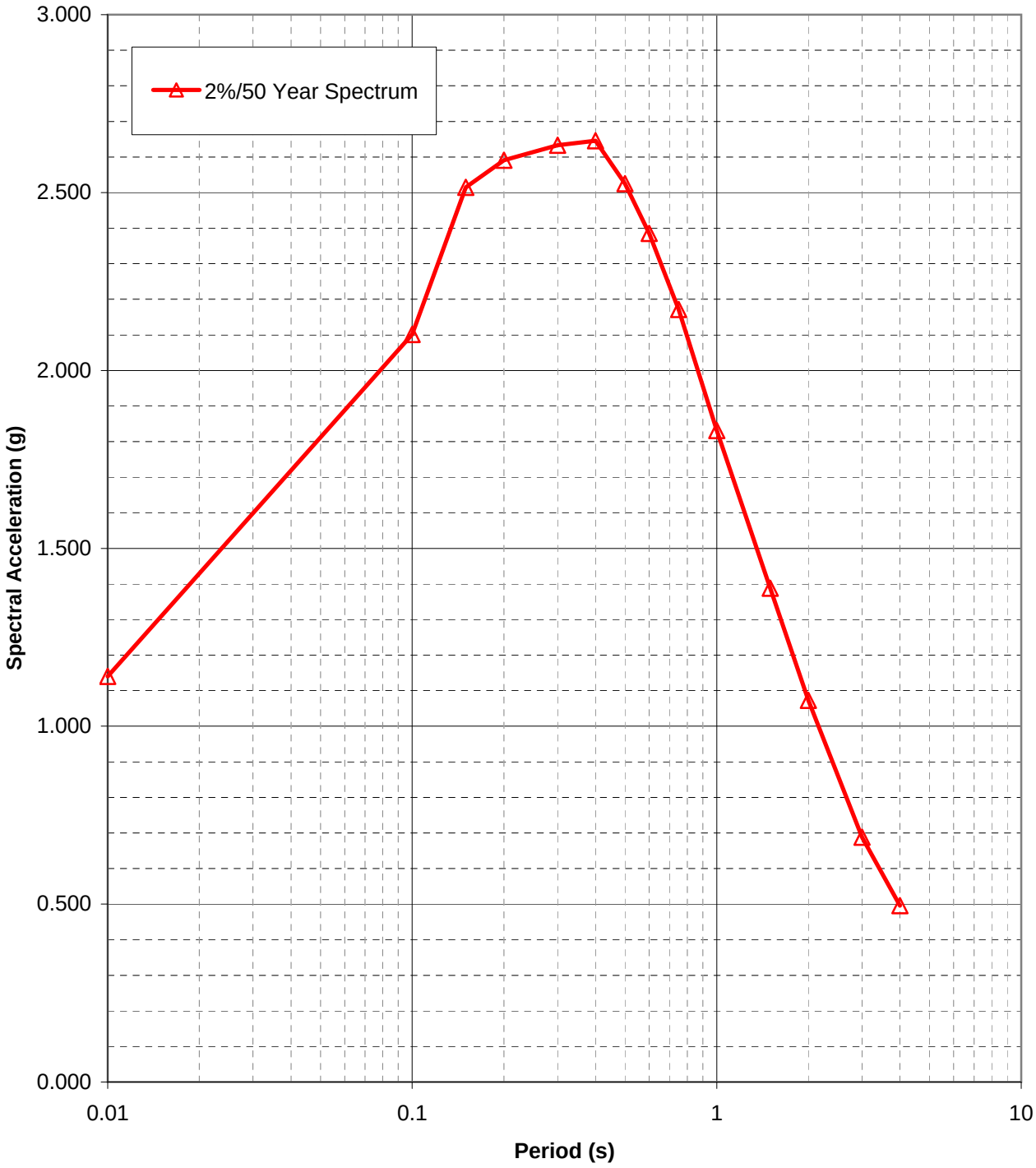
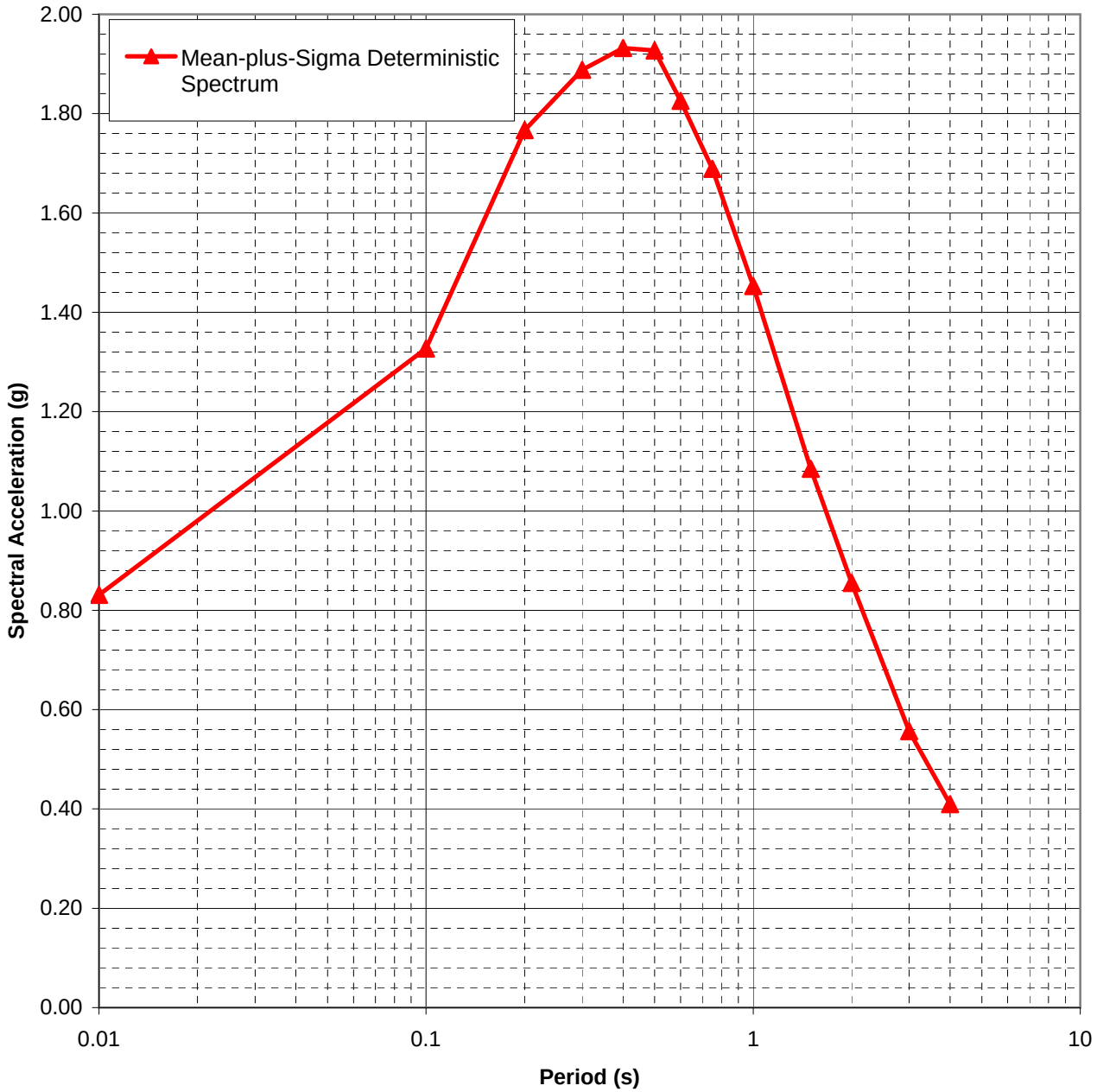


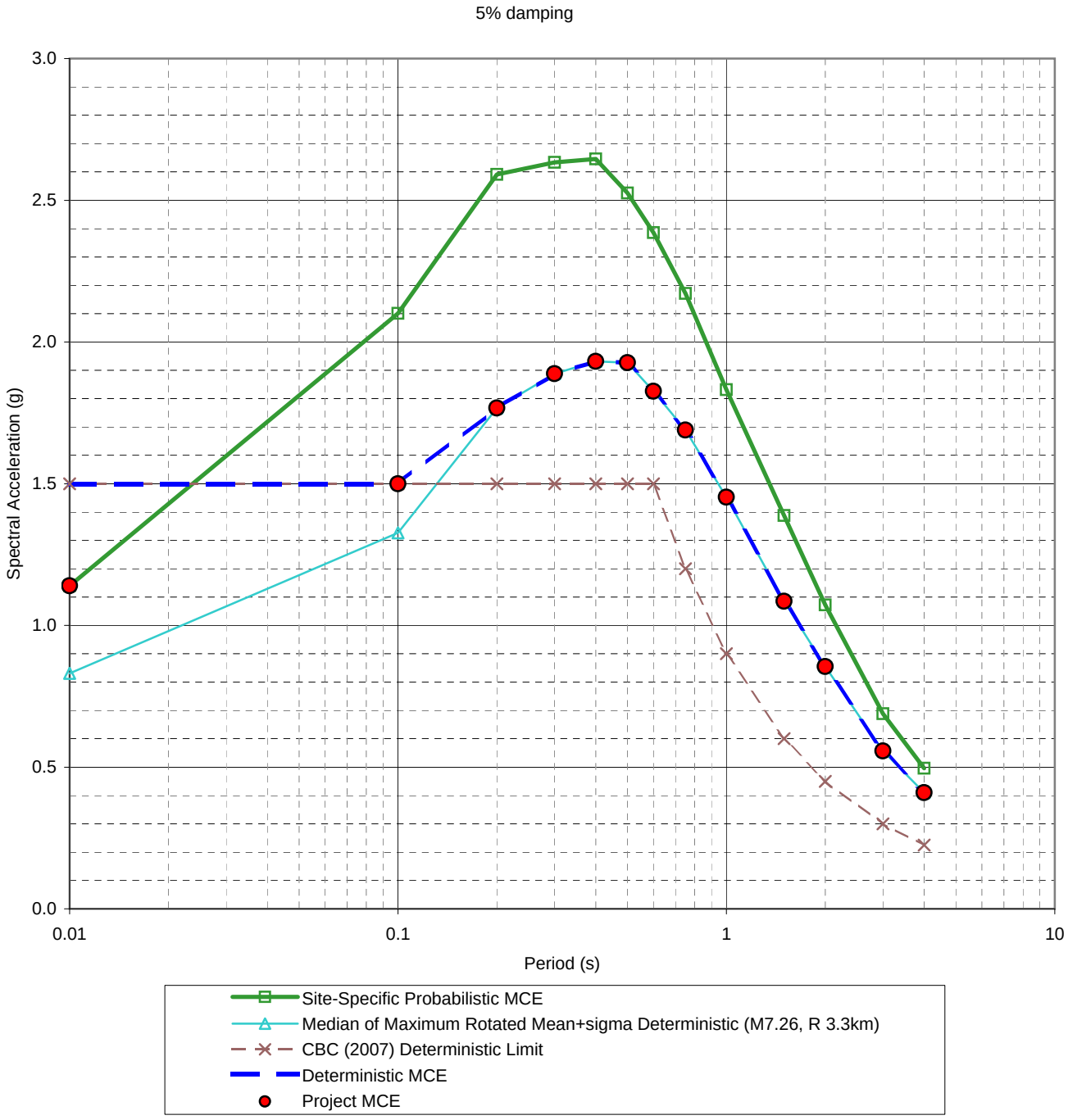
PROBABILISTIC MCE
New Hospital Replacement Project - Patient Tower
Children's Hospital and Research Center
Oakland, California

FIGURE C-1



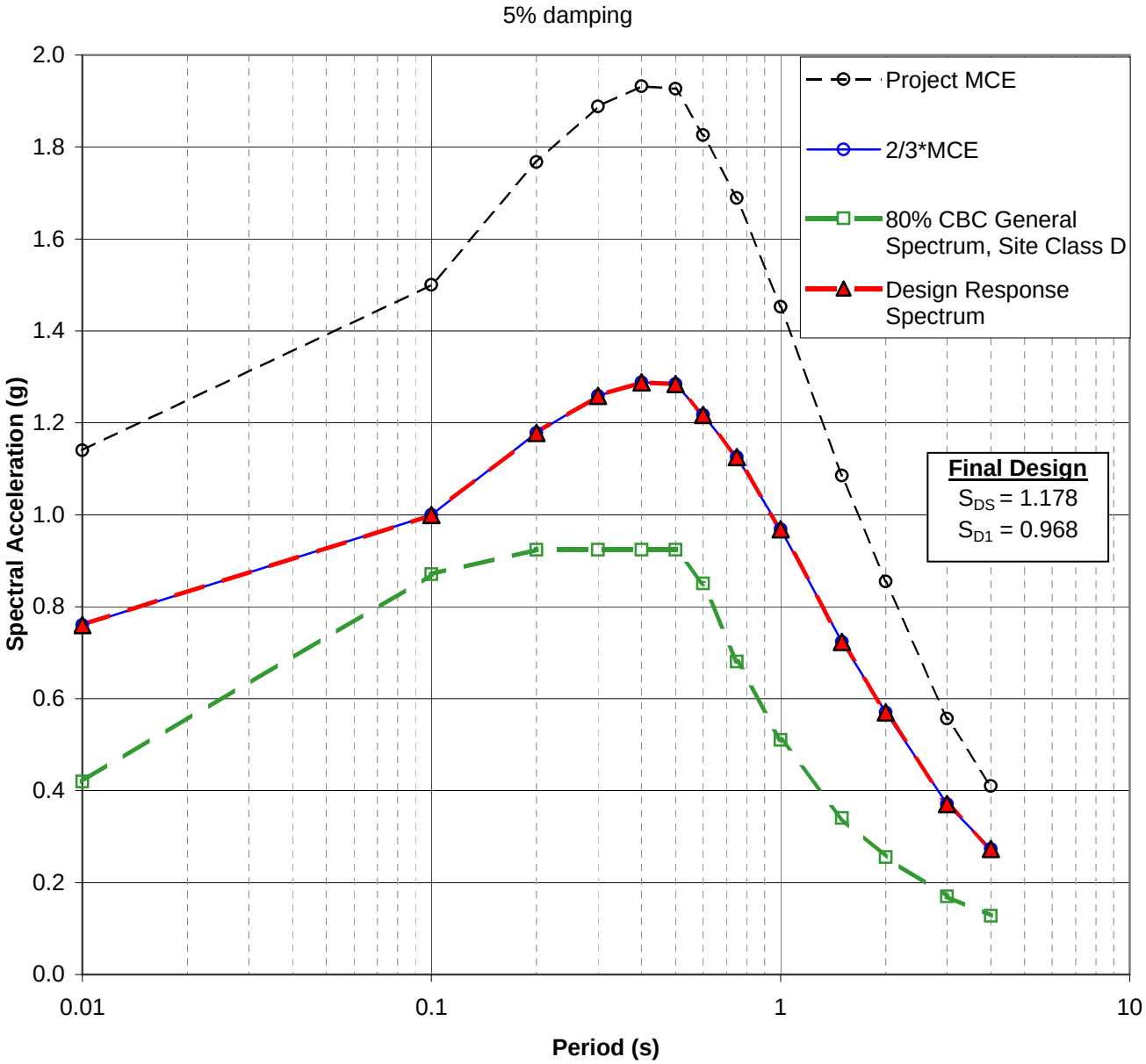
HORIZONTAL DETERMINISTIC RESPONSE SPECTRUM
New Hospital Replacement Project - Patient Tower
Children's Hospital and Research Center
Oakland, California

FIGURE C-2



HORIZONTAL MCE SPECTRA BASED ON 2007 CBC/ASCE 7-05
New Hospital Replacement Project
Children's Hospital and Research Center - Patient Tower
Oakland, California

FIGURE C-3



HORIZONTAL DESIGN RESPONSE SPECTRUM BASED ON 2007 CBC/ASCE 7-05
New Hospital Replacement Project
Children's Hospital and Research Center - Patient Tower
Oakland, California

FIGURE C-4

APPENDIX D
LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

APPENDIX D
LIQUEFACTION AND DYNAMIC DENSIFICATION ANALYSES

INTRODUCTION

This appendix presents a summary of the procedures used to evaluate the potential for liquefaction of soil at the site. The liquefaction evaluations at the site involved the following steps:

1. Evaluating the generalized soil descriptions and laboratory index test results to identify layers that may be potentially liquefiable;
2. Estimating the seismically induced cyclic demands on the potentially liquefiable soil layers;
3. Estimating the liquefaction resistance of the potentially liquefiable soil layers from boring data;
4. Calculating the factor of safety against liquefaction for the potentially liquefiable layers; and
5. Estimating the magnitude of liquefaction induced settlement in layers with a factor of safety less than 1.1

EVALUATION OF POTENTIALLY LIQUEFIABLE SOIL LAYERS

The susceptibility of a granular soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in: a) distribution of grain size, b) soil density, c) aging and cementation, d) fines content, and e) plasticity characteristics of the fines.

Potentially liquefiable layers are identified as relatively loose, submerged, granular sediments. Liquefaction potential is believed to decrease with depth, and current procedures are believed to be overly conservative for depths greater than 20 meters (Davis and Berrill, 1982). Furthermore, aging tends to increase the liquefaction resistance of soil deposits, and Pleistocene soils are considered to have an extremely low potential for liquefaction.

Cohesive materials may also have the potential to liquefy. Generally, if the liquid limit of the cohesive materials is greater than 35 or the soil meets one of the other criteria specified in Seed, et al. (2001), the cohesive layer is not considered to be liquefiable.

Evaluation of the liquefaction susceptibility of coarse-grained layers and deposits was based on the empirical procedure recommended by the National Center for Earthquake Engineering Research (NCEER, 1997) and summarized by Youd et. al. (2001). In the NCEER procedure, potentially liquefiable soil strata are identified as those layers that are relatively loose, submerged granular sediments. Soil strata above the assumed groundwater level were considered not susceptible to liquefaction. A design groundwater depth of 8 feet was used in our analyses.

NCEER recommendations were used to compute liquefaction susceptibility using boring SPT data. The available cyclic shear resistance of the potentially liquefiable strata encountered (based on the normalized SPT N-value) was compared to the estimated cyclic shear stress that would be induced by a given earthquake. The estimated factor of safety against liquefaction is the ratio of the available cyclic shear resistance to the induced cyclic shear stress.

The estimated factors of safety against liquefaction at various depths are a function of the estimated overburden pressure at that depth. In general, higher factors of safety against liquefaction are calculated for higher overburden pressures.

For use with the NCEER procedure, sample depths, fines contents, soil unit weights and field SPT N-values were interpreted and summarized from the borings. The SPT data were then normalized to an overburden pressure of 1 atmosphere and corrected for fines content. A hammer efficiency factor of 1.3 was used to correct the field SPT N-values because the SPT sampling procedure used an automatic trip hammer with a higher efficiency than a standard rope and cathead.

For this evaluation, sublayers were identified at each boring location. Typically, the overall stratigraphic description was obtained from the boring logs, and sublayers were assigned based on SPT blow counts and fines contents. Usually, one sublayer was created for each sample obtained in the boring and that layer was assigned the blow count associated with the sample. A fines content values was assigned to each of those sublayers on the basis of either laboratory tests performed on the sample or the nearest fines content measurement considered representative of the sublayer. Unless bounded by one of the stratigraphic layers shown on the boring logs or the estimated groundwater table, sublayer boundaries were selected midway between adjacent samples.

The seismic hazard analyses for the project (Appendix C) have defined the following design response spectrum values:

- A magnitude 7.26 event on the Hayward fault
- A peak horizontal ground acceleration (PGA) of 0.76g

SUMMARY OF RESULTS AND CONCLUSIONS

The subsurface soils encountered in the borings consist primarily of fine-grained sediment (clay and sandy clay), and have a low susceptibility to liquefaction and dynamic densification. The clayey sand layers contain a significant amount of fines and are typically dense with corrected SPT N-values, (N1)60, greater than 30. These clayey sand layers are unlikely to liquefy during a design seismic event, based on our analyses. However, the liquefaction analyses did identify two isolated, thin layers (Borings B-112 and B-113) that may be susceptible to liquefaction and liquefaction-induced settlement during the design earthquake. The calculation spreadsheets for estimating the liquefaction potential based on the borings are attached in this appendix.

The potential for significant liquefaction and liquefaction-induced settlements at the site is considered low based on the evaluation of the boring data. The underlying soils are predominantly fine-grained and that the identified potentially liquefiable layers: 1) are anticipated to experience relatively minor strains (1 inch or less), 2) do not appear to be laterally extensive layers, and 3) are limited in thickness.

GROUND SETTLEMENT FROM EARTHQUAKE SHAKING

Ground settlement (and other ground effects) during seismic shaking can be caused by liquefaction, lateral spreading and compression of loose fill with voids, rubble, animal burrows or other similar conditions. Settlement from liquefaction or in loose, dry fill can be estimated using empirical procedures. These estimates are based on level ground conditions and are described below. Earthquake-induced settlement can cause downdrag on deep foundations, distress to pavements, gaps between ground supported pavements/structures and deep supported structures and damage to structures founded on shallow foundations.

Liquefaction Induced Settlement

Liquefaction-induced settlements were estimated using the procedure of Tokimatsu and Seed (1984). Using this procedure, volumetric strains were estimated that could occur within submerged layers from dissipation of seismically induced pore pressures. The estimated volumetric strains were then used in conjunction with the corresponding layer thicknesses to evaluate the magnitude of seismically induced settlements at the boring locations.

Seismically Induced Settlement of Dry Fill

Earthquake shaking can also result in seismically induced settlement of relatively loose, dry, granular materials. However, the upper fill is generally cohesive and stiff and therefore settlement of the dry fill is only expected to occur in areas where voids exist.

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-112
Date:	9/23/2008
Calculated By:	SC

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	4
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	100
Design Ground Water Elev. (feet)	92

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY (CL	0	125	12	80	22	N/A	0.0
2	Lean CLAY (CL)	5	125	9	90	20	N/A	0.0
3	Lean CLAY (CL)	8	125	14	90	20	N/A	0.0
4	Clayey SAND (SC)	14	125	18	14		0.59	0.5
5	Clayey SAND (SC)	17	125	17	15		0.46	0.8
6	Sandy CLAY (CL)	22	125	13	66	20	N/A	0.0
7	Silty SAND (SM)	28	125	36	15		N/A	0.0
8	SAND with Silt (SP-SM)	33	125	32	10		N/A	0.0
9	SAND with Silt (SP-SM)	40	125	32	10		N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	1.3

TOTAL SETTLEMENT (inches)	1.3

Table D-1: Liquefaction Analysis for B-112

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY (CL	0.0	5.0	2.5	125.0	12	80	313	0	313	1.70	1.3	1.00	0.75	1.00	20	5.00	1.20	29	0.40	1.21	0.99	0.49	1.15	N/A
2	Lean CLAY (CL)	5.0	3.0	6.5	125.0	9	90	813	0	813	1.57	1.3	1.00	0.75	1.00	14	5.00	1.20	22	0.24	0.71	0.98	0.49	1.15	N/A
3	Lean CLAY (CL)	8.0	6.0	11.0	125.0	14	90	1375	187	1188	1.30	1.3	1.00	0.75	1.00	18	5.00	1.20	26	0.32	0.96	0.97	0.56	1.15	N/A
4	Clayey SAND (SC)	14.0	3.0	15.5	125.0	18	14	1938	468	1470	1.17	1.3	1.00	0.85	1.00	23	2.20	1.04	26	0.32	0.32	0.96	0.63	1.15	0.59
5	Clayey SAND (SC)	17.0	5.0	19.5	125.0	17	15	2438	718	1720	1.08	1.3	1.00	0.85	1.00	20	2.50	1.05	24	0.27	0.27	0.95	0.67	1.15	0.46
6	Sandy CLAY (CL)	22.0	6.0	25.0	125.0	13	66	3125	1061	2064	0.98	1.3	1.00	0.95	1.00	16	5.00	1.20	24	0.27	0.82	0.94	0.70	1.15	N/A
7	Silty SAND (SM)	28.0	5.0	30.5	125.0	36	15	3813	1404	2409	0.91	1.3	1.00	0.95	1.00	41	2.50	1.05	45	N/A	N/A	0.93	0.72	1.15	N/A
8	AND with Silt □(SP-S	33.0	7.0	36.5	125.0	32	10	4563	1778	2784	0.85	1.3	1.00	1.00	1.00	35	0.87	1.02	37	N/A	N/A	0.88	0.71	1.15	N/A
9	AND with Silt □(SP-S	40.0	N/A	N/A	125.0	32	10	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A

NOTES:

*1	Unit Weight of Water is assumed to be			62.4 (pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.00
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7 $(N1)60cs = a + b(N1)60$

	a	b
FC ≤ 5	a = 0	b = 1.0
5 < FC < 35	a = e^(1.76 - (190/FC^2))	b = (0.99 + (FC^1.5/1000))
FC ≥ 35	a = 5.0	b = 1.2

*8

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$$

*9 If PI of Fines>15, Increase CRR by 300%

*10 rd = 1.0 - 0.00765z for z ≤ 9.15m; 1.174 - 0.0267z for z > 9.15m

*11 Magnitude Scaling Factor (NCEER, 1997, mean values)

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table D-2: Liquefaction Analysis for B-112

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-113
Date:	9/23/2008
Calculated By:	SC

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	8
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	95
Design Ground Water Elev. (feet)	87

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY (CL)	0	107	15	90	10	N/A	0.0
2	Lean CLAY with Sand (CL)	5	107	13	60	10	N/A	0.0
3	Lean CLAY with Sand (CL)	5	107	13	90		N/A	0.0
4	Clayey SAND (SC)	20	125	12	35		0.57	0.9
5	Lean CLAY (CL)	26	125	10	90	10	N/A	0.0
6	Sandy Lean CLAY with Gravel (CL)	35	125	34	50	10	N/A	N/A
7							N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.9

TOTAL SETTLEMENT (inches)	0.9

Table D-3: Liquefaction Analysis for B-113

Layer	Soil Type Clay or Sand	Depth to Top of Layer (feet)	Layer Thickness (feet)	Depth to Middle of Layer (feet)	Unit Weight (pcf)	Nfield (SPT) Average Value for Layer	Percent Passing #200 Sieve	Total Overburden Pressure at Middle of Layer (psf)	Pore Pressure at Middle of Layer (psf) *1	Effective Stress at Middle of Layer (psf)	Cn *2	Ce *3	Cb *4	Cr *5	Cs *6	(N1)60	a *7	b *7	(N1)60cs *7	CRR(7.5) *8	Corrected CRR(7.5) *9	rd *10	CSR	Magnitude Scaling Factor *11	FS = CRR(7.5) / CSRxMSF
1	Fill - Lean CLAY (CL	0.0	5.0	2.5	107.0	15	90	268	0	268	1.70	1.3	1.15	0.75	1.00	29	5.00	1.20	39	N/A	N/A	0.99	0.49	1.15	N/A
2	an CLAY with Sand (	5.0	0.0	5.0	107.0	13	60	535	0	535	1.70	1.3	1.15	0.75	1.00	25	5.00	1.20	35	N/A	N/A	0.99	0.49	1.15	N/A
3	an CLAY with Sand (	5.0	15.0	12.5	107.0	13	90	1338	0	1338	1.22	1.3	1.15	0.75	1.00	18	5.00	1.20	26	0.32	0.32	0.97	0.48	1.15	N/A
4	Clayey SAND (SC)	20.0	6.0	23.0	125.0	12	35	2515	936	1579	1.13	1.3	1.15	0.95	1.00	19	5.00	1.20	28	0.37	0.37	0.95	0.74	1.15	0.57
5	Lean CLAY (CL)	26.0	9.0	30.5	125.0	10	90	3453	1404	2049	0.99	1.3	1.15	0.95	1.00	14	5.00	1.20	22	0.24	0.24	0.93	0.77	1.15	N/A
6	Lean CLAY with Gra	35.0	N/A	N/A	125.0	34	50	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
7	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
8	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
9	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
10	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
11	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
12	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
13	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
14	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
15	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
16	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
17	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
18	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
19	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A
20	0	0.0	N/A	N/A	0.0	0	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.15	N/A

NOTES:

*1	Unit Weight of Water is assumed to be		62.4	(pcf)
*2	Cn = (1/Effective stress (tsf))^1/2			
*3	Ce =	RC	1	Value used for calculation
		WT	1	1.3
		A	1.3	
*4	Cb =	0mm to 115mm	1	Value used for calculation
		115mm to 150mm	1 to 1.05	1.15
		150mm to 200mm	1.05 to 1.15	
		>200mm	1.15	
*5	Cr =	<4m	0.75	
		4m to 6m	0.85	
		6m to 10m	0.95	
		10m to 30m	1	
		>30m	<1	
*6	Cs =	Standard Sampler, or sampler with space for liners with liners	1	Value used for calculation
		With space for liners, but no liners	1.2	1.00

*7	(N1)60cs = a + b(N1)60	FC ≤ 5	a = 0
		5 < FC < 35	a = e^(1.76 - (190/FC^2))
		FC ≥ 35	a = 5.0
		FC ≤ 5	b = 1.0
		5 < FC < 35	b = (0.99 + (FC^1.5/1000))
		FC ≥ 35	b = 1.2
*8	$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \bullet (N_1)_{60} + 45]^2} - \frac{1}{200}$		
*9	If PI of Fines>15, Increase CRR by 300%		
*10	rd =	1.0 - 0.00765z	z ≤ 9.15m
		1.174 - 0.0267z	z > 9.15m
*11	Magnitude Scaling Factor (NCEER, 1997, mean values)		

Magnitude	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5
Factor	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.8	1.7	1.7	1.6
Magnitude	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	
Factor	1.5	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1.0	

Table D-4: Liquefaction Analysis for B-113

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-114
Date:	12/17/2008
Calculated By:	AJS

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	8
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	101.5
Design Ground Water Elev. (feet)	93.5

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY (CL)	0	125	12	70	22	N/A	0.0
2	Lean CLAY (CL)	6.5	125	24	90	26	N/A	0.0
3	Lean CLAY (CL)	8	125	16	90	19	N/A	0.0
4	Clayey SAND w/Silt (SC)	21	125	11	20		0.39	0.1
5	Clayey SAND w/Silt (SC)	21.5	125	11	20		N/A	N/A
6							N/A	N/A
7							N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.1

TOTAL SETTLEMENT (inches)	0.1

Table 1: Liquefaction Analysis for B-114

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY (CL	0.0	406	406	0.49	32	1.15	0.43		N/A	0.99	199
2	Lean CLAY (CL)	6.5	906	906	0.49	53	1.15	0.42		N/A	0.98	440
3	Lean CLAY (CL)	8.0	1813	1407	0.61	34	1.15	0.53		N/A	0.97	865
4	ayey SAND w/Silt (S	21.0	2656	1829	0.68	21	1.15	0.59	1.30	0.1	0.95	1247
5	ayey SAND w/Silt (S	21.5	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
6	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
7	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
8	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A

Design Earthquake M = 7.26

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY (CL	264	0.13	512	1.9E-04	N/A	N/A		1.0	0.0	0.000	0.00
2	Lean CLAY (CL)	589	0.29	900	2.4E-04	N/A	N/A		1.0	0.0	0.000	0.00
3	Lean CLAY (CL)	914	0.46	969	4.5E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	ayey SAND w/Silt (S	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
5	ayey SAND w/Silt (S	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
6	0	N/A	N/A	N/A	N/A	below water	#VALUE!	0	1.0	0.0	0.000	N/A
7	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-114

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-115
Date:	12/17/2008
Calculated By:	AJS

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	8
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	101.1
Design Ground Water Elev. (feet)	93.1

SUBSURFACE PROFILE								
Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY (CL)	0	125	10	90	22	N/A	0.0
2	Sandy Lean CLAY (CL)	3.5	125	24	70	26	N/A	0.0
3	Sandy Lean CLAY (CL)	8	125	16	70	19	N/A	0.0
4	Clayey SAND w/Gravel (SC)	20	125	18	25		N/A	0.0
5	Clayey SAND w/Gravel (SC)	26.5	125	18	25		N/A	N/A
6							N/A	N/A
7							N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-115

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY (CL	0.0	219	219	0.49	28	1.15	0.43		N/A	1.00	108
2	andy Lean CLAY (CL	3.5	719	719	0.49	59	1.15	0.42		N/A	0.99	350
3	andy Lean CLAY (CL	8.0	1750	1376	0.61	34	1.15	0.53		N/A	0.97	836
4	ey SAND w/Gravel (	20.0	2906	1955	0.69	33	1.15	0.60		N/A	0.95	1358
5	ey SAND w/Gravel (	26.5	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
6	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
7	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
8	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A

Design Earthquake M = 7.26

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY (CL	142	0.07	358	1.5E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	andy Lean CLAY (CL	467	0.23	829	2.1E-04	N/A	N/A		1.0	0.0	0.000	0.00
3	andy Lean CLAY (CL	894	0.45	961	4.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	ey SAND w/Gravel (	1271	0.64	1131	6.0E-04	N/A	N/A		1.0	0.0	0.000	0.00
5		N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
6	0	N/A	N/A	N/A	N/A	below water	#VALUE!	0	1.0	0.0	0.000	N/A
7	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-115

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-116
Date:	12/17/2008
Calculated By:	AJS

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	8
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	95
Design Ground Water Elev. (feet)	87

SUBSURFACE PROFILE

Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill - Lean CLAY (CL)	0	125	10	90	26	N/A	0.0
2	Clayey SAND/Sandy CLAY (CL/SC)	8	125	24	50	26	N/A	0.0
3	SILT (ML)	19	125	16	90	10	N/A	0.0
4	Lean CLAY w/ sand (CL)	23	125	11	85	22	N/A	0.0
5	Well-graded GRAVEL w/ clay and sand (GW-	30	125	51	15		N/A	0.0
6	Clayey SAND (SC)	38	125	39	15		N/A	0.0
7	Sandy Lean CLAY (CL)	42	125	31	70	22	N/A	0.0
8	Clayey SAND (SC)	48	125	43	15		N/A	0.0
9	Clayey SAND (SC)	50.5	125	43	15		N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-116

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	Fill - Lean CLAY (CL	0.0	500	500	0.49	28	1.15	0.43		N/A	0.99	245
2	SAND/Sandy CLAY	8.0	1688	1344	0.60	50	1.15	0.52		N/A	0.97	807
3	SILT (ML)	19.0	2625	1814	0.68	34	1.15	0.59		N/A	0.95	1233
4	ean CLAY w/ sand (C	23.0	3313	2158	0.71	23	1.15	0.62		N/A	0.94	1535
5	RAVEL w/ clay and s	30.0	4250	2628	0.72	72	1.15	0.62		N/A	0.90	1884
6	Clayey SAND (SC)	38.0	5000	3003	0.70	52	1.15	0.61		N/A	0.85	2096
7	andy Lean CLAY (CL	42.0	5625	3316	0.68	48	1.15	0.59		N/A	0.81	2245
8	Clayey SAND (SC)	48.0	6156	3582	0.66	53	1.15	0.57		N/A	0.77	2351
9	Clayey SAND (SC)	50.5	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A

Design Earthquake M = 7.26

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	Fill - Lean CLAY (CL	325	0.16	541	2.3E-04	0	0.0E+00		1.0	0.0	0.000	0.00
2	SAND/Sandy CLAY	874	0.44	1072	3.8E-04	N/A	N/A		1.0	0.0	0.000	0.00
3	SILT (ML)	1179	0.59	1095	5.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	ean CLAY w/ sand (C	1403	0.70	1055	7.3E-04	0	0.0E+00		1.0	0.0	0.000	0.00
5	RAVEL w/ clay and s	1708	0.85	1697	5.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
6	Clayey SAND (SC)	1952	0.98	1631	6.4E-04	below water	#VALUE!	0	1.0	0.0	0.000	0.00
7	andy Lean CLAY (CL	2156	1.08	1668	6.7E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	Clayey SAND (SC)	2328	1.16	1787	6.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
9	Clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-116

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-117
Date:	12/17/2008
Calculated By:	AJS

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	8
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	104.6
Design Ground Water Elev. (feet)	96.6

SUBSURFACE PROFILE

Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill -Sandy Lean CLAY (CL)	0	125	3	70	20	N/A	0.0
2	Clayey SAND (SC)	3.5	125	0	48		N/A	0.0
3	Sandy Lean CLAY (CL)	7	125	10	70	22	N/A	0.0
4	Sandy Lean CLAY (CL)	8	125	10	70	22	N/A	0.0
5	Clayey SAND (SC)	26	125	10	21		0.32	0.0
6	Clayey SAND w/ gravel (SC)	28	125	18	22		0.66	0.0
7	Clayey SAND w/ gravel (SC)	35	125	60	22		N/A	0.0
8	Sandy Lean CLAY (CL)	48	125	35	70	22	N/A	0.0
9	Silty SAND w/ grave (SM)	65	125	50	15		N/A	0.0
10	Lean CLAY w/ sand (CL)	73	125	27	80	22	N/A	0.0
11	Lean CLAY w/ sand (CL)	76.5	125	27	80	22	N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-117

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	-Sandy Lean CLAY (	0.0	219	219	0.49	12	1.15	0.43		N/A	1.00	108
2	Clayey SAND (SC)	3.5	656	656	0.49	5	1.15	0.42		N/A	0.99	320
3	andy Lean CLAY (CL	7.0	938	938	0.49	25	1.15	0.42		N/A	0.98	455
4	andy Lean CLAY (CL	8.0	2125	1563	0.64	22	1.15	0.56		N/A	0.96	1008
5	Clayey SAND (SC)	26.0	3375	2189	0.71	19	1.15	0.62		0.0	0.94	1562
6	ey SAND w/ gravel (	28.0	3938	2471	0.72	29	1.15	0.63		0.0	0.92	1785
7	ey SAND w/ gravel (	35.0	5188	3097	0.69	83	1.15	0.60		N/A	0.84	2143
8	andy Lean CLAY (CL	48.0	7063	4036	0.62	49	1.15	0.54		N/A	0.71	2492
9	ty SAND w/ grave (S	65.0	8625	4819	0.54	53	1.15	0.471		N/A	0.61	2610
10	an CLAY w/ sand (C	73.0	9344	5179	0.50	35	1.15	0.438		N/A	0.57	2611
11	an CLAY w/ sand (C	76.5	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A

Design Earthquake M = 7.26

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	-Sandy Lean CLAY (	142	0.07	270	2.0E-04	enter	#VALUE!		1.0	0.0	0.000	0.00
2	Clayey SAND (SC)	427	0.21	351	4.6E-04	enter	#VALUE!		1.0	0.0	0.000	0.00
3	andy Lean CLAY (CL	609	0.30	711	3.2E-04	0	0.0E+00		1.0	0.0	0.000	0.00
4	andy Lean CLAY (CL	1016	0.51	887	5.7E-04	0	0.0E+00		1.0	0.0	0.000	0.00
5	Clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
6	ey SAND w/ gravel (	N/A	N/A	N/A	N/A	below water	#VALUE!	0	1.0	0.0	0.000	N/A
7	ey SAND w/ gravel (	2013	1.01	1926	5.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
8	andy Lean CLAY (CL	2623	1.31	1853	6.7E-04	N/A	N/A		1.0	0.0	0.000	0.00
9	ty SAND w/ grave (S	3132	1.57	2074	6.3E-04	N/A	N/A		1.0	0.0	0.000	0.00
10	an CLAY w/ sand (C	3366	1.68	1877	7.0E-04	N/A	N/A		1.0	0.0	0.000	0.00
11	an CLAY w/ sand (C	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-117

PROJECT INFORMATION	
Project Number:	1595.002
Project Name:	CHRCO
Location:	B-118
Date:	12/17/2008
Calculated By:	AJS

CALCULATION PARAMETERS	
Design Fault	Hayward
Design Magnitude	7.26
Design PGAmax (g)	0.76
Design Ground Water Depth (feet)	8
Boring Diameter (inches)	8
Standard Sampler, i.e., without a cut out for liners (Y or N)	Y
Hammer Type RC = Rope and Cathead WT = Wire Trip A = Automatic	A
FS for Liquefaction	1.1
Ground Surface Elev. (feet)	105.7
Design Ground Water Elev. (feet)	97.7

SUBSURFACE PROFILE

Layer	Soil Description	Depth to Top of Layer (feet)	γ_{total} (pcf)	$N_{SPTfield}$ Average Value for Layer	Passing #200 Sieve (%)	Plasticity Index (%)	FS Against Liquefaction	Settlement (inches)
1	Fill -Sandy Lean CLAY (CL)	0	125	12	70	20	N/A	0.0
2	Well-graded SAND (SW)	6	125	17	4		N/A	0.0
3	Well-graded SAND (SW)	8	125	29	4		N/A	0.0
4	Sandy Lean CLAY (CL)	15	125	18	70	22	N/A	0.0
5	Clayey SAND (SC)	24.5	125	7	40		0.28	0.0
6	Lean Clay (CL)	28	125	16	90	22	N/A	0.0
7	Lean Clay (CL)	31.5	125	16	90	22	N/A	N/A
8							N/A	N/A
9							N/A	N/A
10							N/A	N/A
11							N/A	N/A
12							N/A	N/A
13							N/A	N/A
14							N/A	N/A
15							N/A	N/A
16							N/A	N/A
17							N/A	N/A
18							N/A	N/A
19							N/A	N/A
20							N/A	N/A

Dynamic Densification (inches)	0.0
Liquefaction Settlement	0.0

TOTAL SETTLEMENT (inches)	0.0

Table 1: Liquefaction Analysis for B-118

Settlement Calculations Based on Tokimatsu and Seed (1984, 1987)

Layer	Soil Description	Depth to Top of Layer (feet)	Total Stress at Middle of Layer (psf)	Effective Stress at Middle of Layer (psf)	CSR	(N1)60cs *7	Cm	CSR_{eq}	Final Volumetric Strain (%)	Liquefaction Induced Settlement (inches)	rd *10	CSR * Effective Stress
1	-Sandy Lean CLAY (	0.0	375	375	0.49	32	1.15	0.43		N/A	0.99	184
2	ell-graded SAND (SV	6.0	875	875	0.49	29	1.15	0.42		N/A	0.98	425
3	ell-graded SAND (SV	8.0	1438	1219	0.57	42	1.15	0.49		N/A	0.97	691
4	andy Lean CLAY (CI	15.0	2469	1736	0.67	38	1.15	0.58		N/A	0.95	1163
5	Clayey SAND (SC)	24.5	3281	2142	0.71	17	1.15	0.62		0.0	0.94	1522
6	Lean Clay (CL)	28.0	3719	2362	0.72	30	1.15	0.63		N/A	0.93	1710
7	Lean Clay (CL)	31.5	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
8	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
9	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
10	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
11	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
12	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
13	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
14	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
15	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
16	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
17	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
18	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
19	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A
20	0	0.0	N/A	N/A	N/A	N/A	1.15	N/A		N/A	N/A	N/A

Design Earthquake M = 7.26

Layer	Soil Description	Mean Effective Bulk Stress (psf)	Mean Effective Bulk Stress (tsf)	Gmax (tsf)	γeff * Geff/Gmax	γeff (Figure A-3)	γeff (%)	Volumetric Strain (%) M=7.5 (Figure A-4)	Volumetric Strain Ratio (Figure A-5)	Volumetric Strain (%) M=M	Final Volumetric Strain (%)	Dynamic Densification (inches)
1	-Sandy Lean CLAY (	244	0.12	492	1.9E-04	N/A	N/A		1.0	0.0	0.000	0.00
2	ell-graded SAND (SV	569	0.28	723	2.9E-04	enter	#VALUE!		1.0	0.0	0.000	0.00
3	ell-graded SAND (SV	792	0.40	964	3.6E-04	N/A	N/A		1.0	0.0	0.000	0.00
4	andy Lean CLAY (CI	1128	0.56	1115	5.2E-04	N/A	N/A		1.0	0.0	0.000	0.00
5	Clayey SAND (SC)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
6	Lean Clay (CL)	1535	0.77	1205	7.1E-04	below water	#VALUE!	0	1.0	0.0	0.000	0.00
7	Lean Clay (CL)	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
8	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
9	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
10	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
11	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
12	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
13	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
14	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
15	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
16	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
17	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
18	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
19	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A
20	0	N/A	N/A	N/A	N/A	N/A	N/A		1.0	0.0	0.000	N/A

Table 2: Liquefaction Analysis for B-118

APPENDIX F5

Preliminary Geotechnical Design Report, Caltrans Right-of-Way Decertification

FUGRO CONSULTANTS, INC.

Preliminary Geotechnical Design Report
Caltrans Right-of-Way Decertification
Children's Hospital & Research Center
Oakland, California

Prepared for:

CHILDREN'S HOSPITAL & RESEARCH CENTER AT OAKLAND
C/O CLEO CONSTRUCTION MANAGEMENT

April 2014

Fugro Project No. 04.72130048

PRELIMINARY

REVISION TABLE

Rev. #	Date	Approved By:	Description of Change:
0	11/25/13	RLB	Draft Document Release for CHRCO and CLEO review
1	4/28/14	RLB	Preliminary Document Release for CHRCO

PRELIMINARY

April 28, 2014
Project No. 04.72130048

Children's Hospital & Research Center Oakland
747 52nd Street
Oakland, California 94609

Attention: Mr. Doug Nelson

Subject: Preliminary Geotechnical Design Report, Caltrans Right-of-Way Decertification, Children's Hospital & Research Center Oakland, California

Dear Mr. Nelson:

Fugro is pleased to submit this Preliminary Geotechnical Design Report for the Caltrans Right-of-Way Decertification project at Children's Hospital & Research Center Oakland. This study was performed in general accordance with Amendment 1 of our Master Agreement dated June 11, 2013.

The report contains the results of our geotechnical investigation and engineering analyses performed to assist in the design of embankments, retaining walls, and utilities. The report also provides recommendations for grading. This report defines the geotechnical conditions as evaluated from field and laboratory test data and used in the development of the geotechnical design. It provides recommendations and specifications for project design and construction.

We appreciate the opportunity to be of service on this project. Please contact us if we can be of assistance regarding the information presented in the enclosed report.

Sincerely,

FUGRO CONSULTANTS, INC.

Matthew J. Janousek, P.E., G.E.
Project Engineer

Ronald L. Bajuniemi, P.E., G.E.
Senior Consultant

Copies Submitted: (1) Addressee - electronic format

PRELIMINARY

CONTENTS

Page

1.0 INTRODUCTION 1
1.1 Purpose 1
1.2 Project Description 1
2.0 SCOPE OF WORK 2
3.0 REVIEW OF EXISTING DATA 3
4.0 SITE EXPLORATION 3
4.1 Subsurface Exploration Programs 3
4.2 Laboratory Testing Program 4
5.0 SITE CONDITIONS 4
5.1 Highway 24 and Existing Facilities 4
5.2 Surface Conditions 5
5.3 Site Geology 5
5.4 Subsurface Conditions 6
5.4.1 Caltrans Embankment Fill (Af) 6
5.4.2 Alluvium (Qal) 6
5.5 Groundwater 7
5.5.1 Current Study 7
5.5.2 Previous Studies 7
5.5.3 Design Groundwater Elevations 8
6.0 FAULTING AND SEISMICITY 8
6.1 Nearby Faults 8
6.2 Ground Shaking 9
6.3 Site Class, Seismic Design Parameters, and Seismic Design Category 10
6.4 Ground Surface Fault Rupture 10
6.5 Seismic Settlement of Unsaturated Embankment and Alluvial Soils 11
6.6 Liquefaction Potential 11
6.7 Lateral Spreading 12
6.8 Tsunami and Seiche 12
6.9 Landsliding 13
7.0 ANALYSIS AND DESIGN 13
7.1 General 13
7.2 Geotechnical Design Parameters for Retaining Walls 13
7.3 Design of Permanent Tie-Back Walls 14
7.3.1 Apparent Earth Pressures 14
7.3.2 Dynamic Earth Pressures 14
7.3.3 Traffic Surcharges 15
7.3.4 Tieback Anchor Design 15
7.3.5 Soldier Piles and Passive Resistance 15

PRELIMINARY

7.3.6 Global Stability 16
7.3.7 Anchor Installation and Testing 16
7.3.8 Tie-Back Wall Drainage 18
7.3.9 Grouting Considerations 19
7.3.10 Corrosion Protection 20
7.3.11 Tie-Back Construction Considerations 21
7.4 Design of Concrete Gravity Retaining Walls 21
7.4.1 General 21
7.4.2 Lateral Earth Pressures 21
7.4.3 Wall Drainage 22
7.4.4 Support of Gravity Wall 22
7.4.5 Coefficient of Friction 22
7.4.6 Passive Resistance 22
7.5 Site Preparation and Grading 22
7.5.1 Site Preparation 22
7.5.2 Subgrade Preparation 23
7.5.3 Engineered Fill Materials 23
7.5.4 Fill Placement and Compaction 23
7.5.5 Surface Drainage 24
7.6 Utility Construction 24
7.6.1 General 24
7.6.2 Pipe Zone Materials 24
7.6.3 Trench Backfill 25
7.7 Pavements 25
8.0 INVESTIGATION LIMITATIONS 27
9.0 REFERENCES 28

TABLES

Page

Table 1. Summary of Proposed Retaining Walls 2
Table 2. Nearby Faults 9
Table 3. Probabilistic Seismic Hazard Analysis Results 10
Table 4. Seismic Parameters 10
Table 5. Summary of Geotechnical Design Parameters 14
Table 6. Results of Preliminary Corrosivity Testing 20
Table 7. Recommended Pavement Design Alternatives 26

PRELIMINARY

PLATES

Plate

Vicinity Map 1
Exploration Location Plan 2
Geologic Map 3
Cross-Section A-A' 4a
Cross-Section B-B' 4b
Cross-Section D-D' 4d
Cross-Section E-E' 4e
Regional Fault Map 5
Design Acceleration Response Spectrum 6
Apparent Earth Pressure Diagram for Tie-Back Walls ..... 7a and 7b
Lateral Earth Pressures on Gravity Retaining Walls 8

APPENDICES

APPENDIX A FIELD EXPLORATIONS

Key to Terms & Symbols Used on Logs A-1
Boring Logs ..... A-2 through A-7
Key to CPT Interpretation A-8
Log of CPTs ..... A-9 through A-19
Historical Boring Logs

APPENDIX B LABORATORY TESTING PROGRAM

Summary of Laboratory Test Results ..... Plate B-1
Grain Size Curves ..... Plate B-2
Plasticity Chart ..... Plate B-3a and B-3b
Unconsolidated, Undrained Triaxial Test ..... Plates B-4a through B-4k
Direct Shear Test Results ..... Plates B-5a through B-5c
Consolidation Test Results ..... Plate B-6

APPENDIX C LIQUEFACTION EVALUATION RESULTS

CPT Liquefaction Logs ..... Plates C-1 through C-11
Key to CPT Liquefaction Logs ..... Plate C-12
Boring Liquefaction Calculations ..... Plates C-13 through C-18

APPENDIX D CALCULATIONS

Dry Seismic Sottement Calculations ..... Plates D-1 through D-3

PRELIMINARY

1.0 INTRODUCTION

This report presents the results of the geotechnical study conducted by Fugro Consultants, Inc. (Fugro) for the Caltrans Right-of-Way Decertification project at Children's Hospital & Research Center Oakland (CHRCO). The project consists of cutting back the existing Highway 24 freeway embankment and constructing retaining walls as part of a two-phase master plan to expand the CHRCO campus. The general location of the subject embankment is shown on Plate 1 – Vicinity Map. This report addresses the design of retaining walls, utilities, pavement and other flatwork. The site coordinates near the middle of the site are:

Latitude: 37.8377º
Longitude: -122.2662º

CLEO Construction Management is the construction manager for CHRCO, HDR is the project architect and structural engineer, SANDIS is the civil engineer, and Gray-Bowen is assisting in facilitating the review process through Caltrans.

1.1 PURPOSE

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of observed site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the proposed tie-back retaining walls. This geotechnical study provides a baseline for geotechnical recommendations; however, as either new information becomes available or design modifications are investigated, supplemental geotechnical recommendations will be addressed through separate technical memoranda, but not revision to this report.

1.2 PROJECT DESCRIPTION

As currently envisioned, the retaining wall systems will consist of tied-back and gravity retaining walls. To construct the walls, CHRCO will need to obtain Caltrans Right-of-Way (ROW) through the ROW decertification process. As presently planned, the retaining walls will extend from the south end of the CHRCO campus near the Martin Luther King Jr. Way undercrossing to the southside of 53rd Street.

Based on a review of preliminary conceptual project plans provided by SANDIS (2014), we understand that there will be multiple retaining wall configurations, including both single- and multi-level tie-back walls as well as a concrete cantilever gravity wall. Conceptual retaining wall design types, heights, and approximate lengths are summarized in Table 1.

PRELIMINARY

Table 1. Summary of Proposed Retaining Walls

Approximate Wall Length (ft)	Wall Type	Approximate Maximum Wall Free Height (ft)	Backslope Condition Above Wall
375	Single-Tiered Tie-back Wall	25 to 30	Level
190	Two-Tiered: Upper Wall Tie-backs Lower Wall Gravity	Upper Wall: 7 to 10.5 Lower Wall: 10	Upper Wall: Level Lower Wall: 3:1 Slope
570	Single-Tiered Tie-back Wall	15	2:1 Slope

Other improvements will include a greenbelt walkway adjacent to the retaining walls to provide access between 52nd and 53rd Streets, and other associated hardscapes.

We understand that the proposed retaining walls will not be considered a critical structure under the California Building Code and will not be subject to review by the California Office of Statewide Health Planning and Development (OSHPD).

2.0 SCOPE OF WORK

The scope of our services included:

- Review of the readily available as-built data provided by Caltrans. Also compiling and reviewing available geotechnical and geologic data that is contained in our files and is pertinent to the project vicinity. This includes a series of previous reports prepared for various developments at CHRCO;
- Procuring a Caltrans encroachment permit and an Alameda County Public Works Agency (ACPWA) well permit for our exploration program;
- Conducting field exploration and laboratory-testing programs including six (6) rotary wash borings and eleven (11) Cone Penetration Tests (CPT) to supplement the available information on subsurface conditions along the proposed retaining walls;
- Preparation of this report that presents our discussions, conclusions, and recommendations regarding the following:
  - Laboratory tests results;
  - Soil, rock, and groundwater conditions encountered;
  - Discussion of regional and local geologic and seismic conditions;
  - Geotechnical parameters/coefficients for seismic design, based on the 2010 California Building Code and/or the Caltrans ARS method;
  - Criteria for design of retaining walls for two wall types, including active, at rest and passive pressures, surcharge pressure criteria, wall drainage, and dynamic earth pressures;

PRELIMINARY

- Design for permanent tie-backs including tie-back capacities, extent of bonded and unbonded zones, corrosion protection, and load testing requirements;
- Recommendations for surface and subsurface drainage, as appropriate;
- Discussion of construction considerations for the proposed retaining walls;
- Recommendations for earthwork, including site preparation and grading, subgrade preparation, temporary and permanent slopes, excavation, fill placement and compaction, utility trench backfill;
- Recommendations for suitability of excavated, on site materials for reuse as fill or backfill material; and
- Recommendations for asphalt concrete pavement sections.

3.0 REVIEW OF EXISTING DATA

Prior to conducting our field exploration and laboratory testing, Fugro reviewed relevant information relating to geotechnical, geologic, and seismic issues at the site. We obtained and reviewed Caltrans' Highway 24 and 52nd Street on-ramp as-built plans (Caltrans 1970) as well as Log of Test Borings (LOTB) for the retaining wall at the end of Dover Street (Caltrans 1969). Relevant information from the as-built plans and LOTB was used in preparing the cross sections presented in this report, and for our engineering analyses.

Fugro's local predecessor companies completed several geotechnical investigations at CHRCO since 1986. Select relevant historic boring locations from our previous work proximal to the current project site are shown on Plate 2 –Exploration Location Plan. In addition, relevant data from these borings was used in preparing the cross sections presented in this report, and for our engineering analyses.

A complete list of references used in this study is included in Section 9.0 – References. Pertinent previous boring logs used to interpret subsurface conditions at the proposed structures are included in Appendix A.

4.0 SITE EXPLORATION

4.1 SUBSURFACE EXPLORATION PROGRAMS

Current Study. As part of the current investigation to supplement our previous investigations, six (6) hollow-stem-auger and mud rotary wash borings and eleven (11) cone penetrometer tests (CPTs) were performed to a maximum depth of approximately 80 feet below the ground surface (bgs). Approximate locations of the borings and CPTs are shown on Plate 2. Explorations were conducted during August and September of 2013.

Borings were performed by using a CME-55 rig provided by Pitcher Drilling Co. of East Palo Alto, California. CPTs were performed by Fugro using a 20-ton truck-mounted CPT rig.

Drilling and CPT operations were conducted under the technical guidance and observation of a Fugro engineer who also described and packaged the recovered samples.

PRELIMINARY

Sample descriptions were later checked for conformance to the Unified Soil Classification System, as described in ASTM Standard Test Methods D 2487 and D 2488, and the Caltrans 2010 Soil and Rock Logging Manual (Caltrans, 2010).

Prior to beginning field exploration, Fugro obtained the necessary Caltrans and Alameda County Public Works Association (ACPWA) encroachment permits to conduct the field activities. We also contacted Underground Service Alert (USA) prior to our fieldwork in order to obtain utility clearance at our exploration locations. During our field exploration, portions of shoulder and outside lanes of Highway 24 and the 52nd Street on-ramp were closed. Traffic control was provided in accordance with Caltrans requirements by Cal-Safety, Inc. of Fremont, California.

Upon completion of field explorations, borings and CPTs were backfilled with cement grout in accordance with the permits. Explorations performed in paved areas were patched with a concrete plug or asphalt cold patch. Excavated soil cuttings and drilling fluid were placed in 55-gallon drums and temporarily stored off site at a Caltrans staging area. Upon completion of analytical testing, the drums were hauled off the staging area and properly disposed of at an appropriate facility.

Previous Geotechnical Studies. Harza Engineering Company and Kaldveer Associates (two of Fugro's locally acquired predecessor companies) and Fugro have performed several geotechnical studies for the hospital between 1986 and 2011. Our previous investigations included fifteen soil borings (advanced between 1992 and 2008) in relatively close proximity to the toe of the existing Highway 24 embankment. These borings indicate that the subsurface conditions in the exploratory boring locations generally consisted of firm to hard clay and silt interlayered by medium dense to very dense sand and gravel.

4.2 LABORATORY TESTING PROGRAM

Laboratory tests were performed to classify the subsurface soils, determine their engineering properties, and to determine strength and corrosivity characteristics. The purpose of a soil classification system is to describe, record, and transmit properties of soil following standardized procedures. Tests were performed in general accordance with ASTM procedures and Caltrans Test Methods (CTM). The laboratory program included index and classification tests, particle size analyses (ASTM D 422), Atterberg Limits (ASTM D 4318), Unconsolidated Undrained and Consolidated Undrained triaxial compression tests (ASTM D 2850 & D4767), direct shear tests (ASTM D 3080), a consolidation test (ASTM D2435), and soil corrosion tests (CTM 643). Detailed descriptions of laboratory test procedures and a summary of laboratory test results are presented in Appendix B.

5.0 SITE CONDITIONS

5.1 HIGHWAY 24 AND EXISTING FACILITIES

Highway 24 is an 8-lane asphalt paved roadway, separated by a two-way Bay Area Rapid Transit (BART) line that runs between the eastbound and westbound lanes of the highway. In the project vicinity, Highway 24 generally runs north and south, and then curves to

PRELIMINARY

the northeast north of 52nd Street. As-built plans indicate that construction of Highway 24 and the 52nd Street on-ramp was completed between 1969 and 1970. The westbound Highway 24 bridge that crosses over 52nd Street is designated as Caltrans Bridge No. 33-0416L.

As part of the original Highway 24 construction, an approximate 300-foot-long, Type 1 gravity retaining wall was built into the embankment slope at the south end of Dover Street along the CHRCO campus. The wall ranges in height from 6 to 22 feet, with a 2:1 backslope behind the wall up to freeway grade.

Multiple utilities cross through the Caltrans ROW along the project alignment. A 10-foot-wide by 12-foot high Alameda County Flood Control emergency storm drain box culvert crosses beneath the Caltrans embankment at the south end of the hospital campus near Martin Luther King Jr. Way. Surficial run-off catch drains also run underneath the embankment, and empty into larger storm drains at the hospital grade. Additionally, a PG&E vault encroaches into the base of the Caltrans embankment at the eastern terminus of 53rd Street.

5.2 SURFACE CONDITIONS

The proposed retaining walls will run approximately north-south along the western edge of Highway 24 and the 52nd Street on-ramp. The site is bordered by southside of 53rd Street to the north, 52nd and existing hospital buildings to the west, Martin Luther King Jr. Way to the south, and Highway 24 to the east. In the northern portion along south of 53rd Street, several houses (residential and hospital offices) and associated yards and driveways currently are adjacent to the site. Embankment slopes are heavily vegetated with trees ranging in diameter from about 6 to 15 inches, along with smaller shrubs.

The new retaining walls will replace existing Caltrans embankment slopes and a concrete gravity retaining wall. Slopes range in height from about 15 to 30 feet along the hospital campus. 52nd Street on-ramp elevations range from about el. +116 feet at the 52nd Street overcrossing to about el +127 feet near the south end of the hospital at the Martin Luther King Jr. Way overcrossing. Along Highway 24 north of 52nd Street, freeway grade elevation is about el. +132 feet near the 53rd Street terminus.

5.3 SITE GEOLOGY

The Children's Hospital site is located on a gradually sloping outwash plain located between the Berkeley Hills to the east and San Francisco Bay to the west. The native soils at the site consist of Quaternary age alluvial deposits likely including materials from the San Antonio and Temescal Formations, according to Radbruch and Case (1967) and Helley, Lajoie and Burke (1972). Graymer (2000) maps the site as Holocene age alluvial and fluvial deposits, as shown on Plate 3 – Geologic Map.

PRELIMINARY

5.4 SUBSURFACE CONDITIONS

Geologic units are described below in order of increasing depth. Detailed descriptions of deposits encountered in each of the borings and CPTs are presented on the boring and CPT logs in Appendix A. Interpreted subsurface profiles are shown on Plates 4a through 4f.

The attached boring and CPT logs and related information depict location specific subsurface conditions, encountered during our field exploration. Passage of time could result in changes in the subsurface conditions.

Subsurface conditions at the site are generally characterized by four layers, described below in order of increasing depth: I) Caltrans embankment sandy and clayey fill with gravel; II) sandy lean clay alluvium; III) interlayered clayey sand and sandy clay alluvium with varying amounts of gravel, and IV) lean clay alluvium to the maximum depth explored of 80 feet bgs.

5.4.1 Caltrans Embankment Fill (Af)

Caltrans embankment fill encountered in this study consist primarily of medium dense to dense clayey sands and gravels, and interbedded with stiff to hard sandy lean clays with varying amounts of gravel. Fill depths ranged from about 15 to 32 feet below freeway grade. Fugro does not possess documentation indicating if these fills were properly compacted during placement.

In-situ dry unit weights measured in the embankment fill generally range from about 106 to 136 pounds per cubic foot (pcf), with an average of about 120 pcf. Water content measured in this layer range from about 2 to 24 percent, with an average of about 12 percent. Atterberg limits show the Plasticity Index in the clayey materials generally range from about 13 to 26, indicating moderate expansion potential. One sample out of 12 tests in the embankment materials resulted in a Plasticity Index of 44, indicating critical expansion potential. Undrained shear strength (Su) measurements in this layer from pocket penetrometer tests ranged from about 2.5 tsf to greater than 4.5 tsf.

5.4.2 Alluvium (Qal)

Below the embankment fill the alluvial stratigraphy encountered across the site typically consisted of a layer of stiff lean clay, over interlayered clayey sands to sandy clays with varying amounts of gravel over stiff lean clay. To depths of about 10 to 20 feet below the embankment fill, we observed a layer of stiff to very stiff lean clay with various amounts of sand and gravel. Sand and gravel content was observed to increase with depth, and layers of medium dense to dense clayey sand with varying amounts of gravel were generally observed below the surficial stiff clays. Below the interlayered sandy clay and clayey sand, we generally observed very stiff to hard lean clay with sand.

In-situ dry unit weights measured in the upper 50 feet of alluvial soils generally range from about 98 to 124 pcf, with an average of about 110 pcf. Water content measured in these layers range from about 4 to 30 percent, with an average of about 20 percent. Atterberg limits

PRELIMINARY

show the Plasticity Index in the clayey materials generally range from about 10 to 26, indicating low to moderate expansion potential. One sample out of 6 tests in the alluvial materials resulted in a Plasticity Index of 44, indicating critical expansion potential. Undrained shear strength (Su) measurements in this layer from pocket penetrometer tests ranged from about 2.0 tsf to greater than 4.5 tsf.

5.5 GROUNDWATER

5.5.1 Current Study

Groundwater levels were observed to vary significantly across the site during our subsurface investigation. Groundwater was encountered as shallow as about 31 feet below grade in Boring B-05 (about el. +133 feet) and as deep as about 42 feet below grade in Boring B-06 (about el. +103 feet). Groundwater was typically measured within 30 minutes of completing the boreholes because the borings were backfilled immediately after drilling with cement grout in accordance with our Caltrans and Alameda County drilling permits. Therefore, the borings may not have been left open for a sufficient period of time to establish equalized groundwater conditions.

Two pore dissipation tests were performed in the CPTs at CPT-01 and CPT-11. Interpreted dissipation test results suggest static groundwater elevations of about +78 and +97 feet at CPT-01 and CPT-11, respectively.

The CGS Seismic Hazard Zone Report for the Oakland West Quadrangle (1991) presents a map that shows approximate depths to historic high ground water. Groundwater depth contours presented on that map suggest historic high ground water at the Children's Hospital site to be on the order of about 7.5 feet below hospital grade. Fluctuations in the groundwater level may occur from changes in seasons, variations in rainfall, and other factors.

5.5.2 Previous Studies

Groundwater was encountered as shallow as about 8 1/2 feet below grade in historical (February 2008) Boring PB-101 and as deep as about 40 feet below grade in Boring PB-104. During Fugro's August 2008 investigation, groundwater was encountered between 20 feet and 25 feet below hospital grade (about el. +71 to +76).

As part of the geotechnical investigation for the Patient Services Pavilion, Harza (1994) installed two monitoring wells at the existing hospital. In one monitoring well, groundwater was measured at about 22 feet bgs (about el. +79 feet) in 1992 and at about 21 feet bgs (about el. +80 feet) in 2008. In the other well, groundwater was measured at about 16 feet bgs (about el. +86.5 feet) in 1992 and about 12.5 feet bgs (about el. +90 feet) in 2008.

Historically, groundwater levels at the Children's Hospital site have varied considerably. Observed levels range from 6 feet below grade in at the Family House east of Dover Street, to 23 feet below grade in Boring PB-5(86a) (K928-1, 1986), located in the current parking garage area. Boring PB-5 (86a) was converted into a temporary observation well and groundwater was

PRELIMINARY

recorded at a depth of 17 feet below grade three weeks after drilling. Near the Ambulatory Services Center, north of 52nd Street and to the west of Dover Street, groundwater was recorded at depths of 12 to 23 feet during investigations in 1986, 1990 and 1991.

5.5.3 Design Groundwater Elevations

Based on the depths of observed shallow groundwater in boring, CPT, and historical well measurements, we have estimated a design ground water depth of 8 feet below hospital grade (el. +89) for the retaining walls south of 52nd Street, and 8 feet bgs (el. +95) for the retaining walls north of 52nd Street. Fluctuations in groundwater level may occur from changes in seasons, variations in rainfall, and other factors.

6.0 FAULTING AND SEISMICITY

Potential seismic hazards can include ground shaking, surface fault rupture, dry seismic settlement, liquefaction, lateral spreading, tsunami, and landsliding. A discussion of seismic hazards that may potentially impact the site is provided below.

6.1 NEARBY FAULTS

The project site is within a seismically active region of California. We performed a fault search of the project area using current Caltrans Seismic Design Criteria and utilizing the Caltrans ARS Online V2.2.06 (Caltrans 2013). ARS Online is updated on an ongoing basis and was accessed in October 2013 through the Caltrans website.

Table 2 presents a list of faults closest to the site identified using ARS Online and site coordinates corresponding to Latitude: 37.8377º and Longitude: -122.2662º. The site is approximately 2 miles west of the closest active fault zone (Hayward fault) depicted on the Special Studies Zones map (CGS, 1982), as shown on Plate 5 – Regional Fault Map.

PRELIMINARY

Table 2. Nearby Faults

Fault Name	Fault Type[1]	R_x Distance (km)[2]	Maximum Magnitude (Mmax)[3]
Hayward (North)	Strike-Slip	3.2	7.3
Hayward (South)	Strike-Slip	2.4	7.3
San Andreas (Peninsula)	Strike-Slip	26.7	8.0
San Andreas (North Coast)	Strike-Slip	25.5	8.0
Calaveras	Strike-Slip	16.3	6.9

[1] Fault type per Caltrans 2012 Fault Database.
[2] Horizontal perpendicular distance to the fault trace (*fictitious fault trace for sites offset from the mapped fault location) or surface projection of the top of rupture plane measured perpendicular to the fault from the site per ARS Online and Caltrans Geotechnical Services Design Manual Version 1.0 (2009).
[3] Mmax values per ARS Online and Caltrans 2012 Database.

6.2 GROUND SHAKING

ARS Online generally displays information for faults included in the Caltrans 2012 Fault Database and calculates both deterministic and probabilistic (975 year return period event) acceleration response spectra (ARS) for locations in California as described in Appendix B of the Caltrans Seismic Design Criteria (Caltrans 2010). Probabilistic and controlling deterministic response curves recommended for structural design are presented on Plate 6 - Design Acceleration Response Spectrum. The design ARS curve on Plate 6 is taken as the upper envelope values considering both deterministic and response spectrum estimates.

We estimated the average shear wave velocity in the upper 100 feet of foundation soil strata (below Caltrans embankment fill) equivalent to approximately 1,050 feet-per-second (320 meters-per-second) based upon the interpreted soil conditions at the site, geotechnical laboratory testing, and CPT data correlations presented in Caltrans' Geotechnical Services Design Manual, Version 1.0 (2009). Given the site coordinates and this site specific input parameter, the probabilistic response curve controlled the design spectrum at all periods.

We also performed a probabilistic seismic hazard analysis of the site using the USGS deaggregation web application (USGS, 2008b). The assumed shear wave velocity corresponds to a site class D soil profile (stiff soil) and we feel that it approximately represents the subsurface conditions to the ultimate depths explored. On the basis of our analyses, we estimate the site could experience a peak horizontal ground acceleration of about 0.78g from a 975-year return period earthquake event. This event corresponds to 5 percent probability of exceedance in 50 years. Table 3 summarizes peak ground accelerations for the site with additional return periods of 475 and 2475 years.

PRELIMINARY

Table 3. Probabilistic Seismic Hazard Analysis Results

Return Period (years)	Peak Horizontal Ground Acceleration (g)	Mean Earthquake Magnitude (Mw)
475	0.64	6.8
975	0.78	6.8
2475	1.06	6.8

6.3 SITE CLASS, SEISMIC DESIGN PARAMETERS, AND SEISMIC DESIGN CATEGORY

The proposed retaining walls should be designed to resist the lateral forces generated by earthquake shaking in accordance with Caltrans design practice. This section presents seismic design parameters for use with the 2010 California Building Code (CBC). Seismic parameters summarized in Table 4 were determined by the simplified method for spectrum development in accordance with ASCE 7-05 Section 11.1. These parameters were used in the development of the design response spectrum ordinates. The 2010 CBC refers to the design code by American Society of Civil Engineers (ASCE 7-05) for the development of site-specific response spectra for critical facilities within 10 km of an active fault. Site-specific response spectra were developed in accordance with Section 21.2 of ASCE 7-05.

Table 4. Seismic Parameters

Parameter/Coefficient Description	2010 CBC Reference	Parameter/ Coefficient	Value
MCE for 0.2 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(3)	S_s	1.73
MCE for 1.0 sec Spectral Response Acceleration, Site Class B	Figure 1613.5(4)	S_1	0.64
Long-Period Transition Period	ASCE 7-05 Figure 22-15	T_L	8
Soil Profile Type	Table 1613A.5.2	Site Class	D
Site Coefficient	Table 1613A.5.3(1)	F_a	1.0
Site Coefficient	Table 1613A.5.3(2)	F_v	1.5
Spectral Acceleration Parameters	Equation 16-39	S_{DS}	1.16
	Equation 16-40	S_{D1}	0.64

**Site Located at: Latitude 37.8377°; Longitude -122.2662°

6.4 GROUND SURFACE FAULT RUPTURE

The potential for surface fault rupture at the site is low. No known active fault traces are mapped crossing the Children's Hospital Oakland site, based on existing geologic maps and reports including Graymer (2000) and Helley and Graymer (1997). In addition, the site is not

PRELIMINARY

located within a Fault-Rupture Hazard Zone as designated by the Alquist-Priolo Earthquake Fault Zoning Act (1972), as mapped on the official Earthquake Fault Hazard Zone Maps issued by the State of California (1982).

6.5 SEISMIC SETTLEMENT OF UNSATURATED EMBANKMENT AND ALLUVIAL SOILS

Volumetric compression, or soil densification, is settlement caused by earthquake-induced ground shaking that causes soil particles to compress together. Dry cohesionless soils, such as sands and gravels, are susceptible to this type of settlement. Earthquake induced settlements of dry cohesionless soils depends on three main factors, including: the relative density of the soil, the maximum shear strain induced by the design earthquake and the number of shear strain cycles which can be related to earthquake magnitude.

We used the methodology proposed by Pradel (1998) to estimate the degree of dynamic densification potential in the unsaturated alluvial clay and clay fill above groundwater at the project site. The earthquake scenarios used in our analyses correspond to a peak horizontal ground acceleration (PGA) of 0.78g and an anticipated earthquake magnitude (maximum moment magnitude) of 6.8. Soils above groundwater at the site are dense to very dense clayey sands and gravels, and very stiff sandy clays. Our evaluations for the borings suggest that dynamic densification of unsaturated soils above groundwater at this site could range from about 1 to 1-1/4 inches. The majority of the estimated settlement occurs within the embankment fill. However, due to the interbedded nature of the unsaturated soils, we believe magnitude of this phenomenon will vary across the site. Calculations for soil dry densification evaluation are included in Appendix D.

6.6 LIQUEFACTION POTENTIAL

Susceptibility of a soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. Resistance to liquefaction increases with respective increases in a) distribution of grain size, b) soil density, c) aging, d) cementation, e) fines content, and f) plasticity characteristics of the fines. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface, a depth usually considered to be less than about 50 feet.

Liquefaction susceptibility at the project site is considered to be moderate, based on the Association of Bay Area Governments (ABAG) Liquefaction Susceptibility Map (2000). In addition, the site has a moderate to high liquefaction susceptibility as mapped by Witter and others (2006). The site is located in an area designated as a zone of required investigation for liquefaction by California Geological Survey (CGS), as shown on the Seismic Hazard Zones Maps, West Oakland Quadrangle (CGS, 2003).

Seismic settlement resulting from liquefaction in the borings and CPTs was estimated using procedures presented in Idriss and Boulanger (2008, Section 4.4). Earthquake scenarios used in our analyses correspond to a PGA of 0.78g and maximum moment magnitude of 6.8. The procedure for CPTs relates the post-earthquake reconsolidation volumetric strain to the factor of safety against liquefaction and normalized, clean sand, CPT tip resistance (q_{c1Ncs}).

PRELIMINARY

Normalized, clean sand, CPT tip resistance values are estimated using corrections presented in Section 3.11 of Idriss and Boulanger (2008). A correction to tip resistance (Δqc1N) is estimated from estimated fines content (FC) and the normalized tip resistance (qc1N), which is then added to qc1N to estimate qc1ncs. Fines content is correlated to a soil behavior index, which is based on normalized tip resistance and friction ratio.

Based on our liquefaction analyses, several previous studies performed for the hospital and performance of critical hospital structures during previous earthquakes, the materials beneath the proposed retaining walls are generally cohesive and/or dense enough to have a low potential for liquefaction. Estimated liquefaction-related settlements are less than 1 inch, and settlements are not anticipated to be continuous in nature. Liquefaction-related seismic settlement versus depth profiles are presented in Appendix C.

The low potential for liquefaction in the new borings agrees well with previous liquefaction studies around the CHRCO campus that also found low potential for liquefaction. As presented in Plates C-3 though C-6, CPT liquefaction settlement estimates for CPT-01 through CPT-04 are higher than estimated for the borings. The interpreted soil classifications on the CPT logs most likely do not take into consideration the plasticity of the fines, which were determined to be predominantly clayey fines as shown in the laboratory testing. In our estimation, the plasticity of the fines and the predominant grain size distribution of the coarse grained soils will preclude the onsite soils from experiencing the magnitude of liquefaction indicated on Plates C-3 though C-6.

6.7 LATERAL SPREADING

Lateral spreading occurs where the contact between a layer of liquefiable material and the material below is sloped. Saturated sands lose their strength during an earthquake and become fluid-like and mobile. As a result, the ground may undergo large permanent displacements that can damage underground utilities and well-built surface structures. Lateral spreading involves displacement of large blocks of ground down gentle (greater than 0.1 percent) slopes or towards the open face of slopes such as a stream channel. In areas where there is no open face, buckling of the overburden is often observed.

Historic earthquakes in the Bay Area have produced lateral spreading features. However, large-scale lateral spreading is considered unlikely at this site because the Caltrans embankment is on an essentially level plain and the probability for liquefaction at the site is considered low.

6.8 TSUNAMI AND SEICHE

During a major earthquake, strong waves such as tsunamis or seiches may be generated in large bodies of water and may cause damage to structures affected by them. A tsunami (or seismic sea wave) is an open ocean phenomenon caused by faulting, volcanism or other abrupt movements on the ocean floor. A seiche is a wave that occurs in an enclosed basin as a result of fault displacement in the basin bottom, large landslides into the basin, or from periodic oscillation or sloshing of the water in the basin in response to ground shaking.

PRELIMINARY

The potential for tsunami and seiche hazards at this site is considered remote. The site is located at an elevation of about +95 to +100 feet according to the project survey, and about 1.8 miles east of San Francisco Bay. In addition, the Tsunami Inundation Map for Emergency Planning (CGS, 2009) indicates that the site is not in an area that may be inundated by a tsunami.

6.9 LANDSLIDING

A landslide is a movement of a mass of soil down a steep slope when the soil cannot support the weight of overlying soil or rocks. Landslides vary in size and rate of movement. Slides can occur slowly over time or suddenly. Areas susceptible to landslides are those where masses of soils are weakly supported because of natural erosion, changes in ground water or surface water patterns, or human activities such as undercutting. Heavy rains or seismic shaking can also trigger landslides.

A large earthquake in the San Francisco Bay Region may cause movement of active slides and could trigger new slides. Evidence of previous or active slides was not observed during our site investigation. The site is not within a mapped area of existing or potential slope instability. The site is not located within a State of California designated seismically induced landslide hazard zone (CGS, 2003). In addition, the site is not located within a City of Oakland landslide Hazard Zone (Miles and others, 2001).

7.0 ANALYSIS AND DESIGN

7.1 GENERAL

Based on a review of preliminary conceptual project plans provided by SANDIS (2012 and 2013), we understand that there will be multiple retaining wall configurations, including both single- and multi-level permanent tie-back walls as well as a concrete cantilever gravity wall. Retaining wall designs include single wall tie-back systems with horizontal and sloping backfill, as well as a two-tiered system consisting of an upper tie-back wall and a lower concrete gravity wall. The following sections provide geotechnical design recommendation for each anticipated retaining wall system.

7.2 GEOTECHNICAL DESIGN PARAMETERS FOR RETAINING WALLS

Based on our observations and review of geotechnical laboratory test results, we have developed recommended geotechnical parameters to be used in the design of the proposed retaining walls systems. Table 5 summarizes key geotechnical parameters used in our analyses.

PRELIMINARY

Table 5. Summary of Geotechnical Design Parameters

Soil Type	Total Unit Weight (pcf)	Shear Strength Parameters		
		Undrained Strength, s_u (tsf)	Drained Friction Angle, ϕ' (deg)	Drained Cohesion, c' (tsf)
Caltrans Embankment Fill	125	1.5	30	0
Alluvium	120	1.0	28	0

7.3 DESIGN OF PERMANENT TIE-BACK WALLS

7.3.1 Apparent Earth Pressures

Plates 7a and 7b present our recommended apparent earth pressure distributions for soldier pile and lagging walls with single rows and multiple levels of tiebacks. Apparent earth pressures acting on the tieback retaining walls will depend on the spacing and diameter of the solider piles, backslope inclination, and soil properties. Pressure distributions can be modified to accommodate various soldier pile diameters (b), spacings (s), and wall heights (H). Soil unit weight (γ), soil friction angle (ϕ), and passive earth pressure coefficient (K_p) used for the calculations are shown on the plates. We assume that the walls will be constructed using conventional top-down construction, with the lagging placed behind the soldier piles against the relatively undisturbed face of a near-vertical excavation. Soldier piles and tiebacks should be designed for a factor of safety of at least 1.5 against sliding and overturning. The number and vertical spacing of the tiebacks should be determined based on the structural requirements needed to resist the recommended earth pressures.

Walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.

7.3.2 Dynamic Earth Pressures

Anchored walls can be designed using pseudostatic analyses based on the Mononobe Okabe approach (FHWA, 2011). The dynamic earth pressure that should be considered in the pseudostatic analyses is dependent on the anticipated ground motions at the site and the stiffness of the wall retaining the soil. We estimated dynamic earth pressures using the Mononobe Okabe method and assuming a horizontal ground acceleration of 0.39g (about 1/2 of the probabilistic peak ground acceleration) and assuming drained backfill conditions.

The estimated resulting force corresponding to the seismic force increment of dynamic earth pressure for those conditions is estimated to be about $12H^2$ and $18H^2$ pounds per linear foot for level and 2h:1v sloping backfill, respectively. This force can be assumed to act over the face of the wall following an inverted triangular pressure distribution with the resultant force acting 2/3H above the base of the wall.

PRELIMINARY

7.3.3 Traffic Surcharges

Provisions for traffic surcharges are shown on Plates 7a and 7b as an equivalent 2-foot soil surcharge resulting in a uniformly applied lateral pressure of 80 psf. Traffic loading or surface loads acting more than one-half the height of the wall behind the top of the wall can be neglected.

Traffic loading can also influence the lagging. Traffic loading or surface loads acting more than one-half of the clear spacing between solider piles behind the top of the wall can be neglected when designing the lagging.

7.3.4 Tieback Anchor Design

Bond Strength

Tieback anchors can be used to provide additional lateral resistance for retaining structures. Drill holes for tieback anchors used on the project will penetrate dense to very dense clayey sand and gravel, and very stiff sandy lean clay. For preliminary design of tiebacks, the ultimate unit bond stress for pressure-grouted anchors installed in Caltrans embankment materials can be assumed to be about 35 psi. A minimum factor of safety of 2 should be applied to the bond stress to determine tie-back lengths at the design load.

Bond stress and transfer loading should be confirmed during construction by performance and proof testing, as recommended in this report. Performance testing will also be needed to establish drilling and grouting methods, and grout takes that will provide the required capacity. We recommend a minimum bond length of 10 feet and that all anchors be designed and installed with provisions for post or secondary grouting (i.e. a grout tube should be installed with the anchor). Secondary or additional stages of grouting or lengthening the anchors beyond those zones, if encountered, may be needed to achieve the design capacities.

Free-Stressing Length/Unbonded Zone

The bond zone for tiebacks should be developed beyond a 60-degree line projected upward into the backslope material from the base of the wall, and provide for an unbonded (ungrouted) free-stressing length of at least 15 feet for strand tendons and 10 feet for bar tendons. Anchors should be designed and constructed at an inclination of between 10 and 15 degrees below horizontal. The angle should be adjusted as necessary to achieve embedment.

7.3.5 Soldier Piles and Passive Resistance

We anticipate that soldier beams will be constructed by drilling a hole, inserting a steel wide flange beam or reinforcing cage, and placing concrete or other suitable backfill around the beam. The design of the tieback wall should include a check that there is sufficient passive pressure on the embedded portion of the pile to resist kick-out between the lowest row of tiebacks and the base of the excavation relative to the estimated earth pressures. The passive pressuer (Pp) needed to resist kick-out can be estimated using the equation shown on Plates 7a and 7b. A factor of safety of at least 1.5 should be applied to the passive pressure to estimate

PRELIMINARY

the factor of safety against kick-out. Passive resistance should not be used for the upper 1-foot of the soil that is not constrained at the ground surface by slab-on-grade or pavements. For estimating the passive resistance of widely spaced foundation elements (such as piers or piles for a soldier beam wall that are spaced at least 3 pile diameters apart), the passive resistance of the pile can be assumed to act over three times the width of the pile. For drilled-in soldier beams backfilled with lean-mix concrete, the full 3 pile diameters may be used for passive resistance provided the lean-mix concrete has a compressive strength of at least 50 psi.

Solider piles should be embedded at least 10 feet below the base of the excavation in front of the wall. If the base of the excavation is irregular or slopes downward away from the shoring, only the portion of the pile that is embedded below a 10-foot horizontal setback between the nearest edge of the soldier pile and the face of the adjacent slope or excavation should be considered for the pile embedment.

Lagging

Lagging placed between soldier beams can consist of timber or pre-cast concrete. Lagging should be designed to accommodate an estimated lateral earth pressure equal to 0.6 times the design lateral pressure up to a maximum of 400 psf. Timber lagging should be chemically treated and be at least 3 inches thick for clear spans of 7 feet or less, and 4 inches for clear spans greater than 7 feet.

7.3.6 Global Stability

Global stability analysis of a tie-back wall system is an iterative process based on several factors, including soil shear strength, tie-back bonded length and capacity, and soldier pile passive resistance and embedment depth below the wall. We recommend that soldier piles be embedded at least 10 feet below the bottom of wall for initial design. When final tie-back lengths and capacities are determined, final soldier pile embedment depths should be determined through global stability analysis.

7.3.7 Anchor Installation and Testing

Tie-backs can consist of tendons or reinforcing bars that are cast into subhorizontally drilled holes. The contractor should be responsible for providing tie-backs with sufficient strength to provide for the design capacity that is specified. Anchors should be tested to at least 1.5 times the design axial load. The anchors should be designed such that the maximum test load does not exceed 80 percent of the guaranteed ultimate tensile strength of the anchor tendon. Following testing, the anchors should be locked-off at approximately 75 percent of the design load.

Each tie-back anchor should be load tested during construction on the basis of either a performance test or a proof test. Performance testing is used to establish and evaluate procedures for installing production anchors, grouting, confirm design loads, and evaluate suitable bond lengths and grouting to provide the anchor capacity. Performance testing should be performed on initial three anchors prior to production and/or installation, and on at least 5

PRELIMINARY

percent of the remaining production anchors. The geotechnical professional should select the locations for the performance testing.

Once the initial performance tests are completed and the final design lengths are established, the production installation of the ground anchors can proceed. The proof test provides a general evaluation of each of the production anchors by loading the anchor to a test load 1.5 times greater than the design load. A proof test should be performed to evaluate the load capacity of each anchor, and the potential creep of the constructed anchor at the maximum load. The production anchors should be constructed using the methods established from the performance test anchors, and each of the production anchors should be proof tested. The project specifications should provide for the geotechnical professional to observe the testing, and for the contractor to modify the anchor length and methods of construction to achieve the design anchor capacity, if needed. Additional performance tests should be performed if subsurface conditions are encountered during production anchor installation that require modification to the construction methods.

Performance Testing

Prior to performance testing, an alignment load (AL) should be applied to the ground anchor to stabilize the bar and seat the load frame and jacking equipment. The deflection of the anchor under the alignment load will serve as a reference for movement that occurs during testing. Incremental axial loads should be applied to the ground anchor in the following sequence: 0.25, 0.50, 0.75, 1.00, 1.25, and 1.5 times the design load. An unload-reload cycle should be performed at each test-load increment. The unload-reload should consist of incrementally unloading the anchor to the AL, and then reloading the anchor in the same sequence to the next test load. During unloading, the applied load should not fall below the AL.

Each load increment should be held for at least 1 minute, with the exception that a longer creep test should be performed at the highest load increment as recommended herein. The load time should begin when the axial load first reaches the load for the increment being applied. Axial movements should be obtained at 6 seconds, 15 seconds, 30 seconds and 1 minute for each load increment. The residual movement should be measured 1 minute after the final load and/or each load increment has been removed to the AL.

Proof Testing

In addition to the performance testing specified above, we recommend that all of the remaining ground anchors be proof tested. The deflection of the anchor under the alignment load will serve as the reference for movement that occurs during testing. After the alignment load has been applied, the element should be loaded to 0.25, 0.50, 0.75, 1.00 1.25, and 1.5 times the design load. Each load increment should be maintained for at least 1 minute, and the deflection at 6 seconds, 15 seconds, 30 seconds and at the end of 1 minute should be recorded. The residual movement should be measured 1 minute after the final load and/or each load increment has been removed to the AL.

PRELIMINARY

Creep Testing

A short-term creep test should be performed at the maximum test load for performance or proof testing. The maximum test load should be maintained at all times during the entire creep-testing program. The creep test should consist of maintaining the maximum load for at least 10 minutes. Deflection readings should be recorded at 6 seconds, 30 seconds, 1, 3, 5 and 10 minutes. If the difference between the 1-and 10-minute readings is greater than 0.04 inches, the creep test should be continued to 50 minutes. Creep testing should be performed on one of the initial three anchors, and on at least 1 percent of the remaining production anchors. The geotechnical professional should select the locations for the performance testing.

Readings should be taken at 5-minute intervals for the extended test-load period. The difference in deflection between the 5-minute and 50-minute reading should be less than 0.08 inches, and the movements should be plotted versus time to evaluate if the anchor capacity is exceeding the creep limited strength of the anchor. If the creep rate exceeds 0.08 inches per log cycle of time, the duration of the test can be extended to evaluate if the creep rate will diminish to less than 0.08 inches per log cycle.

Ground anchors having deflections greater than 0.08 inches per log cycle of time during creep testing should be replaced using modified construction methods, have secondary grouting performed, or additional anchors should be installed to provide the design capacities; and then be retested.

The contractor should be responsible for all aspects of the construction, provide testing equipment, and make modifications to construction methods and materials as-needed to comply with the specifications. Lock-off or transfer loads should be applied to the anchor subsequent to testing..

7.3.8 Tie-Back Wall Drainage

Recommended apparent earth pressures herein assume walls are fully back drained to prevent the buildup of hydrostatic pressures. Adequate drainage could be provided by means of either weep holes with permeable material installed behind the walls or by means of a system of subdrains.

For the subdrain system, the top of the perforated pipe should be below the bottom of the adjacent slab or grade at the toe of the wall. Drains should consist of a drain rock layer at least 12 inches thick that extends to within 2 feet of the ground surface. Four-inch diameter perforated plastic pipe should be installed (with perforations down) along the base of the walls on a two-inch-thick bed of drain rock. The pipe should be sloped to drain by gravity to a suitable drainage facility. Drain rock should conform to Caltrans specifications for Class 2 permeable material. A more open-graded material, such as 3/4-inch crushed rock, could be used provided the rock is wrapped in a geotextile filter fabric (Mirafi 140 N or equivalent) to reduce the migration of fine-grained soils into the drain rock. Paving or a two-foot-thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration. Drain pipes should outlet to an appropriate drainage facility.

PRELIMINARY

Alternatively, wall back-drainage can be provided by prefabricated drainage material (such as Miradrain 6000 or an approved alternative). The drainage material can be installed on the back (soil) face of the wall and should terminate at a 4-inch diameter perforated plastic pipe surrounded by at least 6 inches of drain rock as defined above.

7.3.9 Grouting Considerations

The capacity of ground anchors will depend on the type of drilling method used, grouting procedures, and subsurface conditions at the location of each anchor. Anchors can be constructed using only tremie-placed grout. Higher anchor capacities can typically be achieved if secondary grouting is performed following the initial construction of the anchor. Secondary grouting typically consists of pumping grout under high pressure into the anchor bond zone to increase the pullout resistance of the anchor.

Centralizers should be provided along the length of the anchor at no more than 10 feet on-center. The centralizers should be manufactured from materials that are not detrimental to the corrosion resistance of the reinforcing element. Closer spacing between centralizers should be provided, as needed, to maintain grout cover and support the anchor.

We recommend that ground anchors that rely on developing frictional resistance along the interface between the walls of the anchor shaft and gravity-placed grout be installed with provisions for secondary grouting. A grout tube should be installed with the anchor that will allow for secondary, high-pressure grouting and/or fracturing to increase the anchor capacity, if needed. The secondary grouting could be used in the event that test results indicate that the primary grouting did not provide at least the recommended capacity. The anchor should be re-tested for capacity after secondary grouting.

Grout used for anchor construction should provide a 28-day compressive strength of at least 4,000 pounds per square inch and having a water-cement ratio of no greater than 0.5. Admixtures that control bleed, improve plasticity, reduce water content, and retard set and shrinkage can be used in the grout. Expansive admixtures should only be used for filling encapsulations and anchorage trumpets. Expansive admixtures should not be permitted in the length of the bond zone.

Grout should be placed from the lowest point of the drill hole by tremie or other injection methods such that water and debris in the drill hole is displaced by the grout during placement. As the grout is placed, the tremie pipe should be withdrawn such that a sufficient head of grout is maintained over the end of the pipe, and that voids from removal of the grout pipe are being filled with grout. The second stage of grouting used to fill the void between the bond zone and the head of the anchor should not be performed until after testing of the anchor and lock-off are completed. The grout should be placed from the bottom of the hole, and be overfilled with grout until fresh, non-contaminated grout surfaces at the top of the tieback.

PRELIMINARY

7.3.10 Corrosion Protection

To evaluate the corrosion potential of the subsurface soils at the sites, six samples were transported to the analytical laboratory for chloride and soluble sulfate content, pH, and resistivity testing. Samples were tested from estimated tie-back bonded zones in the embankment fill and upper alluvium. Corrosion testing was performed by Cooper Testing Laboratory of Palo Alto, and CERCO Analytical of Concord, California in accordance with California Test Method 643. Preliminary corrosivity test results are presented in Table 6 and in Appendix B.

Table 6. Results of Preliminary Corrosivity Testing

Boring No.	Soil Type	Depth (feet)	Chloride (ppm)	Sulfate (ppm)	pH	Resistivity (ohm-cm)
B-01	Well-graded GRAVEL with clay and sand (GW-GC)	15.5	<15	430	7.7	2,100
B-02	Sandy Lean CLAY with gravel (CL)	25	<15	150	7.7	1,600
B-03	Clayey GRAVEL with sand (GC)	15	<15	190	8.0	1,900
B-04	Sandy Lean CLAY with gravel (CL)	15.5	<15	64	8.1	1,600
B-05	Clayey SAND with gravel (SC)	15.5	<15	670	7.6	840
B-05	Sandy Lean CLAY (CL)	35	<15	16	7.6	2,165

Note: ppm = parts per million

According to the current Caltrans Corrosion Guidelines, a corrosive site is defined as a site where "…chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less." In addition, Caltrans (2012) suggests soil materials can be considered to have a low propensity for corrosion if the resistivity of the soil is greater than 1,000 ohm-cm. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor.

Based on laboratory test results of the soil samples tested, in general the near-surface soils in the project vicinity do not represent a corrosive environment. However, the test results from B-05 indicate the embankment fill materials are moderately corrosive. Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration and further testing may be warranted during construction to confirm soil corrosivity within the design tie-back zones.

Because the tieback anchors will be permanent, we recommend the anchors be provided with factory-provided corrosion protection. We recommend the anchors have a double corrosion protection system consisting of an inner core of grout and an outer smooth (in the stressing length) or corrugated (in the bonded zone) PVC sheath.

PRELIMINARY

7.3.11 Tie-Back Construction Considerations

Tie-back-supported walls are typically constructed from the top down to help limit the exposed height of the wall during excavation of the soil, and avoid the need for a separate temporary shoring system. Construction of the walls should be performed in accordance with Caltrans Standard Specifications Section 46.1, "Ground Anchors and Soil Nails". The initial excavation lift is typically 3 to 4 feet. If excavated lifts exceed 5 feet or the excavated face will be exposed for long periods of time, stability testing should be performed in accordance with Section 19-301A(3)(b) of the Standard Specifications. Movement of the walls should be monitored during construction, and any signs of bulging or excessive deformation of the excavated soil face should be addressed immediately.

The tie-back drilling method should be left to the discretion of the contractor, but should include procedures for flushing and/or cleaning the drill hole prior to placement of grout. The drilling method should be reviewed in advance by the project geotechnical engineer. The geotechnical engineer should observe the tie-back construction for subsurface conditions and confirm that the tie-back is constructed according to the geotechnical design recommendations.

7.4 DESIGN OF CONCRETE GRAVITY RETAINING WALLS

7.4.1 General

The gravity cantilevered retaining wall must be designed to resist lateral earth pressures and any additional lateral loads caused by surcharging or seismic pressure. The following recommendations were developed based on the dense to very dense clayey sand and gravel, and very stiff sandy lean clay soils encountered at the site. Retaining wall backfill less than 5 feet deep should be compacted using light compaction equipment to at least 90 percent of the soil's maximum dry density. Backfill greater than 5 feet deep should be entirely compacted to at least 95 percent of the soil's maximum dry density. If heavy compaction equipment is used, the wall should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

7.4.2 Lateral Earth Pressures

Plate 8 presents our recommended lateral earth pressure distributions for a gravity cantilever retaining wall and sloping ground conditions (3h:1v or flatter). The gravity retaining wall will be free to move/rotate, and therefore should be designed for active earth pressure. We recommend the gravity retaining wall be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf), assuming a 3h:1v sloping backfill. In addition, a uniform seismic pressure of 30H (psf) should be applied to the entire wall height, where H is the height of backfill above the top of the wall footing.

The gravity retaining wall should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load. The design surcharge should include the anticipated surcharge caused by construction equipment.

PRELIMINARY

7.4.3 Wall Drainage

The recommended lateral pressures for the gravity retaining wall assume the wall is fully back drained to prevent the buildup of hydrostatic pressures. Refer to Section 7.3.8 of this report for recommended wall drainage alternatives.

7.4.4 Support of Gravity Wall

We anticipate gravity wall footings will bear entirely on properly compacted embankment fill or on firm, undisturbed native soils. However, if unexpected, undocumented hospital grade fill materials are encountered within the limits of excavation during grading, the undocumented fill should be reworked and placed as engineered fill according to the requirements in Section 7.5 of this report, or the footings should be deepened to extend through the undocumented fill layer. All footing excavations should be observed by our representative prior to placing reinforcement. Where possible, footings should not bear within a utility easement.

7.4.5 Coefficient of Friction

Long-term resistance to lateral loads may be provided by friction along the base of continuous spread footings. A friction coefficient of 0.30 times the dead load may be used to evaluate the allowable frictional resistance along the bottom of footings bearing on subgrades prepared as discussed above.

7.4.6 Passive Resistance

In addition to friction, a long-term passive pressure equal to an equivalent fluid pressure of 300 pounds per cubic foot (pcf) can be used for lateral load resistance against the footing where the foundation is poured neat against engineered fill or undisturbed native material. Unless confined by concrete or pavement, the passive resistance of the uppermost one foot of soil should be neglected.

7.5 SITE PREPARATION AND GRADING

7.5.1 Site Preparation

The site should be cleared of all obstructions, including concrete, asphalt pavement, buried foundations, slabs, utility lines, trees and associated root systems, and debris. Removed concrete, asphalt concrete, and baserock may be reused as fill, provided the material is broken up to meet the requirements in Section 7.5.4 - Engineered Fill Materials. Holes resulting from the removal of root systems of larger trees could extend to depths of 3 feet, and laterally to the drip line of each tree. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with suitable material compacted to the requirements in Section 7.5.5 - Fill Placement and Compaction. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

PRELIMINARY

After clearing, portions of the site containing surface vegetation or organic laden topsoil should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by the Geotechnical Engineer at the time of construction. Stripped materials should be removed from the site, or stockpiled for later use in landscaping, if approved by the owner.

7.5.2 Subgrade Preparation

Following excavation to the required grades, soil subgrades in areas to receive engineered fill (as defined in Section 7.5.4) or slabs-on-grade should be scarified to a depth of at least 6 inches, moisture conditioned to at least slightly above optimum moisture content, and compacted to at least 95 percent of the soil's maximum dry density. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. If the subgrade is allowed to become dry, it should be moisture conditioned to eliminate shrinkage cracks.

In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry. Conversely, it may require that scarification and aeration be performed in any soils that are too wet. There was no indication of weak or unstable soils in our borings; however, after the removal of existing pavements, the exposed subgrade materials may be above their optimum moisture content, and may be locally unstable. If areas of unstable soils are encountered at the time of construction, the geotechnical engineer should review conditions in the field and provide recommendations for stabilization procedures.

7.5.3 Engineered Fill Materials

All fill placed at the site should consist of engineered fill meeting the requirements presented in this report, except for landscaping materials which are placed on level ground. On-site soil below the stripped layer and having an organic content of less than 3 percent by volume can be used as fill where needed except where "non-expansive" import is required beneath slabs-on-grade. All engineered fill placed at the site, including on-site soils, should not contain rocks or lumps larger than 4 inches in greatest dimension and contain no more than 15 percent larger than 2.5 inches.

"Non-expansive" fill should be predominantly granular, have an organic content of less than 3 percent by volume, should have a liquid limit less than 40 percent, have a plasticity index not exceeding 15, and should be free of environmental contaminants or debris. Imported fill should consist of "non-expansive" fill, and satisfy California Environmental Protection Agency's Department of Toxic Substances Control (Cal EPA DTSC) guidelines for clean, import fill material.

7.5.4 Fill Placement and Compaction

Engineered fill should be compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM Designation D1557 (latest edition). The upper 6 inches of

PRELIMINARY

subgrade soils beneath pavements should be compacted to at least 95 percent of the soil's maximum dry density. Fill material should be spread and compacted in lifts not exceeding 8 inches in pre-compacted thickness. Moisture content of the natural on-site moderately expansive clayey soils reused as fill should be 2 to 3 percent above the optimum moisture content for the soil at the time of compaction. In order to achieve satisfactory compaction of the subgrade and fill materials, the water content may need to be adjusted at the time of construction. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet.

7.5.5 Surface Drainage

Positive surface gradients should be provided adjacent to the retaining walls to direct surface water toward suitable discharge facilities. Water from roof downspouts should be carried away from the buildings in solid pipes and discharged into suitable drainage facilities. Surface water should not be allowed to pond adjacent to the retaining walls or on pavements and sidewalks.

7.6 UTILITY CONSTRUCTION

7.6.1 General

Utility trench excavations and backfill should conform to Caltrans Standard Specifications, Greenbook Standards, and/or any local jurisdictional agency requirements. Excavations more than 5-feet deep should be excavated with sloping sidewalls, braced, shored, or shielded in accordance with federal and state standards and safe construction practice. Shoring and bracing of the trench sidewalls will be required in accordance with OSHA regulations. Loose, soft fill and alluvial soils may be encountered in trench excavations. The contractor will be responsible for design and implementation of shoring systems and safe working conditions.

Excavation bottoms are anticipated to be within firm embankment fill or alluvial materials; however, if wet, soft, or yielding conditions are encountered, the bottom of the trench excavation should be stabilized prior to placement of pipe bedding so that the trench subgrade is firm and unyielding.

Compacted fill materials for the pipeline will consist of pipe-zone materials and trench backfill materials. The following subsections provide our suggested material types for pipe zone and trench backfill materials.

7.6.2 Pipe Zone Materials

Pipe zone materials are herein defined as those select earth materials used as pipeline bedding and shading, and as structure bedding. Pipe zone materials should consist of clean sand with a minimum Sand Equivalent (SE) of 30 or crushed angular gravel to facilitate placement and achieve uniform support for pipe. Gravel should conform to gradation

PRELIMINARY

requirements in Caltrans Standard Specifications, or equivalent. Pipe zone materials should extend from at least 6 inches below the pipe to 12 inches above the crown.

Pipe zone materials should be properly placed and mechanically compacted in order to achieve a minimum of 90 percent relative compaction as determined by standard test method ASTM D1557. Backfill should be placed in loose lifts no greater than 6-inches thick and mechanically compacted. Gravel, if used for pipe zone backfill, should be placed in 6-inch lifts and mechanically densified/vibrated.

The trench width should be sufficient to allow compaction equipment to operate between the pipe springline and trench wall. We recommend that the trench be a least 2 feet wider than the pipe on each side to allow for shoring installation and compaction of the backfill. Jetting or flooding of pipe-zone materials should not be allowed.

7.6.3 Trench Backfill

Trenches should be backfilled with materials satisfying the criteria described above for fill, placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts may be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Onsite soil used for trench backfill should be compacted to at least 90 percent of the soil's maximum dry density by mechanical means only (jetting should not be permitted). Sand can be used for trench backfill if the sand is compacted to at least 95 percent of the soil's maximum dry density and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill below slab and pavements should be compacted to at least 95 percent of the soil's maximum dry density.

7.7 PAVEMENTS

We have developed the following pavement design recommendations based on an R-value of 5, based on the critical expansion potential of the clayey soils at the site and R-value tests performed on bulk samples collected during our previous studies. We developed the following alternative preliminary pavement sections based on Chapter 630 of the State of California Department of Transportation Highway Design Manual and assumed traffic indices. Pavement designs for pavement life of 11 to 20 years are presented in Table 7.

PRELIMINARY

Table 7. Recommended Pavement Design Alternatives

Location	Anticipated Pavement Life (years)	Pavement Components		Total Thickness (inches)
		Asphalt Concrete (inches)	*Caltrans Class 2 Aggregate Base (inches)*	
Automobile Parking (T.I. = 4.5)	20	2.5	9.0	11.5
Automobile Access (T.I. = 5.5)	20	3.0	12.0	15.0
Heavy Truck Access (T.I. = 6.5)	20	3.5	15.0	18.5

The traffic indices used in our design were established assuming a typical mix of automobile and "delivery or garbage" truck type of use in the proposed development after construction has been completed. However, if the pavements are planned to be placed prior to, or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. Therefore, if the pavement sections will be used for construction access, our firm should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use. If requested, we could provide recommendations for a phased placement of the asphalt concrete to minimize the potential for mechanical scars caused by construction traffic on the finished grade.

The traffic indices should provide the indicated pavement lives with only a normal amount of pavement maintenance. Selection of the design traffic parameters, however, was based on engineering judgment, and not on an equivalent wheel load analysis developed from a traffic study or furnished to us.

In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils and trench backfill should be protected against saturation. Planned concrete sidewalks, driveways, and curb and gutters should be supported directly on the properly compacted native soils. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas and the pavement section. In addition, a compacted impermeable soil plug should be constructed within any lateral or other trench backfill that passes beneath the curb and gutter and under the adjoining pavement. In addition, water should never be allowed to pond behind the curb and gutter during or after the completion of construction.

Aggregate Base for use in flexible pavements should conform to Caltrans Standard Specification Section 26-1.02A for Class 2 Aggregate Base. The Aggregate Base used in the pavement sections should be compacted to 95 percent of the soil's maximum dry density (ASTM D1557) and should be firm and unyielding.

PRELIMINARY

8.0 INVESTIGATION LIMITATIONS

This report has been prepared for specific application to the project as described herein. In our opinion, the findings, conclusions, professional opinions, and recommendations presented herein were prepared in accordance with generally accepted geotechnical engineering practice of the project region.

Although information contained in this report may be of some use for other purposes, it may not contain sufficient information for other parties or uses. If any changes are made to the project as described in this report, the conclusions and recommendations in this report shall not be considered valid unless the changes are reviewed and the conclusions and recommendations of this report are modified or validated in writing by Fugro.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

In performing our professional services, in our opinion, we have used generally accepted geologic and geotechnical engineering principles and have applied that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report.

Users of this report should recognize that the construction process is an integral design component with respect to the geotechnical aspects of a project, and that geotechnical engineering is inexact due to the variability of natural and man-induced processes, which can produce unanticipated or changed conditions. Proper geotechnical observation and testing during construction, thus, are imperative in allowing the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Fugro be retained during retaining wall construction and site grading to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions, or methods of construction, differ from those anticipated.

PRELIMINARY

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PRELIMINARY

APPENDIX A
FIELD EXPLORATIONS

GENERAL

The field exploration program consisted of a surface reconnaissance and a subsurface exploration program. As a part of the geotechnical exploration for the project, six exploratory borings, designated B-01 through B-06, and eleven Cone Penetration Test (CPT) soundings, designated C-01 through C-11, were conducted between August 25 and September 6, 2013, to a maximum depth of about 81.5 feet bgs.

Upon completion of our field explorations, borings and CPTs were backfilled with neat cement grout and patched at the ground surface to match existing conditions. Boring and CPT logs, as well as keys for the soil classification (Plates A-1 through A-19), are included as part of this appendix. Boring and CPT logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times. Approximate exploration locations for the current study, as well as relevant historical borings, are shown on Plate 2 - Exploration Location Plan.

HOLLOW-STEM-AUGER AND MUD ROTARY WASH BORINGS

Borings were drilled with a truck-mounted CME-55 drill rig equipped with hollow-stem-augers and mud rotary wash drilling equipment provided by Pitcher Drilling Co. of East Palo Alto, California. Based on the information provided by the driller, hammer energy ratio of the drill rig used was approximately 86 percent.

Representative soil samples were collected using one of three sampler types:

- Modified California (3.0–inch-OD, 2.38-inch-ID) split-barrel,
- Standard Penetration Test (2.0–inch-OD, 1.375-inch-ID) split-barrel,
- Thin-walled Shelby tube (3.0–inch-OD), or
- Pitcher barrel Sampler (3.0-inch OD, 2-inch I.D.)

Boring logs presented in Appendix A provide details regarding the types of samplers used at each location, percent recovery, and soil description. Samples were conducted at approximately 5-foot intervals or where strata changes were observed in order to estimate the relative densities and obtain samples for classification purposes. SPTs were performed in accordance with American Society for Testing of Materials (ASTM) Standard Test Method (D1586) and involved driving a thick-walled tube sampler into the bottom of the boring with successive blows of a 140-lb hammer falling freely through 30 inches. The sampler is driven through three 6-inch intervals or until refusal. The number of blows required to penetrate each interval is recorded and the aggregate number of blows to drive the sampler over the final 12 inches is termed the "N-value". For an approximate comparison to the smaller diameter SPT N-values, the blow counts recorded during the driving of Modified California samplers were multiplied by a factor of 0.63. The N-values on the Log of Test Boring (LOTB) sheets represent

PRELIMINARY

uncorrected values. Shelby tubes were advanced using direct push methods to obtain relatively undisturbed samples in clay layers.

Disturbed SPT samples collected during drilling were retained in sealed, labeled plastic bags and Modified California liner samples were sealed onsite and placed in plastic containers. Relatively undisturbed Shelby tubes were sealed onsite. During the drilling program, samples were transported on a regular basis to Fugro's laboratory in Oakland, California. Representative samples were selected for laboratory testing to assist with sample descriptions and determination of engineering properties of the soils encountered.

CONE PENETRATION TESTING

CPT soundings were advanced using Fugro in-house equipment equipped with a hydraulic press mounted inside a 30-ton truck to advance the probe utilizing the self-weight. A Fugro engineer monitored the real time CPT data at a display station inside the truck and determined optimal locations to perform pore pressure dissipation tests. The results of those tests helped in estimating the depth to groundwater.

CPT logs show tip and sleeve resistances, friction ratio, pore pressure, soil behavior type (SBT), and correlated friction angle and undrained shear strength values. Friction ratio is defined as the resistance of the cone sleeve divided by the resistance of the cone tip reported as a percentage. Plate A-8, Key to CPT Logs defines the SBT correlation as suggested by Robertson and Wride (1990). Pore pressure dissipation test results accompany their respective sounding logs on independent plates.

PRELIMINARY

APPENDIX B
LABORATORY DATA

PRELIMINARY

APPENDIX B
LABORATORY DATA

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

In-situ water content was determined on 72 samples of the materials recovered from the borings in accordance with ASTM Test Designation D-2216. This water content is recorded on the boring logs at the appropriate sample depths.

Dry density determinations were performed on 43 samples of the subsurface soils to evaluate its physical properties. The results of this test are shown on the boring logs at the appropriate sample depths.

Gradation tests were performed on 11 samples of the subsurface soils to assist the classification of the sandy soils and to determine the grain size distriubtion. The results of these tests are presented on Plate 2 in this Appendix.

The percent passing the No. 200 sieve was measured on four of the subsurface soil samples to aid in the classification of these soils. The test was performed in accordance with ASTM Designation D-1140. The result of the test is shown on the boring logs at the appropriate sample depths.

Atterberg limit determinations were performed on 19 of the subsurface soil samples to determine the range of water content over which these materials exhibit plasticity. The Atterberg limits were determined in accordance with ASTM Test Designations D-428 and D-424. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of the test are presented on Plates B-1a through B-1c and Plates B-3a through B-3c in this Appendix, and on the log of the boring at the appropriate sample depth.

Triaxial (Unconsolidated-Undrained) compression tests were performed on 11 relatively undisturbed samples of the subsurface soils to evaluate the undrained shear strengths of these materials. Tests were performed in accordance with ASTM Test Designation D-2850 on a sample having a diameter of 2.4 inches and a height-to-diameter ratio of at least two. The results of this test are presented on Plates B-4a through B-4k in this Appendix and on the boring logs at the appropriate sample depths.

Direct shear tests were performed on two multistage select driven ring samples to evaluate the shear strength of earth materials. Tests were performed in accordance with standard test method ASTM D-3080. Summary plots of the direct shear data are presented on Plates B-5b and B-5b in this Appendix.

A consolidation test was performed on one dual-odometer select driven ring sample of fine-grained earth material. The test was conducted in general accordance with standard test

PRELIMINARY

method ASTM D2435, Standard Test Method for One-Dimensional Consolidation Properties of Soils. Consolidation test results are presented on Plate B-6 in this Appendix.

Corrosion testing was performed by Cooper Testing Laboratory of Palo Alto, and CERCO Analytical of Concord, California, on six soil samples at various depths. Tests conducted included chloride content, sulfate content, pH, and resistivity. Corrosion tests results are presented on Plates B-1a through B-1c in this Appendix.

PRELIMINARY

APPENDIX C
LIQUEFACTION EVALUATION RESULTS

PRELIMINARY

APPENDIX D
CALCULATIONS

PRELIMINARY

PLATES

PRELIMINARY

APPENDIX A
FIELD EXPLORATIONS

PRELIMINARY