GEOTECHNICAL INVESTIGATION, SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN, CHULA VISTA, CALIFORNIA

Prepared For:

SHARP HEALTHCARE
8695 Spectrum Center Boulevard
San Diego, California 92123

Project No. 603541-002

July 18, 2013

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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To: Sharp HealthCare
8695 Spectrum Center Boulevard
San Diego, California 92123

Attention: Ms. Pat Nemeth

Subject: Geotechnical Investigation, Sharp Chula Vista Medical Center Master Plan, Chula Vista, California

In accordance with your request and authorization, we have conducted a geotechnical study for the proposed Master Plan of the Sharp Chula Vista Medical Center located in Chula Vista, California. Based on the results of our study, it is our opinion that the proposed Master Plan of the site is feasible provided the geotechnical recommendations contained in this report are implemented during design and construction. In particular, mitigation of existing undocumented fill will be necessary. Specifically, undocumented fill having a thickness up to approximately 15 feet is located within the proposed East Patient Care Building footprint and up to approximately 22 feet within the proposed Central Plant footprint. This report provides recommendations for the mitigation of the compressible materials relative to the proposed improvements, and provides a summary of the current investigation and general geotechnical conclusions and recommendations for the Master Plan.

If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

PROFESSIONAL GEOLOGIST
ROBERT C. STROH
NO. 2099
CERTIFIED
ENGINEERING
GEOLOGIST
EXP. 10/31/13
STATE OF CALIFORNIA

Robert C. Stroh, CEG 2099
Senior Engineering Geologist

Sean Colorado, GE 2507
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
SEAN A. COLORADO
No. 2507
Exp: 12/31/13
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: (1) Addressee
(1) McCarthy Building Companies Inc., Attention: Mr. Randy Burns
(1) NTD Architecture, Inc., Attention: Mr. Wayne Hunter
(1) KPFF Consulting Engineers, Attention: Mr. Aaron Reynolds

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1.0 INTRODUCTION

In accordance with your authorization we have performed a geotechnical investigation of the site to assist in the preparation of the Sharp Chula Vista Medical Center Master Plan (Figure 1). This report presents our findings, conclusions and recommendations for the site with regard to geotechnical conditions.

1.1 Purpose and Scope

Specifically, the purpose of our investigation was to identify and evaluate the geologic hazards and significant geotechnical conditions present at the site in order to provide geotechnical recommendations for the proposed structures and associated site improvements. Taking into consideration previously completed geotechnical work at the site, our scope of services included:

- Prior to our subsurface exploration, we notified Underground Service Alert (USA) to screen the proposed exploration locations for the presence of subsurface utilities.
- In accordance with the County of San Diego Department of Environmental Health (DEH) requirements, we obtained boring permit waivers for our subsurface excavations.
- We performed a subsurface evaluation consisting of drilling, logging, and sampling of twenty (20) exploratory borings. At the completion of drilling, the borings were backfilled with bentonite grout (per DEH standards) and patched as appropriate. Drill cuttings were stored temporarily in 55-gallon drums on the site and were later disposed of at a proper disposal facility by an approved hauling subcontractor.
- We conducted geotechnical laboratory testing on selected soil samples. We performed lab testing consisting of dry unit weights, moisture contents, direct shear, grain size, plasticity, expansion, R-value, sand equivalent, and corrosivity tests including - minimum electrical resistivity, pH, and water soluble sulfates and chlorides content tests.
- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations with respect to the proposed geotechnical design, site grading and general construction considerations. Specifically, this report provides the following:

- Vicinity map and site plan showing approximate locations of soil borings;
- Logs of soil borings, and laboratory test results;
- Discussion of the site and subsurface conditions;
- Discussion of field exploration methods and laboratory test procedures;
- Discussion of faulting and seismicity in the region;
- Discussion of potential geologic hazards, which may impact the site;
- Site Classification type and Site Coefficients based on 2010 California Building Code (CBC). In addition, for planning purposes, we have also provided seismic parameters in accordance with the 2012 International Building Code (IBC).
- Discussion of anticipated excavation conditions;
- Soil parameters and recommendations for design of temporary shoring;
- Discussion of groundwater conditions, need for temporary dewatering, if any, and preliminary dewatering information, if any;
- Guidelines for earthwork construction, including recommendations for site preparation, fill and backfill placement, and compaction;
- Discussion of the possible foundation types;
- Soil parameters for foundation design;
- Estimated foundation settlements;
- Lateral earth pressures for design of permanent basement walls; and
- A preliminary screening of the soil properties affecting corrosion of concrete and steel;
- Preliminary pavement design;

1.2 Site Location and Description

The Master Plan area is located at 751 Medical Center Court (APN 641-010-28) and is currently occupied with the existing hospital, subsidiary structures, parking deck structure, and other site improvements (Figure 1). Specifically, the hospital is located in the central portion of the site and consists of the Main Tower, the Main Hospital, the West Tower, Administration, the O.R. Addition, and the MRI addition (Figure 2). A parking deck is located west of the hospital and surface paved parking lots are located easterly and south easterly of the hospital. A helicopter pad is located in the upper portion of the property in the northeastern corner of the site. To the south of the hospital is the Birch Patrick Convalescent Facility. Other medical office buildings are located to the east of the hospital parking lot and across Medical Center Court to the southwest.

With regard to site topography, the upper portion of property is situated along the top of a hill at a topographic elevation of approximately 460 feet above mean sea level (msl). The topographically lowest portion of the site is located in the eastern portion of the site at the toe of the fill slope with an elevation of approximately 390 feet msl. The lowest western portion of the site, west of the parking deck area, is approximately 405 feet msl. In addition, another low area is located just east of the Birch Patrick Convalescent Facility within the existing surface pavement parking area, at approximately 445 feet msl.

The site is bound along the north by a moderately sloping descending cut slope. Based on our review of the topographic data the cut slope is approximately 33-feet high at an inclination of approximately 2.2:1 (horizontal:vertical). Along the eastern portion of the site a descending natural slope that transitions into a fill slope is also present having a height of approximately 40 feet at an inclination of approximately 4:1 (horizontal:vertical).

Total topographic relief across the property is approximately 60 feet, with an average elevation difference across the portion of the campus proposed for improvements at approximately 30 feet. In general, the overall property is located on a topographic hill and descends southward and westward toward existing medical office facilities and the Birch Patrick Convalescent Facility.

Site Coordinates:
Latitude: 32.6191º N
Longitude: 117.0228º W

1.3 Project Description

Based on our review of conceptual plans by NTD Healthcare, Cuningham Group, dated 2013, we understand that new site development associated with the Master Plan consists of generally three phases (Figure 2 and Plate 1).

Phase I – Make-Ready Phase:

The Make-Ready phase of the Master Plan is proposed to consist of the construction of a new 40,000 square-foot, six level parking structure located along the eastern boundary of the Master Plan area. In addition, a proposed new loop access road and utility corridor is proposed along the periphery of the Master Plan area. To accommodate employee and customer parking during the Make-Ready phase and construction of the parking structure, two temporary surface parking lots, located southwest of the Master Plan area, are proposed off-site, and one surface parking lot is proposed in the southwestern portion of the Master Plan area.

Phase II – New East Patient Care Building

Phase II of the Master Plan includes the proposed construction of a new East Patient Care Building located adjacent to the current surgery on the east side of the existing East Tower. The new building is proposed to consist of 4 floors of 36 bed nursing units (144 beds), expansion of the surgery area which will be attached to the existing surgery, and the expansion of kitchen facilities which will be attached to the existing kitchen. Also proposed is a new Central Plant with chillers located southeast of the new East Patient Care Building. Although, not indicated on the conceptual plans nor included in the scope of this report, we also understand that the Main Hospital (East Podium) is also intended to be upgraded to a Structural Performance Category 5 (SPC-5), as part of the Master Plan.

Phase III – Future West Patient Care Building

Although not included in the scope of this report, the Phase III portion of the Master Plan includes long term planning to the year 2030 and a possible future West Patient Care Building located in the location of the existing parking deck in the northwestern portion of the hospital campus.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

The subsurface exploration performed for this geotechnical investigation consisted of the excavation, logging, and sampling of twenty (20) exploratory hollow-stem borings (Borings B-1 through B-20). The approximate locations of the exploration borings are shown on Figure 2 and Plate 1. The purpose of the borings was to investigate the underlying stratigraphy, physical characteristics, and specific engineering properties of the soils within the area of the proposed improvements. In addition we have also plotted the locations of borings from a Woodward-Clyde study dated April 25, 1989, covering the northeastern portion of the site.

2.1 Exploratory Borings

Borings were excavated to depths between approximately 4.5 feet to 101 feet below the existing ground surface (bgs). The boring explorations were generally performed using a heavy duty truck mounted hollow-stem auger drill rig, with 8-inch diameter continuous flight auger. During the exploration operations, a Certified Engineering Geologist from our firm prepared geologic logs and collected bulk and relatively undisturbed samples for laboratory testing and evaluation. After logging, the excavations were backfilled with bentonite grout and patched where appropriate. In addition for reference, we have included boring logs from Woodward-Clyde dated April 25, 1989, covering the northeastern portion of the site. The boring logs are provided in Appendix B.

2.2 Exploratory Trenches

Leighton (2013) previously excavated six trenches to provide coverage for potential faulting within portions of the Master Plan area. The trenches totaled approximately 1,100 lineal feet. Trench depths ranged between 7 and 15 feet with an average depth of approximately 7 feet. In addition, two additional fault studies have been completed at the site. Specifically, the existing Main Hospital facility was relocated to a position where minor faults did not transect the new facility footprint (Woodward-Gizenski & Associates, 1973), and a Geocon (1998) study indicated the presence of minor faults located in the southeastern parking area west of the existing medical office building (MOB) prompting relocation of that new MOB facility to avoid the mapped minor faults. The locations of these previously completed trenches are depicted in Leighton (2013).

2.3 Previous Exploration

Previous geotechnical reports have been performed within the site area and for nearby parcels to the north and southwest of the subject site. The following reports (ordered chronologically) were reviewed as part of our background study for the project:

- Leighton and Associates, 2008, Fault Hazard Study, Proposed Senior Care Campus at Vista Hill, 730 Medical Center Court, Chula Vista, California, dated June 23.
- URS, 2006, Updated Geotechnical Evaluation, Sharp Chula Vista Medical Center, Chula Vista, California, dated August 10, revised February 8, 2007
- Geocon, 1998, Geotechnical Investigation, Chula Vista Medical Plaza Medical Office Building, Chula Vista, California, dated November 19.
- Leighton and Associates, 1996, Evaluation of Faulting and Seismicity, Proposed Veteran's Home, Chula Vista, California, dated July 2.
- Woodward-Clyde, 1989, Geotechnical Investigation for the Proposed Additions to the Main Hospital and Overhead Parking Deck, Community Hospital of Chula Vista, Chula Vista, California, dated April 25.
- Robert Prater Associates, 1988, Fault Location Study, Vista Hill Hospital Expansion, RTC, CDU, and Support Buildings, Chula Vista, California, dated September 21.
- Robert Prater Associates, 1988, Radiocarbon Dating Analysis, Vista Hill Hospital Expansion, RTC, CDU, and Support Buildings, Chula Vista, California, dated October 20.
- Woodward-Clyde Consultants, 1986, Fault and Geologic Hazards Investigation, Proposed Vista Hill Hospital Expansion, San Diego County, California, dated September 2.
- Woodward-Clyde, 1984, Geotechnical Investigation for the Proposed South Bay Community Convalescent Hospital of Chula Vista, California, dated April 20.
- Woodward-Gizenski & Associates, 1973, Additional Engineering and Geological Study, General Hospital Facility, Community Hospital of Chula Vista, California, dated March 15.

Our review of the consultant reports referenced above, along with our review of available geologic literature, indicates that the general site area is transected by northeasterly trending minor faults. In addition, our review indicates that the site has localized fill within the northwestern and eastern portions of the site with thicknesses on the order of up to 35 feet.

2.4 Geotechnical Laboratory Testing

Laboratory testing performed on soil samples representative of on-site soils obtained during the recent subsurface exploration included tests of moisture and density, shear strength, grain size, plasticity, maximum density and optimum moisture content, R-value, and a screening geochemical analysis for corrosion. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C. In-situ moisture and density test results are provided on the boring logs (Appendix B). In addition for reference, we have included laboratory testing from Woodward-Clyde dated April 25, 1989, covering the northwestern portion of the site.

3.0 SUMMARY OF GEOLOGIC CONDITIONS

3.1 Geologic and Tectonic Setting

The site is located in the coastal section of the Peninsular Range Province, a geomorphic province with a long and active geologic history throughout Southern California (Norris and Webb, 1990). Throughout the last 54 million years, the area known as the "San Diego Embayment" has undergone several episodes of marine inundation and subsequent marine regression, resulting in the deposition of a thick sequence of marine and nonmarine sedimentary rocks (Figure 3) on the basement rock of the Southern California batholith (Kennedy and Tan, 2008).

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest (Jennings, 2010). Several of these faults are major active faults. The Whittier-Elsinore, San Jacinto, and San Andreas faults are major active fault systems located northeast of the study area and the Agua Blanca-Coronado Bank and San Clemente faults are active faults located west of the project area (Figure 4). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.

3.2 Local Geologic Setting

During Eocene time, sediments located east of the site were eroded and then deposited in a westerly direction within deep-water fan and delta environments, while uplift of basement materials to the west resulted in deposition of coarse-grained sediments eastward. Simultaneously, additional uplift along the east then resulted in continued deposition of alluvial fan deposits westward. The site is located near the western limits of a broad structural trough formed by downwarping and normal faulting along the Rose Canyon fault system and the La Nacion Fault Zone (LNFZ) see Figure 5.

Gradual emergence of the region from the sea occurred in Pleistocene time, and numerous wave-cut platforms, most of which were covered by relatively thin marine and nonmarine terrace deposits, formed as the sea receded from the land. Specifically, the site is located in an area where deep-water fan and delta environments have now been exposed due to continued uplift, faulting and erosion. Accelerated fluvial erosion during periods of heavy rainfall, coupled with the lowering of the base sea level during Quaternary times, resulted in the rolling hills, mesas, and deeply incised canyons which characterize the landforms we see in the general site area today

3.3 Site-Specific Geology

Based on the site specific subsurface exploration, and our review of pertinent geologic literature and maps, the site is generally underlain by a thin layer of undifferentiated fill, topsoil, colluvium, pedogenic soil horizons, Oligocene-age Otay Formation and Pliocene-age San Diego Formation. A brief generalized description of each of these units as encountered in the exploration borings are presented below. Detailed descriptions are presented on the exploration boring logs (Appendix B). The lateral and vertical extent of the geology underlying the site are depicted on Plates 1 and 2.

3.3.1 Undocumented Fill (Afu)

Fill soils were placed during the initial mass grading of the site in the 1970s, and later in the 1980s and 1990s. Where fills are generally less than 5 feet in thickness they are not depicted on the Geotechnical Map (Plate 1). Fills deeper than 5 feet are located in the northwestern portion of the site, northwest of the parking deck, the northeastern portion of the site parking lot and as retaining wall backfill. As encountered in the borings, the fill soils generally consisted of brown to dark brown, dry to moist, loose to medium dense, silty sands.

3.3.2 Topsoil and Colluvium (not mapped)

Although not encountered in our boring explorations, localized occurrences of these units were noted in our fault exploration trenching (Leighton, 2013). As encountered, these units were generally light brown and ranged to dark brown, dry to wet, loose to medium dense, porous, silty sands with abundant rootlets. Generally the contact of either the topsoil or colluvial units with the underlying bedrock units was sharp and irregular in character. Thicknesses for the unit ranged from less than a foot to up to 5 feet. Based on the generally brown to light colors, lack of consolidation and cementation.

3.3.3 Very Old Paralic Deposits (Qvop)

As encountered in our boring excavations, these deposits generally consisted of light to medium brown silty sandstone with scattered interbedded cobble-gravel conglomerate and coarse-grained sandstone, dry to damp, very dense. Locally light reddish brown zones were present. This unit was encountered in the upper portions of the site only near the helicopter pad (Boring B-19). The Very Old Paralic Deposits are middle to early Pleistocene in age and correlate to the Lindavista Formation.

3.3.4 San Diego Formation (Tsdss)

As encountered in our boring excavations, the San Diego Formation generally consisted of fine- to locally medium-grained sandstones. The sandstones encountered during our study were generally light brown to light olive brown, damp to moist, dense to very dense, slightly cemented and friable to very friable. Typically, the unit was micaceous, contained various amounts of iron oxide staining, scattered zones of abundant carbonate blebs, stringers, and infilled fractures. Locally the San Diego Formation contains very dense siltstone and hard claystone interbedded layers. The San Diego Formation is early Pleistocene to Pliocene in age.

3.3.5 Otay Formation (To)

As encountered in our boring excavations, the Otay Formation generally consisted of fine- to locally medium-grained sandstones and locally silty claystone. The sandstones encountered during our study were generally light brown to light olive brown, damp to moist, dense to very dense, slightly cemented and friable to very friable. Where the unit becomes more clayey the coloration typically darkens to gray. Typically, the unit was micaceous, contained various amounts of iron oxide staining. Locally the Otay Formation contains very dense siltstone and hard claystone interbedded layers. Claystone interbedded layers often consist of waxy bentonite. The Otay Formation is late Oligocene in age.

3.4 Geologic Structure

Based on our field observations and subsurface exploration, the site is underlain by favorably oriented geologic structure consisting of generally massive fine-grained sandstone of the San Diego and Otay Formations. Specifically, our review of pertinent geologic references (Appendix A), and the results of our previous subsurface exploration (Leighton, 2013), bedding within the San Diego and Otay Formation is generally flat lying with localized dips of between 3 to 5 degrees south to southwest.

3.5 Landslides

Several formations within the San Diego region are particularly prone to landsliding. These formations generally have high clay content and mobilize when they become saturated with water. Other factors, such as steeply dipping bedding that project out of the face of the slope and/or the presence of fracture planes, will also increase the potential for landsliding.

No landslides or indications of deep-seated landsliding were indicated at the site during our field exploration or our review of available geologic literature, topographic maps, and stereoscopic aerial photographs (Appendix A). Furthermore, our field reconnaissance, review of City of Chula Vista hazard maps (Figure 6), and review of Soil-Slip Susceptibility Maps (USGS, 2003), indicate the site is mapped has having a low susceptibility to soil slip. However, based on CGS, 1995, Open-File Report 95-03, the site is mapped has “3-1 – Generally Susceptible” to landslides. Therefore, we have performed slope stability analysis for the site slopes. Additional discussion of slope stability is discussed in the following sections of this report. It should be noted that the closest mapped landslide is approximately 2,000 feet northeast of the site along the very steep northerly descending slope of Telegraph Canyon (CGS, 1995; and Kennedy and Tan 2008).

3.6 Slope Stability

Based on topographic data provided, the site is bound along the north by a moderately sloping cut slope within the San Diego and Otay Formation. Based on our review of the topographic data the cut slope is approximately 33-feet high at an inclination of approximately 2.2:1 (horizontal:vertical). Along the eastern portion of the site a descending natural slope within the San Diego and Otay Formation is also present having a height of approximately 40 feet at an inclination of approximately 4:1 (horizontal:vertical). Based on our observations of the cut and natural slopes within this portion of the site and elsewhere across the site, we observed no indication of slope failures. In addition, we observed only slight sloughing along the toes of any of these slopes. Elsewhere, slightly sloping to moderately sloping natural topography also had no indication of slope failures.

In addition to the native cut slope and natural slope described above, an approximately 2.3:1 (horizontal:vertical) approximately 35-foot high fill slope is located along the eastern portion of the site. Based on our observation of this fill slope, we observed no indication of slope failures. In addition, we observed only slight sloughing along the toe of this slope.

At the time of drafting this report, proposed grading plans for the site were not available for our review. However, based on the proposed locations of site improvements and structure types, we anticipate that proposed grading will consist of minor cuts and fills between 5 feet and 10 feet. Updated analysis should be performed based on Final designs. Our slope stability analysis for the site considered only the existing site conditions. The slope stability calculations are presented in Appendix D.

Table 1
Soil Strength Parameters

Soil Type	Friction Angle (degrees)	Cohesion (psf)
Artificial Fill	28	350
San Diego Formation	39	100
Otay Formation	36	200
Anisotropic	12	150

Our deep-stability search routines considered surfaces analyzed using Spencer's Method of limit equilibrium analysis. In addition, the Otay Formation is generally considered a slide-prone formation in the San Diego area. Therefore, we have modeled presumptive clay seams within the Otay Formation based on observed and referenced data. Our model includes presumptive clay seams are oriented into the analyzed sections (having southwest dips) between 3 and 5 degrees.

Pseudostatic slope stability analysis was performed using a seismic coefficient of 0.26 determined using the methods of Bray and Travasarou (2009). The coefficient determination was based on a 5 cm median seismic displacement threshold and site spectral acceleration based on the 2010 CBC design spectra. A 20 percent increase was considered for dynamic strengths for surfaces along presumptive clay seams. The slope stability calculations are presented in Appendix D. Our analysis indicated a static factor of safety of 1.5, or greater and pseudostatic slope stability of 1.0, or greater.

3.7 Expansive Soils

Based on our field observations, subsurface investigation, and laboratory testing, highly expansive soils were not observed at the site. However, localized more clayey expansive soils were observed at boring B-1 at a depth between 10 and 15 feet below the ground surface. An expansion index test performed on representative clayey soils at the site indicated an Expansion Index of 62 and is classified as Medium. Therefore, measures to mitigate expansion potential are considered necessary during design and construction.

3.8 Hydrocollapse and Compressible Soils

Based on the results of our subsurface exploration, the potential for hydro-collapse of the underlying San Diego and Otay Formation is considered low at the site. Our opinion is supported by our observation of in-place drive samples which indicated a dense to hard, non-porous character for the underlying sandstone, siltstone, and claystone materials. Based on generally low sampler blow counts and visual observations, fill materials exhibit a potential for settlement under loading. As a result, where settlement sensitive improvements are planned, existing fill soils at the site are considered compressible. Therefore, measures to mitigate settlement potential are considered necessary during design and construction.

3.9 Soil Corrosivity

A screening of the onsite materials for corrosivity was performed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on a representative soil sample obtained during our subsurface evaluation.

Laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. Two representative samples were tested. The samples tested had a measured pH of 7.71 and 8.01, and measured minimum electrical resistivity of 878 and 3,044 ohm-cm, respectively. Test results also indicated that the samples had a chloride contents of 24 and 12 ppm, and soluble sulfate contents of 0.0375 and 0.0150 percent (by weight in soil).

3.10 Surface and Ground Water

Ground water was not encountered during our subsurface exploration. Based on site topography and Department of Water Resources well data, we estimate ground water is greater than 150 feet in depth (elevation 300 feet above msl) below the site. Based on site topography, surface water likely drains in various directions away from the center of the site which is generally located at the top of a topographic high. Perched ground water may develop on less permeable layers such as between the existing fill unit and the underlying San Diego and Otay Formation at the site, and on interbedded less permeable units such as claystone. It should be noted that ground water levels may fluctuate during periods of precipitation. Nevertheless, based on the above information, we do not anticipate ground water will be a constraint to the construction of the project.

3.11 Infiltration

The results of our subsurface exploration and laboratory testing indicate that on-site fill soils are of a generally silty sandy nature having relatively good infiltration rates. However, sites located in areas underlain by the San Diego and Otay Formations are known to contain both permeable and impermeable layers which can transmit and perch ground water in unpredictable ways and some LID measures may not be appropriate for the site.

3.12 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 1997), the site is not located within a flood zone (Figure 7). In addition, based on our review of dam inundation and topographic maps, the site is not located within a dam inundation area (Figure 8).

3.13 Exceptional Geologic Conditions

Exceptional geologic items are items that are present across the State of California, and occur on a site by site basis. We have addressed the presence or non-presence of these items typically present across the State in the sections below.

3.13.1 Hazardous Materials

Our scope of work has not included evaluation of the site for hazardous materials and we are not aware of any such reports that pertain to the site.

3.13.2 Regional Subsidence

Due to the depth of ground water and the dense nature of the underlying Eocene-age deposits combined with the close proximity of Mesozoic rock, the possibility of regional subsidence is considered to be nil.

3.13.3 Non-Tectonic Faulting

Surface expressions of differential settlement, such as ground fissures, can develop in areas affected by ground water withdrawal or banking activities, including geothermal production. The site location is not within an area affected by differential settlement caused by non-tectonic sources.

3.13.4 Volcanic Eruption

The proposed site is not located within or near a mapped area of potential volcanic hazards (Miller, C.D., 1989). The nearest volcanic activity is located in the Salton Sea area of southern California. Therefore, volcanic activity is not considered a hazard at the site.

3.13.5 Asbestos

Due to the lack of proximal sources of serpentinic or ultramafic rock bodies, naturally-occurring asbestos is not considered a hazard at the site.

3.13.6 Radon-222 Gas

Historically, Radon-222 gas has not typically been recognized as an environmental consideration in San Diego County. In particular the site area is not mapped as containing organic rich marine shales commonly characterized to potentially contain Radon-222 gas. Therefore, based on our review of the referenced literature, and our site exploration, the potential for the occurrence of Radon-222 gas at the site is considered low.

4.0 FAULTING AND SEISMICITY

4.1 Faulting

The California Mining and Geology Board (now referred to as the California Geologic Survey or CGS) defines an active fault as a fault which has had surface displacement within Holocene time (about the last 11,000 years). The Rose Canyon fault for example is considered active. Furthermore, the State Geologist has defined a potentially active fault as any fault considered to have been active during Quaternary time (last 1,600,000 years). This definition is used in delineating Special Studies Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972 and as subsequently revised (Hart, and Bryant, 2007). The intent of this act is to assure that unwise urban development does not occur across the traces of active faults.

Although similar to the State definition, the City of San Diego (1999) defines a Potentially Active fault, as a fault that has had activity within the last 1.6 million years (Quaternary Period) and can be demonstrated to be inactive during the last 11,000 years (Holocene Epoch). For the purpose of this report, we utilize the City of San Diego definition when referring to fault activity levels.

The primary seismic risk to the San Diego metropolitan area is the Rose Canyon fault zone located approximately 7.5 miles west of the site (Appendix E). The Rose Canyon fault zone consists predominantly of right-lateral strike-slip faults that extend south-southeast bisecting the San Diego metropolitan area (Figure 4). Various fault strands display strike-slip, normal, oblique, or reverse components of displacement. The Rose Canyon fault zone extends offshore at La Jolla and continues north-northwest subparallel to the coastline. The offshore segments are poorly constrained regarding location and character. South of downtown, the fault zone splits into several splays that underlie San Diego Bay, Coronado, and the ocean floor south of Coronado (Treiman, 1993; Kennedy and Clarke, 1999). Portions of the fault zone in the Mount Soledad, Rose Canyon, and downtown San Diego areas have been designated by the State of California (CGS, 2000 and 2003a) as being Earthquake Fault Zones.

A geologic map covering the Imperial Beach Quadrangle (Kennedy and Tan, 1977), an updated geologic map by Kennedy and Tan (2008), and fault maps by

Treiman (1984 and 1993) indicate the site is east of the main La Nacion Fault trace and within a right step-over and associated zone of deformation. As previously mentioned, the LNFZ extends approximately 20 miles (32 kilometers) from the United States/Mexico border along the east side of Chula Vista and National City northward to the Mission Valley area. The fault zone comprises a series of parallel to subparallel, closely spaced west dipping, normal faults which include the La Nacion, Sweetwater and Chula Vista fault strands. The fault strands within the LNFZ generally dip 60 to 75 degrees west and appear to have had predominantly dip-slip movement throughout their history (west side down). The Pliocene-aged San Diego Formation has been displaced a minimum of 256 feet while early Pleistocene deposits have been displaced a minimum of 224 feet (Artim and Pickney, 1973). Fault strands of the LNFZ typically juxtapose the San Diego Formation and Otay Formation and often separate the Lindavista Formation and San Diego Formation. The nearest active fault is the Rose Canyon fault located approximately 7.5 miles west of the site (Figure 4).

4.1.1 Surface Rupture

Based on the results of our previous fault study (Leighton, 2013), the subject site is transected by several minor and discontinuous northeast trending (N10ºE to N45ºE) faults associated with the La Nacion Fault zone. The faults generally dip northwest at 30º to 45º, with a few faults dipping with similar inclination southeast creating zones of down-dropped San Diego Formation (Plate 1 and 2). Of the faults encountered at the site, only one fault was interpreted to be more than 200 feet in length. The remaining faults, including previously mapped faults by others, all appear less than 200 feet in length and do not extend to the overlapping trenches.

Based on the results of our previous study (Leighton, 2013), we conclude that the faults transecting the site, as observed in our exploration trenches, do not constitute a surface rupture hazard. Therefore, the potential for ground rupture due to faulting at the site is considered low. However, based on previously contrasting results concerning the recency of movement along the LNFZ, we recommend that essential facilities maintain a setback distance from the mapped fault traces as previously identified (Leighton, 2013), see Plate 1.

Ground lurching is defined as movement of low density materials on a bluff, steep slope, or embankment due to earthquake shaking. Since the site is relatively flat and removed from any over-steepened slopes (slopes steeper than 2:1 horizontal to vertical inclination), lurching or cracking of the ground surface as a result of nearby or distant seismic events is unlikely.

4.2 Historical Seismicity

Historically, the San Diego region has been spared major destructive earthquakes. The most recent earthquake on the Rose Canyon fault in San Diego occurred after A.D. 1523 but before the Spanish arrived in 1769. Studies by Rockwell and Murbach (1999) indicate that the earthquake occurred at A.D. 1650 ± 125. Two additional earthquakes, the 1800 M6.5 and 1862 M5.9, may have also occurred in the Rose Canyon fault zone. However, no direct evidence of ground rupture within the Rose Canyon fault zone for those events was recorded.

The site location with respect to significant past earthquakes ($\geq$M5.0) is shown on the Historical Seismicity Map in Appendix E. The historic seismicity for the site has been tabulated utilizing the computer software EQSEARCH (Blake, 2000). The results are presented in Appendix E. The results indicate that the maximum historical site acceleration from 1800 to present has been estimated to be 0.16g.

4.3 Seismicity

The site can be considered to lie within a seismically active region, as can all of Southern California. Specifically, the Rose Canyon fault zone located approximately 7.5 miles west of the site is the ‘active’ fault considered having the most significant effect at the site from a design standpoint.

4.3.1 Site Class

Utilizing 2010 California Building Code (CBC) procedures, we have characterized the site soil profile to be Site Class D based on our experience with similar sites in the project area and the results of our subsurface evaluation that indicate existing site fills on the order of up to 25 feet in thickness underlie the site.

4.3.2 2010 CBC Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 2 are the spectral acceleration parameters for the project determined in accordance with the 2010 CBC (CBSC, 2010a) and the USGS Worldwide Seismic Design Values tool (Version 3.1.0).

Table 2
2010 CBC Mapped Spectral Acceleration Parameters

Site Class	D		
Site Coefficients	F_a	=	1.084
	F_v	=	1.631
Mapped MCE Spectral Accelerations	S_S	=	1.041g
	S_1	=	0.385g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.128g
	S_{M1}	=	0.627g
Design Spectral Accelerations	S_{DS}	=	0.752g
	S_{D1}	=	0.418g

The peak horizontal ground acceleration associated with the Maximum Considered Earthquake Ground Motion is 0.45g. The peak horizontal ground acceleration associated with the Design Earthquake Ground Motion is 0.30g.

Since the mapped spectral response at 1-second period (S_1) is less than 0.75g, then all structures are subject to the criteria in Section 1613A of the 2010 CBC are considered to fall within Seismic Design Category D.

4.3.3 2012 IBC Risk-Targeted Mapped Spectral Acceleration Parameters

Risk-targeted mapped spectral accelerations will be adopted in the 2013 California Building Code. For consideration in planning, we are providing the following parameters based on the 2012 International Building Code. As previously discussed, the effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 3 are the risk-targeted spectral acceleration parameters for the project determined in accordance with the 2012 International Building Code (IBC, 2012) and the USGS Worldwide Seismic Design Values tool (Version 3.1.0).

Table 3
2012 IBC Risk-Targeted Mapped Spectral Acceleration Parameters

Site Class	D		
Site Coefficients	F_{PGA}	=	1.149
	F_a	=	1.149
	F_v	=	1.730
Mapped MCE_R Spectral Accelerations	S_S	=	0.878g
	S_1	=	0.335g
Site Modified MCE_R Spectral Accelerations	S_{MS}	=	1.009g
	S_{M1}	=	0.580g
Design Spectral Accelerations	S_{DS}	=	0.673g
	S_{D1}	=	0.387g

Utilizing ASCE Standard 7-10, in accordance with Section 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Maximum Considered Earthquake Geometric Mean (MCE_G). For a Site Class D, the peak horizontal ground acceleration (PGA) is 0.35g and the probabilistic geometric mean peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.40g.

It is noted that the formalized California amendments are not yet published and the 2013 California Building Code will not be adopted until January 1, 2014. As such, further review and updating of the parameters in Table 3 should be performed if these are to be utilized for design. Additionally, although response spectra are less than those determined by the 2010 CBC, based on ASCE 7-10 it is anticipated that the ground motion considered in geotechnical analysis will be the Site Modified MCE instead on two-thirds of that ground motion event as required in the current 2010 CBC. That change could affect seismic loading on retaining walls and psuedostatic slope stability analyses. These parameters and analyses should be revisited once the 2013 CBC becomes available if the 2013 CBC is tube utilized in design.

4.3.4 Site-Specific Ground Motion Analysis

The site is not located in a Seismic Hazard Zone, an Alquist-Priolo Earthquake Fault Zone or in a seismic hazard zone designated in the Safety Element for the City of Chula Vista. Therefore, per Section 4-317(e) of the California Administrative Code the development of a site-specific ground motion analysis is not required per Section 1615A.1.2A of the 2010 CBC.

4.4 Secondary Seismic Hazards

Seismic hazard analysis has been performed considering seismicity prescribed by the 2010 CBC. In general, secondary seismic hazards can include soil liquefaction, seismically-induced settlement, lateral displacement, surface manifestations of liquefaction, landsliding, seiches, and tsunamis. A summary of those potential hazards is presented in the table below:

Table 4
Summary of Secondary Seismic Hazards

Improvement	Soil Liquefaction and Surface Manifestations	Seismically Induced Settlement	Lateral Displacement	Landsliding	Seiches and Tsunamis
Parking Structure	Low	Low	Low	Low	Low
Loop Roadway/Utility Corridor	Low	Yes	Low	Low	Low
East Patient Care Building	Low	Yes	Low	Low	Low
Central Plant	Low	Yes	Low	Low	Low
Future West Patient Care Building	Low	Yes	Low	Low	Low

Specifically, the potential for secondary seismic hazards at the subject site is discussed below.

4.4.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of excess pore-water pressure during strong ground shaking. Liquefaction is associated primarily with loose (low density), granular, saturated soil. Effects of severe liquefaction can include sand boils, excessive settlement, bearing capacity failures, and lateral spreading.

Due to an absence of a shallow ground water table and the presence of loose to medium dense fine-grained silty sandy and clayey fill materials underlain by very dense San Diego and Otay sandstone and claystone materials, the potential for liquefaction at the site is low. In addition, the site is not located within a mapped liquefaction hazard zone (Figure 9).

4.4.2 Seismically-Induced Settlement

Dynamic settlement of soils can occur as a result of strong vibratory ground shaking. Due to the dense nature of the underlying San Diego and Otay Formation, the potential for dynamic settlement is considered to be low within these units.

The potential for dynamic settlement of the existing fill was evaluated using the procedures of Tokimatsu and Seed (1987) as adapted by Pradel (1998). Specifically, these areas are located within the southwestern portion of the proposed East Patient Care Building and across the footprint of the proposed new Central Plant. In addition, portions of the proposed loop roadway and utility corridor located along the eastern boundary of the Master Plan area are subject to dynamic settlement. Based on our analysis, up to approximately 1/2 inch of dynamic settlement is estimated where fills are deepest (Appendix E).

4.4.3 Surface Manifestation of Liquefaction and Dynamic Settlement

Due to absence of a shallow groundwater table and the generally fine-grained silty and sandy fill materials in turn underlain by dense San Diego and Otay Formations, the surface manifestation of dynamic settlement is anticipated to be minor.

4.4.4 Lateral Spreading or Flow Failure

Due to the low potential for liquefaction, and dense nature of the onsite materials, the potential for lateral spreading flow failure is low.

4.4.5 Tsunamis or Seiches

Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. A seiche is an oscillation (wave) of a body of water in an enclosed or semi-enclosed basin that varies in period, depending on the physical dimensions of the basin, from a few minutes to several hours, and in height from several inches to several feet. Based on the elevation (approximately 450 feet msl) and inland location of the site, the potential for damage due to either a tsunami or seiche is low.

5.0 CONCLUSIONS

Based on the results of our investigation of the site, it is our opinion that the proposed Sharp Chula Vista Medical Center Master Plan is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications. The following is a summary of the significant geotechnical factors that we expect may affect development of the site. Our conclusions and recommendations were derived based on the current 2010 CBC and should be revisited if design is proposed under other Codes.

- Existing compacted fill thickness across the site ranges up to approximately 30 feet in localized areas. Specifically, the proposed location of the new east patient care building has existing undocumented fill up to approximately 15 feet thick within the southeast portion of the proposed building footprint. Locally, existing fills are present in Boring B-10 near the west side of the addition. The proposed new central plant has existing undocumented fill up to approximately 22 feet thick within the eastern portion of the proposed building footprint. Based on our document review (Appendix A) and the results of our study, the existing fill soils are considered to be potentially compressible.
- Due to the generally dense sandy character of formational materials underlying the site and lack of adverse geologic conditions, landsliding and mass movement is considered to be unlikely.
- Ground water was not encountered during our investigation and is not anticipated to be a constraint to construction of the proposed structure or site improvements.
- Localized onsite soils were found to have a very low to medium potential for expansion.
- The San Diego and Otay Formation appear to provide moderate infiltration of surface water. However, due to the potential for encountering less permeable interbedded claystone and cemented sandstone within the San Diego and Otay Formation, they are not considered suitable for storm water management strategies that utilize infiltration.
- Exceptional geologic hazards are not anticipated to impact the site or the proposed site development.
- Active faults do not transect or project toward the site. The closest active fault is the Rose Canyon fault located approximately 7.5 miles to the west.

- The site is transected by several potentially active faults. Based on the results of our previous fault study (Leighton, 2013), we conclude that the faults transecting the site, as observed in our previous exploration trenches, do not constitute a surface rupture hazard. Therefore, the potential for ground rupture due to faulting at the site is considered low. However, based on previously contrasting results concerning the recency of movement along the LNFZ, we recommend that essential facilities maintain a setback distance from the mapped fault traces as previously identified, see Plate 1.
- The peak horizontal ground acceleration associated with the Maximum Considered Earthquake Ground Motion is 0.45g. The peak horizontal ground acceleration associated with the Design Earthquake Ground Motion is 0.30g.
- The potential for liquefaction at the site is considered to be low. Differential seismic settlement of less than 1/2 inch is estimated considering the existing site conditions.
- The potential for slope instability at the site is considered to be low.
- Based on the subsurface exploration of the soils underlying the site, we anticipate that fill materials can be excavated with conventional heavy-duty earthwork equipment. Where excavations or borings are proposed into the San Diego and Otay Formation, sloughing within zones of friable sands should be anticipated.
- Laboratory test results indicate the granular onsite soils have a negligible potential for sulfate exposure on concrete and a high corrosion potential to buried uncoated ferrous metals.

6.0 PRELIMINARY RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, excavation, and fill operations. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix G. In case of conflict, the following recommendations shall supersede those in Appendix G.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill, engineered structures, or hardscape should be cleared of surface and subsurface obstructions, including any existing debris and undocumented, loose, or unsuitable fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off site. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 8 inches, brought to optimum or above-optimum moisture conditions, and recompacted to at least 90 percent relative compaction based on ASTM Test Method D1557.

6.1.2 Removals of Compressible Soils in Building Pad Areas

Potentially compressible fill soils that may settle as a result of wetting or settle under the surcharge of engineered fill and/or foundation loads should be removed and placed as moisture conditioned engineered fill. Based on the results of our subsurface exploration, we anticipate fill removal depths on the order of between 5 to 25 feet will be necessary within building pad areas of the East Patient Care Building and Central Plant. The deepest removals will be located in the far northeastern portions of the site near the descending fill slope. The lateral limits of the bottom of the remedial removals should extend to outside the structure footprint a distance of 10 feet. The bottom of the removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

Although not a part of the scope of this study, it should be noted that, based on our review of pre-grading and post-grading topography, and previously completed geotechnical reports for the design of the existing parking deck (Appendix A), removals on the order of 35 feet or deep foundations should be anticipated at the location of the future West Patient Care Building within the northwestern portion of the site.

In general, the old fill and native soil that is removed may be reused and placed as fill provided the material is moisture conditioned to above optimum moisture content, and then recompacted prior to additional fill placement or construction. Soil with an expansion index greater than 50 should not be used within 5 feet of finish grade in the building pad. The actual depth and extent of the required removals should be confirmed during grading operations by the geotechnical consultant.

Table 5
Structure Bearing Condition and Anticipated Maximum Remedial Grading

Location	Bearing Condition	Remedial Grading Depth (bgs)
Parking Structure	Cut/Fill	5 feet
East Patient Care Building	Cut/Fill	15 feet
Central Plant	Fill	25 feet
Future West Patient Care Building	Fill	35 feet

As an alternative to the above recommended removals and fill recompaction, deep foundations may be considered. Additional recommendations are provided in subsequent sections of this report regarding the design of deep foundations.

6.1.3 Cut/Fill Transition Mitigation

Although grading plans were not available at the time of this report, the proposed Parking structure and East Patient Care structure are situated where a cut/fill transition beneath the structure is anticipated. The lateral limits of the bottom of the remedial removals should extend to outside the structure footprint a distance of 10 feet.

Parking Structure

To mitigate the impact of the underlying cut/fill transition condition beneath the Parking structure, the shallow formational materials should be over-excavated to at least 5 feet below finish grade, or 3 feet below the bottoms of proposed foundations, whichever is deeper. Alternatively, all footings for the proposed structure can be extended through the engineered fill and a minimum of 6 inches into competent formational material. The additional depth can be filled with concrete or controlled low-strength material (CLSM) prior to placement of foundation reinforcing steel and concrete.

East Patient Care Building

To mitigate the impact of the underlying cut/fill transition condition beneath the East Patient Care Building structure, the shallow formational materials should be over-excavated to at least 10 feet below finish grade, or 5 feet below the bottoms of proposed foundations, whichever is deeper.

To accomplish the proposed transition over-excavation adjacent to existing structures, we recommend that a temporary 4:1 (horizontal:vertical) slope be excavated from 1 foot above the bottom of the existing foundation depth outward until to at least 10 feet below finish grade, or 5 feet below the bottoms of proposed foundations within the formational material. Should this approach leave existing fills in place under new foundations, deeper excavation should be performed locally.

The over-excavated material should be replaced with properly compacted fill. Where the material is being placed against the 4:1 temporary cut slope, the slope should be benched (Appendix G). Where not bound by existing structures, the over-excavation should laterally extend at least 10 feet beyond the building pad area and associated settlement-sensitive structures.

6.1.4 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. Temporary excavations less than 4 feet in depth, such as utility trenches with vertical sides, should remain stable for the short period required to construct the utility, provided they are free of adverse geologic conditions and friable dry soils.

It should be noted that the site is underlain by dense and moderately cemented materials of the San Diego and Otay Formation. The excavatability of the San Diego and Otay Formation material with conventional heavy-duty construction equipment is expected to require normal effort. It should be noted that heavy ripping and possible rock breaking may be needed in locally cemented and concretionary zones within the formational units. If oversize material (typically over 6 inches in maximum dimension) is generated, it should be placed in non-structural areas or hauled off-site.

In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back if workers are to enter such excavations. Temporary sloping gradients should be determined in the field by a "competent person" as defined by OSHA. For preliminary planning, sloping of fill soils at 1:1 (horizontal to vertical) may be assumed where surcharge loading is not present. Excavations greater than 20 feet in height will require an alternative sloping plan or shoring plan prepared by a California registered civil engineer.

6.1.5 Engineered Fill

In areas proposed to receive engineered fill, the existing upper 8 inches of subgrade soils should be scarified then moisture conditioned to moisture content at or above the optimum content and compacted to 90 percent or more of the maximum laboratory dry density, as evaluated by ASTM D 1557. Soil materials utilized as fill should be free of oversized rock, organic materials, and deleterious debris. Rocks greater than 6 inches in diameter should not be placed within 2 feet of finished grade. Fill should be moisture conditioned to at least 2 percent above the optimum moisture content and compacted to 90 percent or more relative compaction, in accordance with ASTM D 1557. Although the optimum lift thickness for fill soils will be dependent on the type of compaction equipment utilized, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness.

In pavement roadway areas the upper 12 inches of subgrade soils should be scarified then moisture conditioned to a moisture content at or above optimum content and compacted to 95 percent or more of the maximum laboratory dry density, as evaluated by ASTM D 1557.

Placement and compaction of fill should be performed in general accordance with the current City of Chula Vista grading ordinances, California Building Code, sound construction practice, these recommendations and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix G.

6.1.6 Earthwork Shrinkage/Bulking

The volume change of excavated onsite materials upon recompaction as fill is expected to vary with material and location. Typically, the surficial soils and formational sandstone materials vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimates cannot be determined. However, based on the results of our geotechnical analysis and our experience, a 5 percent shrinkage factor is considered appropriate for the existing fill and a 0 to 5 percent bulking factor is considered appropriate for the San Diego and Otay Formation.

6.1.7 Import Soils

Although not anticipated, if import soils are necessary to bring the site up to the proposed grades, these soils should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D4829) and have a low corrosion impact to the proposed improvements. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import. The contractor should provide evidence that all import materials comply with DTSC requirements for import materials.

6.1.8 Removal and Recompaction

Excluding the settlement sensitive building pad areas discussed above in Section 6.1.2, existing fill and disturbed soils within the limits of proposed improvements should also be partially removed, moisture conditioned, and recompacted. Removal depths may be limited to 3 feet below site improvements. Where utilities and pipes are planned in existing fills, the trench subgrade should be scarified at least 8 inches; moisture conditioned and re-compacted to at least 90 percent prior to placement of bedding materials.

6.1.9 Expansive Soils and Selective Grading

Based on our laboratory testing and observations we anticipate the onsite soil materials will generally possess a low expansion potential. It should be noted however that more highly expansive materials may be locally encountered as observed in Boring B-1. Therefore, should more expansive materials be encountered selective grading may need to be performed. In addition, to accommodate conventional foundation design, the upper 5 feet of materials within building pads and 10 feet outside the limits of the building foundations should have a very low to low expansion potential (EI<50).

6.2 Foundation and Slab Considerations

The proposed structures may be constructed with conventional foundations. Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils encountered within 5 feet of pad grade have a very low to medium potential for expansion (EI<50). If more expansive materials are encountered and selective grading cannot be accomplished, revised foundation recommendations may be necessary. The foundation recommendations below assume that the all building foundations will be underlain by properly compacted fill.

6.2.1 Shallow Spread Footing Foundations

Where soils within 5 feet of pad grade have a very low to low expansion potential (EI <50), proposed structures may be supported by spread footings. Footings should extend a minimum of 18 inches beneath the lowest adjacent finish grade. At these depths, footings may be designed for a maximum allowable (FS>3) bearing pressure of 4,000 pounds per square foot when founded in properly compacted fill. Considering that the ultimate bearing capacity is at least 14,000 psf, the allowable pressures may be increased by one-third when considering loads of short duration such as wind or seismic forces. The minimum recommended width of footings is 18 inches for continuous footings and 18 inches for square or round footings. Continuous footings should be designed in accordance with the structural engineer’s requirements and have a minimum reinforcement of four No. 5 reinforcing bars (two top and two bottom). Reinforcement of individual column footings should be per the structural requirements.

6.2.2 Drilled Pile Foundations

If more heavily loaded elements are planned or deep foundations are desired to bypass existing undocumented fill materials, support of those elements on cast-in-drilled hole (CIDH) piles may be considered. Allowable (FS ≥3) axial capacities for CIDH piles were developed using the computer program SHAFT (Version 6.07) produced by Ensoft, Inc. The preliminary analyses considered site conditions, with up to 25 feet of fill underlain by dense formational material. Appendix F presents the applicable preliminary design curves for 2 to 3 foot diameter CIDH piles. Upward capacity equal to one-half the total axial/compressive value may be utilized to resist tensile loads. Pier settlement is anticipated to be less than 1/4 inch under design loads and normal service conditions. The design graph in Appendix F is based on center to center pile spacings of at least 3 pile diameters. Where piles are spaced more closely, reduction in pile capacity is necessary. Construction of piles should be sequenced such that the concrete of constructed piles are allowed to setup prior to construction of piles within 3 diameters. Lateral loads on the face of caissons/piers in areas of level ground surface may be resisted by using a lateral bearing of 300 psf/foot elevation. Where piles are situated closer than 5 diameters (center to center) apart, reduction in lateral bearing is needed and should be reviewed by the geotechnical consultant on a case-by-case basis. More rigorous analysis can also be performed if piles are elected.

All pile installation should be performed under the observation of the geotechnical consultant and consistent with standard practice. Drilling equipment should be powerful enough to drill into the dense to very dense/cemented formational material with cobbles to the design penetration depths. Once a pile excavation has been started, it should be completed within 8 hours, which includes inspection, placement of the reinforcement, and placement of the concrete.

Due to the friable character of the formational materials underlying the site, caving of soils is possible at the site. If caving occurs, a starter casing should be used to protect the top of the borehole to mitigate caving conditions. In addition, the contractor should also be prepared to employ casing or other methods of advancing the drilled pile excavation to mitigate caving. Use of casing should be at the contractor’s discretion. If pile excavations become bell-shaped and cannot be advanced due to severe caving, the caved region may be filled with a sand/cement slurry and redrilled. Redrilling may continue when the slurry has reached suitable set and strength. In this case, it may be prudent to utilize casing or other special methods to facilitate continued drilling after the slurry has set.

6.2.3 Foundation Setback

We recommend a minimum horizontal setback distance from the face of slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on the Table 6 below. The minimum recommended setback distance from the face of retaining wall is equal to 1.5 times the height of the retaining wall. This distance is measured from the outside bottom edge of the footing, horizontally to the slope or retaining wall face, and is based on the slope or wall height. However, the foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 6 Minimum Foundation Setback from Slope Faces	
Slope Height	Minimum Recommended Foundation Setback
Less than 5 feet	7 feet
Greater than 5 feet	10 feet

Please note that the soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a grade beam foundation system to support the improvement.

In addition, open or backfilled utility trenches that parallel or nearly parallel structure footings should not encroach within an imaginary 2:1 (horizontal to vertical) downward sloping line starting 9 inches above the bottom edge of the footing and should also not be located closer than 18 inches from the face of the footing. Deepened footings should meet the setbacks as described above. Also, over-excavation should be accomplished such that deepening of footings to accomplish the setback will not introduce a cut/fill transition bearing condition.

Where pipes cross under footings, the footings should be specially designed. Pipe sleeves should be provided where pipes cross through footings or footing walls and sleeve clearances should provide for possible footing settlement, but not less than 1 inch around the pipe.

6.2.4 Floor Slabs

Slab-on-grade should be at least 5 inches thick and be reinforced with No. 4 rebars 18 inches on center each way (minimum) placed at mid-height in the slab. We recommend control joints be provided across the slab at appropriate intervals as designed by the project architect. Where moisture-sensitive finishes are planned, underslab moisture protection should be designed by the project architect in accordance with Section

4.505 of the 2010 California Green Building Standards Code (CBSC, 2010).

The potential for slab cracking may be reduced by careful control of water/cement ratios. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to minimize cracking of the slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted tile, marble tile, or other crack-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations. If heavy vehicle or equipment loading is proposed for the slabs, greater thickness and increased reinforcing may be required. The additional measures should be designed by the structural engineer using a modulus of subgrade reaction of 150 pounds per cubic inch. Additional moisture/waterproofing measures that may be needed to accomplish desired serviceability of the building finishes and should be designed by the project architect.

6.2.5 Settlement

For conventional footings, the recommended allowable-bearing capacity is based on a maximum total and differential static settlement of 3/4 inch and 1/2 inch. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected where a large differential loading condition exists. Pile settlements are expected to be less than 1/4 inch.

6.2.6 Moisture Conditioning

The building pad and site flatwork subgrade soils should be maintained at a moisture content at least 2 percent above optimum. Testing to confirm the moisture content should be performed prior to placing building slab underlayment and site flatwork.

6.3 Lateral Earth Pressures and Retaining Wall Design

Should retaining walls be included in the project, Table 7 presents the lateral earth pressure values for level or sloping backfill for walls backfilled with fully drained soils of very low to low expansion potential (less than 50 per ASTM D4829).

Table 7 Static Equivalent Fluid Weight (pcf)		
Conditions	Level	2:1 Slope
Active	35	55
At-Rest	55	65
Passive	300 (Maximum of 3 ksf)	100 (slopping down)

Unrestrained (yielding) cantilever walls up to 10 feet in height should be designed for an active equivalent pressure value provided above. If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case-by-case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform lateral pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform pressure equal to 0.35q should be applied to the wall. The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. A typical drainage design is contained in Appendix F. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). If foundations are planned over the backfill, the backfill should be compacted to 95 percent. Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight as outlined in Table 6.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, the friction coefficient of 0.4 may be used at the concrete and soil interface. These values may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

To account for potential redistribution of forces during a seismic event, retaining walls providing lateral support where exterior grades on opposites sides differ by more than 6 feet fall under the requirements of 2010 CBC Section 1615A.1.6 and/or ASCE 7-05 Section 15.6.1 and should also be analyzed for seismic loading. For that analysis, an additional uniform lateral seismic force of $8H^2$ pounds per foot acting at 0.6H should be considered for the design of the retaining walls with level backfill, where H is the height of the wall. This value should be increased by 150% for restrained walls.

6.4 Shoring of Excavations

We anticipate excavations in the northeastern portion of the site to be on the order of 20 feet bgs for the proposed Master Plan. Accordingly, and because of the limited space, temporary shoring of vertical excavations will be required. We recommend that cuts be retained by a soldier beam and lagging shoring system deriving passive support from cast-in-place soldier piles and (lagging-shoring system) with tie-backs. Specialty engineers and contractors with local knowledge of the downtown San Diego area soil conditions typically perform shoring of excavations of this size should be utilized for structural design and construction of the system.

Based on our experience with nearby projects, it is our opinion that the caving potential of the on-site soils is moderate. To accommodate installation of the shoring in the dense to hard underlying geologic units, wide-flange sections may be installed into pre-drilled holes surrounded by concrete. If caving of the drilled holes occurs, drilling slurry or casing may be required. In addition, caving of drilled holes for the tieback anchors may be encountered.

For design of temporary tie-back shoring we recommend a restrained active pressure of 20H assuming a rectangular distribution. All shoring systems should consider adjacent surcharging (such as the presence of construction equipment). The above pressures do not include hydrostatic pressures. A uniform horizontal pressure of equivalent to 2 additional feet of soil should be exerted against the walls that are adjacent to vehicular traffic. Additional surcharge loading from the adjacent buildings should also be considered and shoring elements designed to minimize deflection and preserve the necessary factor of safety for existing footings.

For design of tie-backs, we recommend a concrete-soil bond stress of 1,000 psf of the concrete-soil interface area for straight shaft anchors installed by a competent contractor. This value should be considered only behind the 30 degree line (measured from the vertical) up from the base of the excavation. Temporary tie-back anchors should be individually proof-tested to 150 percent of design capacity. Further details and design criteria for tie-backs can be provided as appropriate. Since design of retaining systems is sensitive to surcharge pressures behind the excavation, we recommend that this office be consulted if unusual load conditions are anticipated. Care should be exercised when excavating into the on-site soils since caving or sloughing of these materials is possible. We recommend that the void space behind lagging be filled with sand/cement slurry. Field testing of tie-backs and observation of soldier pile excavations should be performed during construction.

6.5 Design Ground Water Elevation

As previously discussed in Section 3.3, ground water was not observed in our exploration borings. Based on the results of our subsurface explorations and our experience with similar projects in the site area, we anticipate ground water to be at a depth of 100 feet or more. We do not anticipate that the static ground water will be encountered during the construction of the proposed project. Ground water levels may fluctuate during periods of precipitation.

6.6 Monitoring of Shoring

Settlement monitoring of adjacent sidewalks and structures should be performed to evaluate the performance of the shoring. Shoring of the excavation is the responsibility of the contractor. Extreme caution should be used to minimize damage to existing pavement, utilities, and/or structures caused by settlement or reduction of lateral support. Sequencing of underpinning, shoring installation, excavation and dewatering will be critical to control of deflections and settlement. Once the shoring contractor is selected, a detail excavation phasing plan should be submitted and reviewed by the shoring designer and geotechnical engineer.

The shoring should be surveyed for vertical and horizontal deflection by the Civil Engineer at the top, mid-point, and bottom of each wall face (4 faces) at 50-foot intervals along the wall length. Vertical settlements should be surveyed along an alignment behind the wall at each of the mid-wall monitoring points to a distance behind the wall equal to 1/2 times the wall height. The survey points should be established prior to the start of construction and continued on a weekly basis as the construction proceeds and while the excavation remains open. After completion of the excavation, the survey interval may be extended based on evaluation by the geotechnical consultant.

6.7 Dewatering

We do not anticipate that ground water will be encountered during construction and subterranean levels and foundation excavations will not extend below the ground water table. Therefore, dewatering during construction is not anticipated.

6.8 Preliminary Pavement Design Considerations

Based on R-value and SE test results, we have utilized an R-value of 40 for preliminary design of surface pavements at parking lot locations and an R-value of 30 for pavements associated with the loop driveway. Actual subgrade R Value results should be verified during grading and adjustment made to the base thicknesses as appropriate. If more clayey materials with lower R-value are placed as subgrade in proposed pavement areas, increased base thickness will be necessary.

6.8.1 Flexible Pavement Section

It is our understanding that three types of vehicular traffic are to be considered for pavement design; those are auto parking, auto driveway and fire lane/industrial. Table 8 below provides the traffic indices we have considered in our analysis. For the purposes of developing a traffic index for the project, we have utilized the City of Chula Vista, Subdivision Manual, Section 3, General Design Criteria, dated March 13, 2012.

Table 8 Design Traffic Index Values	
Traffic	Traffic Index
Auto Parking	5.0
Auto Driveway	6.0
Fire Lane/Industrial	9.0

Flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design and are summarized below in Table 9.

Table 9 AC over Aggregate Base Pavement Sections				
Traffic	*R-Value	TI	AC (in)	Aggregate Base (in)
Auto Parking	40	5.0	3	4
Auto Driveway	30	6.0	3	9
Fire Lane / Industrial Driveway	30	9.0	5	13

*assumed value based on preliminary laboratory testing

6.8.2 Rigid Pavement Section

Where Portland Cement Concrete pavements are planned, Table 10 presents PCC pavements sections.

Table 10 PCC Pavement Sections				
Traffic	*R-Value	TI	PCCP (in)	Aggregate Base (in)
Auto Parking	40	5.0	5.5	--
Auto Driveway	30	6.0	7	--
Fire Lane / Industrial Driveway	30	9.0	8	--

*assumed value based on preliminary laboratory testing

Regular crack control joints should be provided for PCC pavement to mitigate the potential for adverse cracking. We recommend that sections be as nearly square as possible. A mix that provides a minimum 600 psi modulus of rupture should be utilized. The actual pavement design should also be in accordance with City of Chula Vista and ACI criteria. All pavement section materials should conform to and be placed in accordance with the latest revision of the Greenbook and American Concrete Institute (ACI) codes and guidelines.

For trash truck aprons, we recommend a full depth of Portland Cement Concrete section of 7 inches with No. 4 bars at 24 inches on center, each way steel and crack-control joints as designed by the project civil or structural engineer. We recommend that jointed sections be as nearly square as possible.

6.8.3 Pavement Section Materials

Prior to placement of the aggregate base material, the upper 12 inches of subgrade soils (including beneath the curb and gutter and 6-inches behind the curb and gutter) should be scarified, moisture-conditioned (or dried back) as necessary to 2 percent above optimum moisture content and compacted to a minimum 95 percent relative compaction based on ASTM Test Method D1557. Aggregate base should be compacted to a minimum 95 percent relative compaction in accordance with ASTM Test Method D1557. Flexible pavements should be constructed in accordance with current Greenbook Specifications. Crushed aggregate base should have a minimum sand equivalent of 40.

Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the mass grading operations.

6.9 Geochemical Considerations

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as “sulfate attack.” Soluble sulfate results (Appendix C) indicated a negligible soluble sulfate content. We recommend that concrete in contact with earth materials be designed in accordance with Section 4 of ACI 318-11 (ACI, 2011).

Minimum resistivity and pH tests were performed on representative samples of subgrade soils (Appendix C). Based on our results, the site soils have a high corrosion potential to buried uncoated metal conduits (Caltrans, 2003). We recommend measures to mitigate corrosion be implemented during design and construction.

6.10 Concrete Flatwork

Concrete sidewalks and other flatwork (including construction joints) should be designed by the project civil engineer and should have a minimum thickness of 4 inches. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 3 percent or above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement.

6.11 Control of Ground Water and Surface Waters

Regarding Low Impact Development (LID) measures, we are of the opinion that infiltration basins, and other onsite storm water retention and infiltration systems can potentially create adverse perched ground water conditions. Therefore, given the site geologic conditions and project type, infiltration type LID measures are not considered to be appropriate for this site and project.

Surface drainage should be controlled at all times and carefully taken into consideration during precise grading, landscaping, and construction of site improvements. Positive drainage (e.g., roof gutters, downspouts, area drains, etc.) should be provided to direct surface water away from structures and improvements and towards the street or suitable drainage devices. Ponding of water adjacent to structures or pavements should be avoided. Roof gutters, downspouts, and area drains should be aligned so as to transport surface water to a minimum distance of 5 feet away from structures. The performance of structural foundations is dependent upon maintaining adequate surface drainage away from structures.

Water should be transported off the site in approved drainage devices or unobstructed swales. We recommend a minimum flow gradient for unpaved drainage within 5 feet of structures of 2 percent sloping away.

The impact of heavy irrigation or inadequate runoff gradient can create perched water conditions, resulting in seepage or shallow ground water conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to the change in moisture content of foundation soils, which may cause distress to a structure and improvements, moisture content of the soils surrounding the structure should be kept as relatively constant as possible. Below grade planters should not be situated adjacent to structures or pavements unless provisions for drainage such as catch basins and drains are made.

All area drain inlets should be maintained and kept clear of debris in order to function properly. In addition, landscaping should not cause any obstruction to site drainage. Rerouting of drainage patterns and/or installation of area drains should be performed, if necessary, by a qualified civil engineer or a landscape architect.

6.12 Construction Observation

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced excavations. The interpolated subsurface conditions should be checked by Leighton Consulting, Inc. in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by a representative of this office. We recommend that all excavations be mapped by the geotechnical consultant during grading to determine if any potentially adverse geologic conditions exist at the site.

6.13 Plan Review

Final project grading and foundation plans should be reviewed by Leighton Consulting as part of the design development process to ensure that recommendations in this report are incorporated in project plans.

7.0 LIMITATIONS

The recommendations contained in this report are based on available project information. Changes made during design development, should be reviewed by Leighton Consulting, Inc. to determine if recommendations are still applicable. Any questions regarding the contents of this report should be directed to the attention of Robert Stroh, CEG, (858) 300-4090 of Leighton Consulting, Inc.

The field evaluations, and geologic analyses presented in this fault hazard report have been conducted in general accordance with current practice and the standard of care exercised by geologic consultants performing similar tasks in the project area. No other warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report.

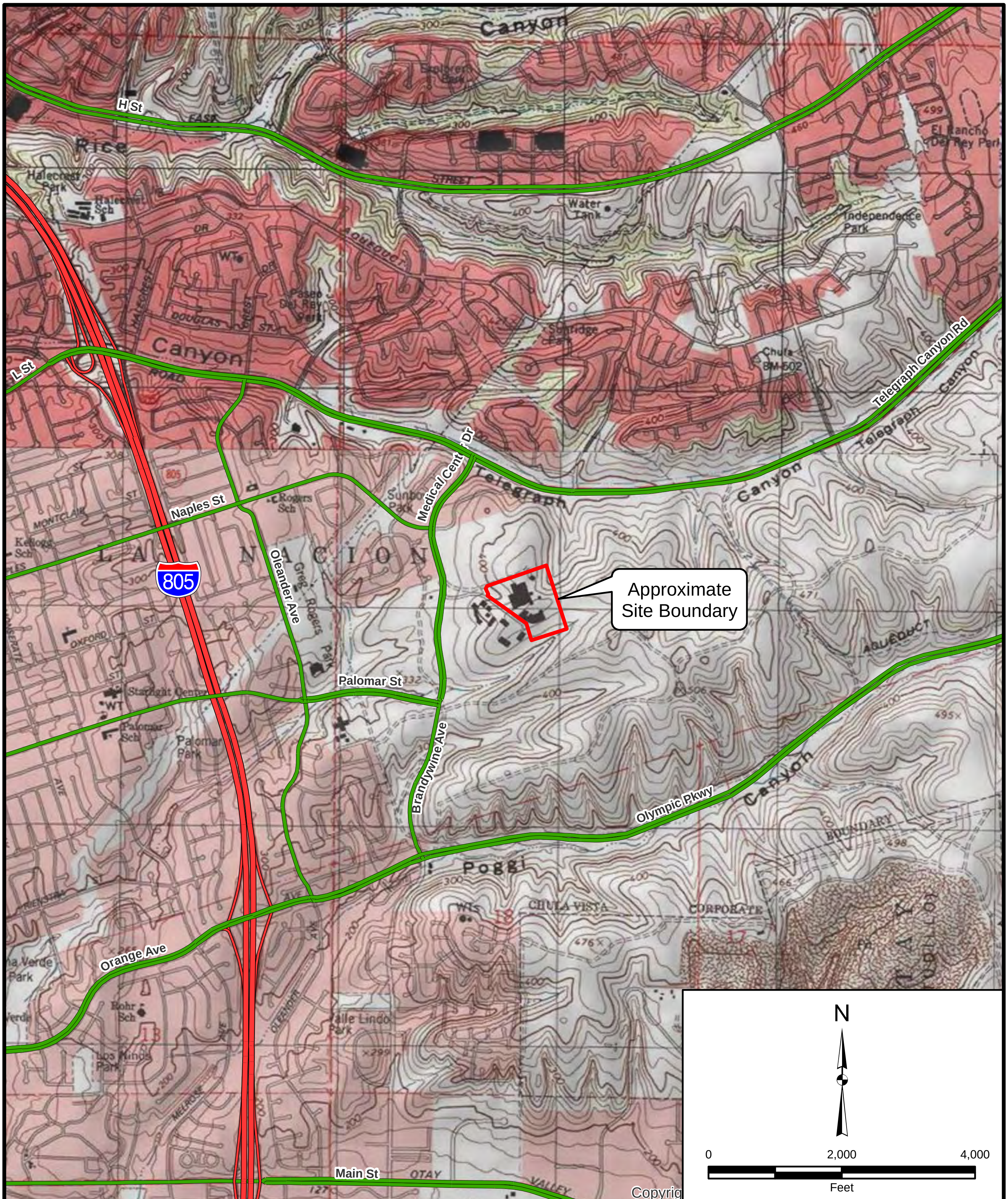
The nature of many sites is such that differing geological conditions can occur over small areal distances and under varying climatic conditions. The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete and therefore preliminary. The findings, conclusions, and recommendations presented in this report are considered preliminary and can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction in order to confirm that our preliminary findings are representative for the site.

IMPORTANT: As stipulated in Section 1803A.1 of the 2010 California Building Code, recommendations in this report are not valid until the report is reviewed and approved by OSHPD. Anyone using this report before OSHPD approval does so at their own risk.

This report was prepared for the sole use of Sharp HealthCare for use with the Sharp Chula Vista Medical Center Master Plan in accordance with generally accepted California licensed geological practices at this time in California.

Please note that our evaluation was limited to assessment of the geologic aspects of the project, and did not include evaluation of structural issues, environmental concerns or the presence of hazardous materials. Our conclusions, recommendations and opinions are based on an analysis of the observed site conditions. If geologic conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request.

Figures



Project: 603541-002	Eng/Geol: SAC/RCS
Scale: 1 " = 2,000 '	Date: July 2013

Thematic Info: ESRI Resources Center, 2013

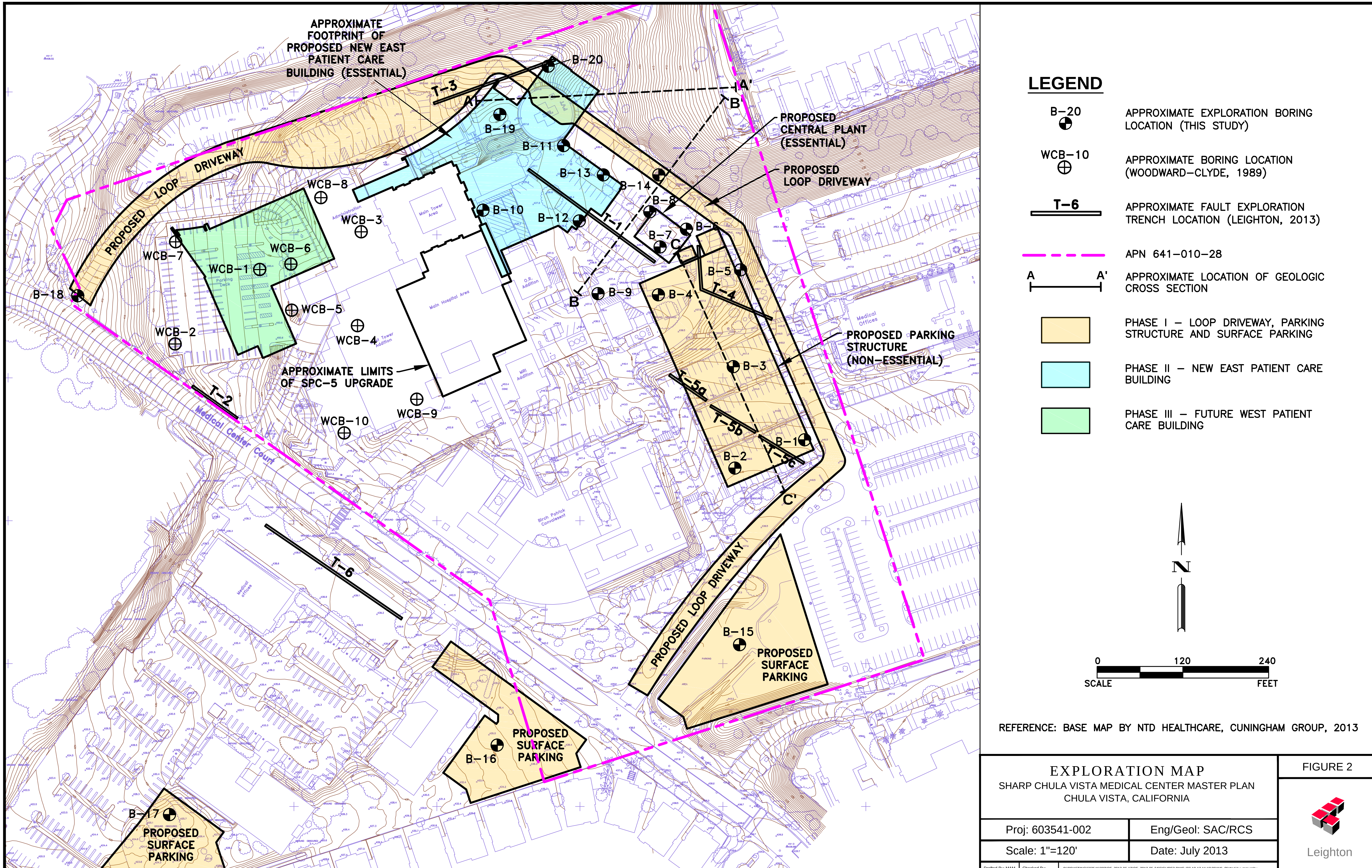
Author: MAM

SITE LOCATION MAP

SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN
CHULA VISTA, CALIFORNIA

Figure 1

Leighton


APPROXIMATE FOOTPRINT OF PROPOSED NEW EAST PATIENT CARE BUILDING (ESSENTIAL)
PROPOSED CENTRAL PLANT (ESSENTIAL)
PROPOSED LOOP DRIVEWAY
PROPOSED LOOP DRIVEWAY
PROPOSED PARKING STRUCTURE (NON-ESSENTIAL)
APPROXIMATE LIMITS OF SPC-5 UPGRADE
PROPOSED LOOP DRIVEWAY
PROPOSED SURFACE PARKING
PROPOSED SURFACE PARKING
PROPOSED SURFACE PARKING
Medical Center Court
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
B-9
B-10
B-11
B-12
B-13
B-14
B-15
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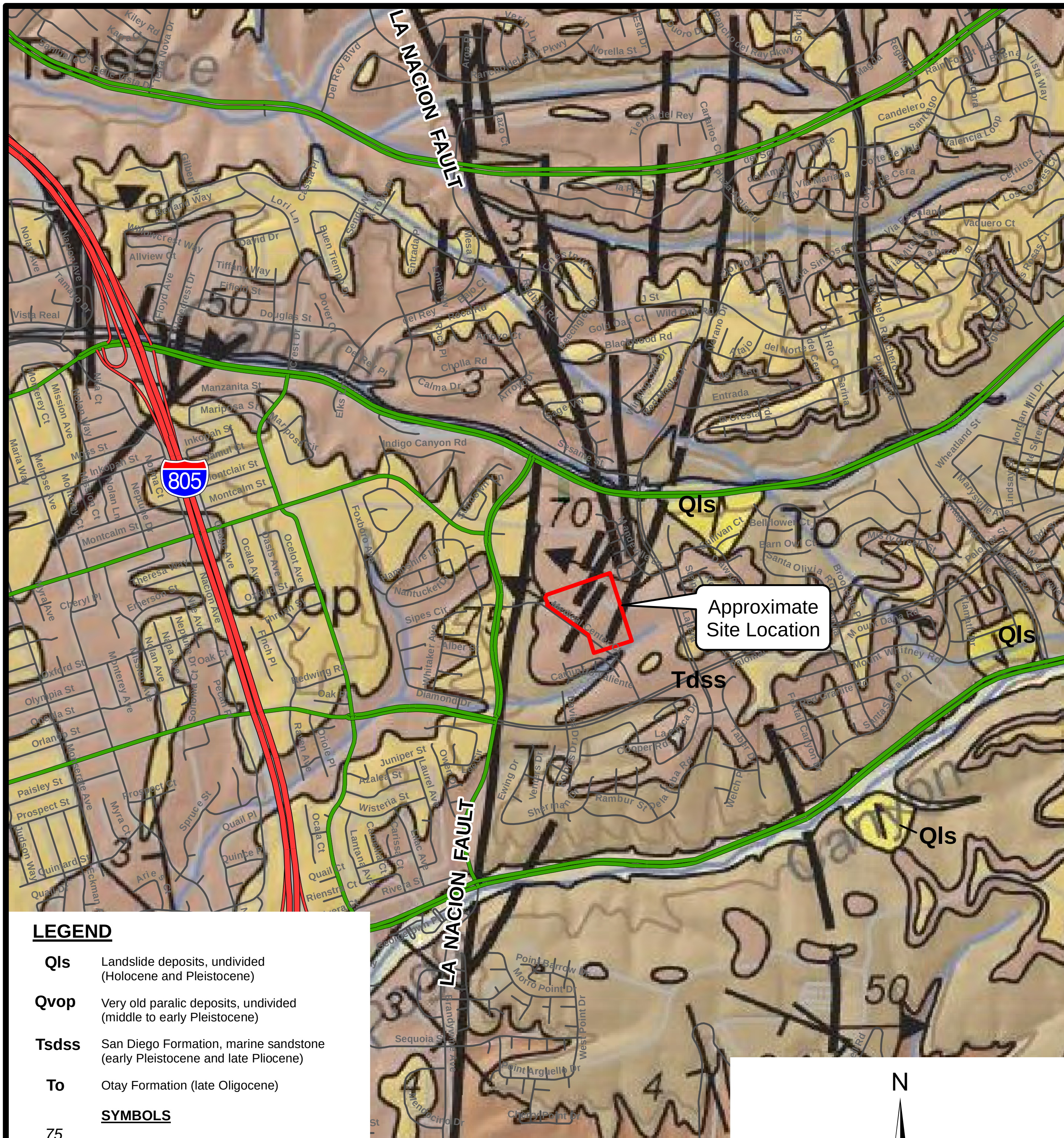
B-19
B-20
WCB-1
WCB-2
WCB-3
WCB-4
WCB-5
WCB-6
WCB-7
WCB-8
WCB-9
WCB-10
T-1
T-2
T-3
T-4
T-5a
T-5b
T-5c
T-6
A
A'
B
B'
C
C'
LEGEND
B-20 APPROXIMATE EXPLORATION BORING LOCATION (THIS STUDY)
WCB-10 APPROXIMATE BORING LOCATION (WOODWARD-CLYDE, 1989)
T-6 APPROXIMATE FAULT EXPLORATION TRENCH LOCATION (LEIGHTON, 2013)
APN 641-010-28
A A' APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION
PHASE I – LOOP DRIVEWAY, PARKING STRUCTURE AND SURFACE PARKING
PHASE II – NEW EAST PATIENT CARE BUILDING
PHASE III – FUTURE WEST PATIENT CARE BUILDING
N
0
120
240
SCALE
FEET
REFERENCE: BASE MAP BY NTD HEALTHCARE, CUNINGHAM GROUP, 2013
EXPLORATION MAP
SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN
CHULA VISTA, CALIFORNIA
FIGURE 2
Proj: 603541-002
Eng/Geol: SAC/RCS
Scale: 1"=120'
Date: July 2013
Drafted By: MAM
Checked By:
Leighton



LEGEND

Qls	Landslide deposits, undivided (Holocene and Pleistocene)
Qvop	Very old paralic deposits, undivided (middle to early Pleistocene)
Tsdss	San Diego Formation, marine sandstone (early Pleistocene and late Pliocene)
To	Otay Formation (late Oligocene)

SYMBOLS

75 — Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. Arrow and number indicate direction and angle of dip of fault plane

3 — Strike and dip of beds

Contact - Contact between geologic units

Project: 603541-002	Eng/Geol: SAC/RCS
Scale: 1 " = 2,000 '	Date: July 2013

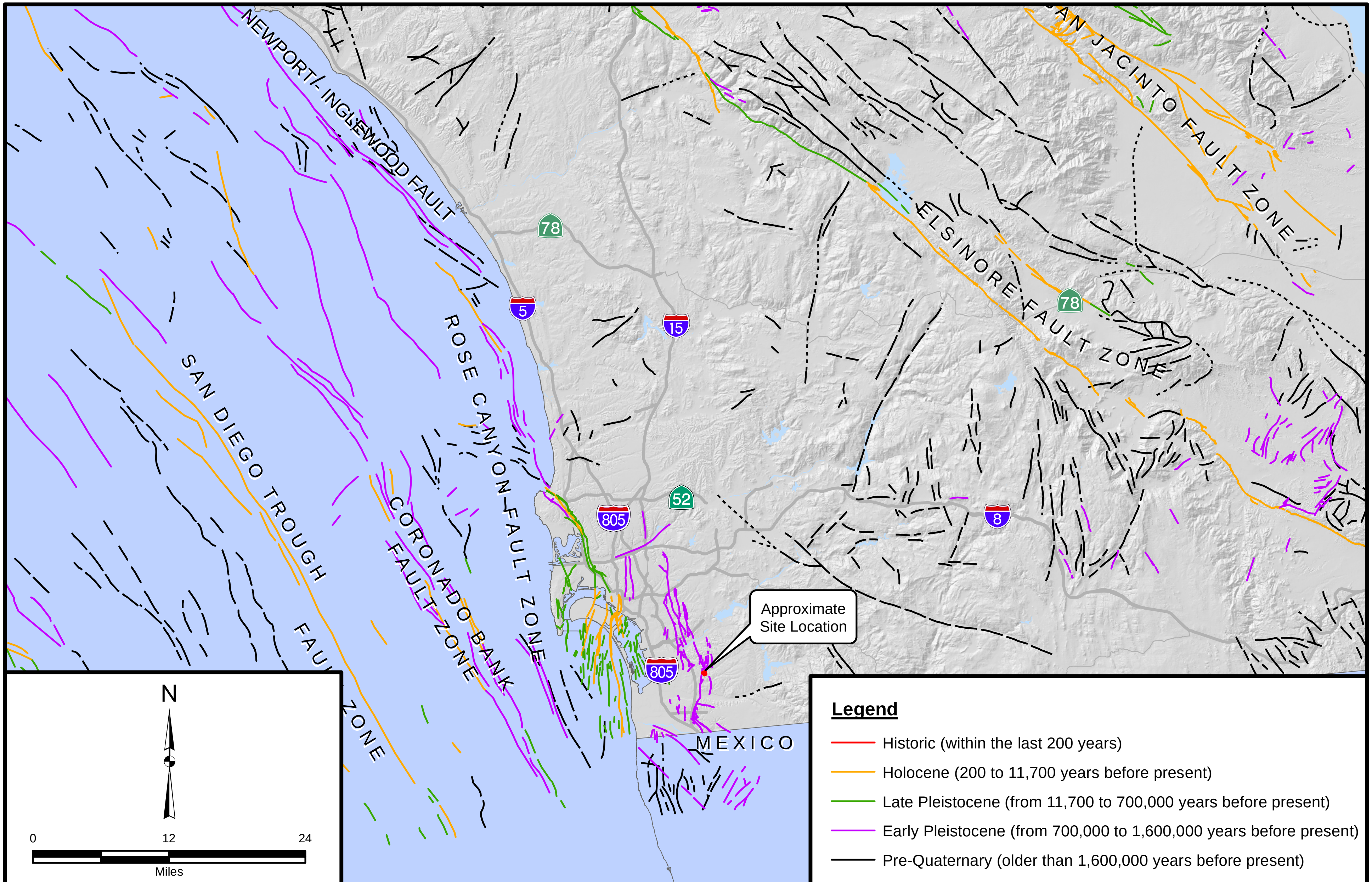
Plate 1 of 2 IN:
Kennedy, M.P. and Tan, S.S., 2008, Geologic map of the San Diego 30' x 60' quadrangle, California: California Geological Survey, Regional Geologic Map No. 3, scale 1:100000
Author: MAM

REGIONAL GEOLOGY MAP

SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN
CHULA VISTA, CALIFORNIA

Figure 3

Leighton



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Scale: 1 " = 12 miles	Date: July 2013

Reference: Jennings and Bryant, 2010 (CGS)

Author: MAM

REGIONAL FAULT MAP

SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN
CHULA VISTA, CALIFORNIA

Figure 4

Leighton

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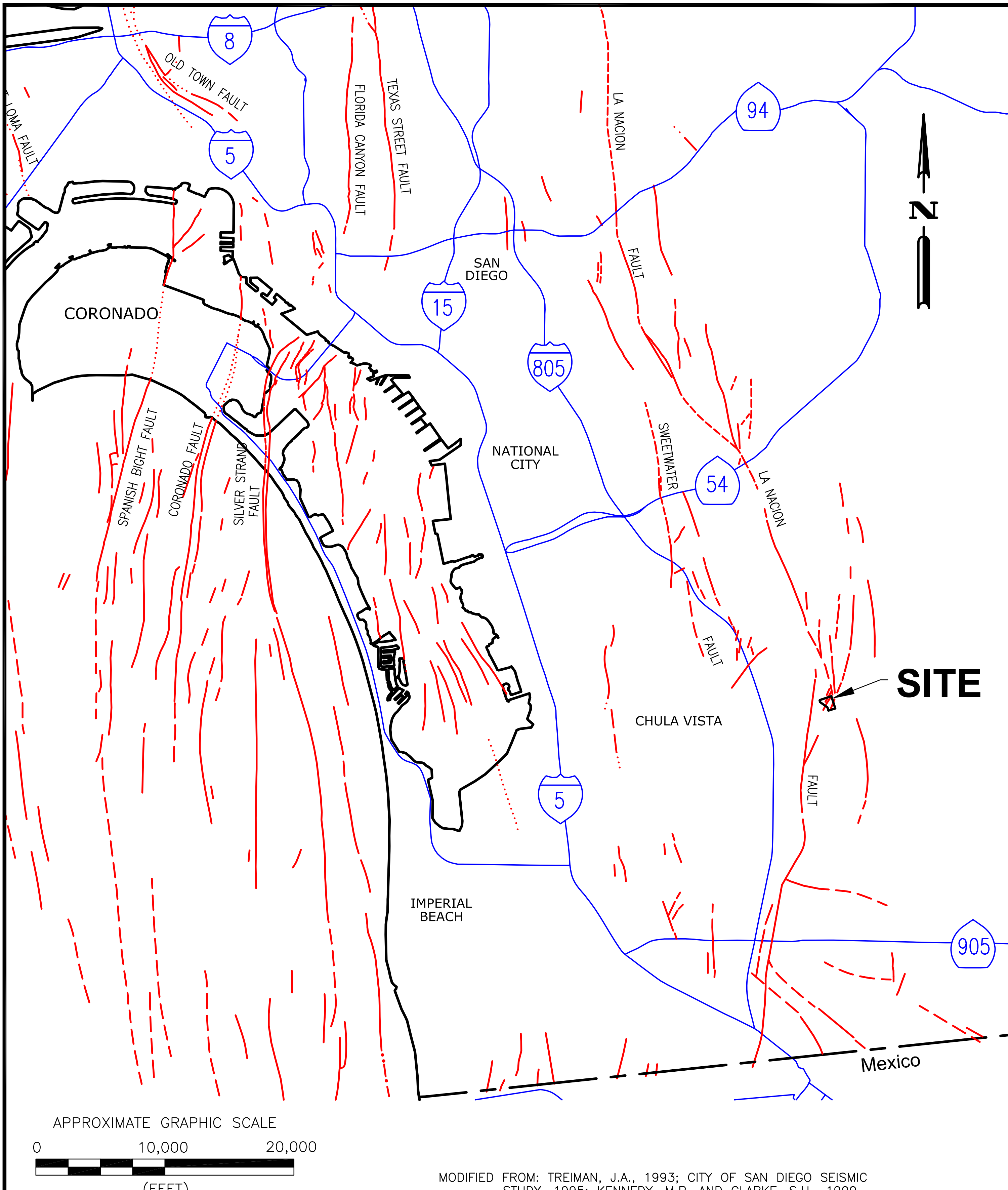
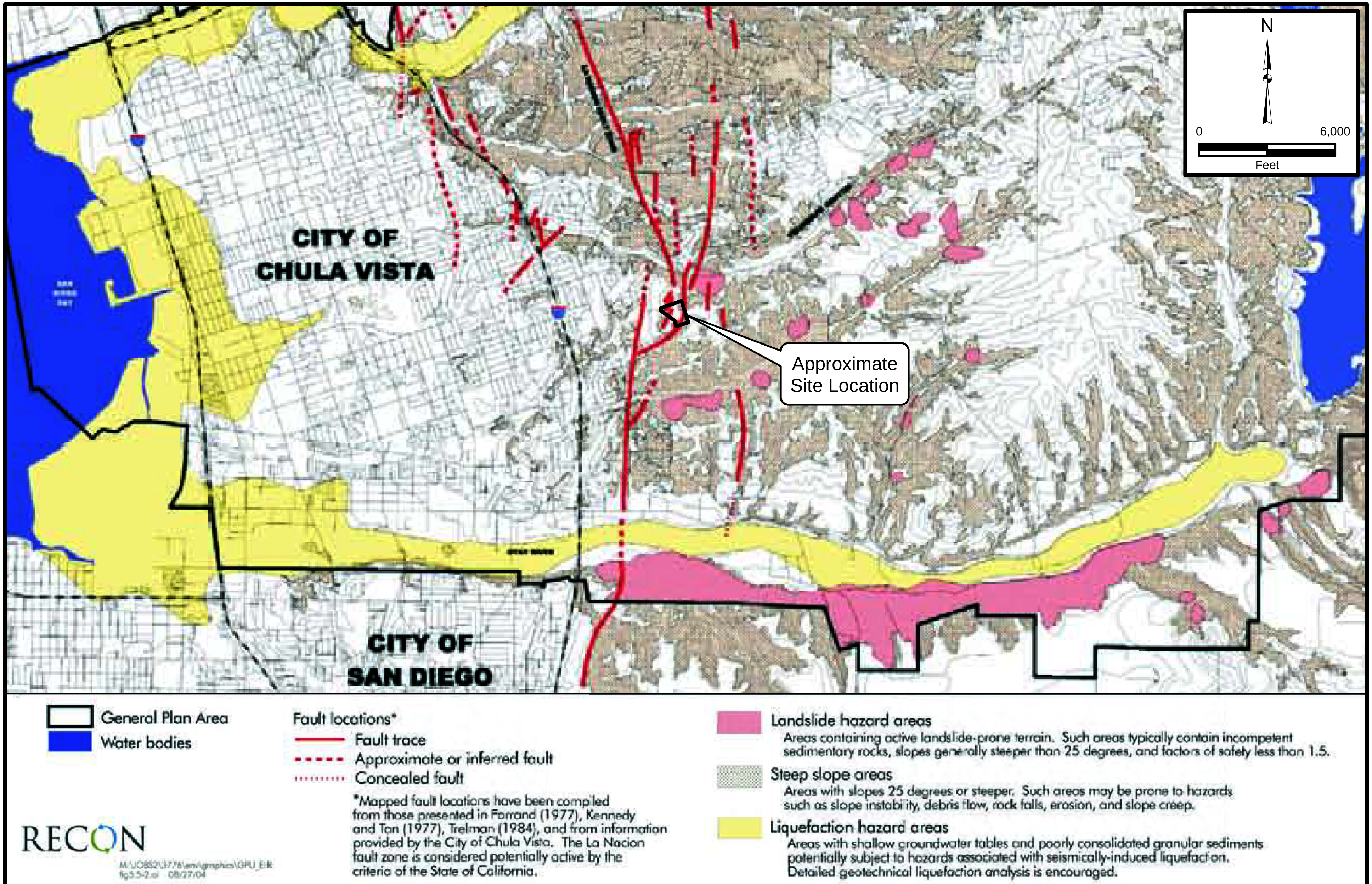


MODIFIED FROM: TREIMAN, J.A., 1993; CITY OF SAN DIEGO SEISMIC STUDY, 1995; KENNEDY, M.P. AND CLARKE, S.H., 1999.

Proj: 603541-002	Eng/Geol: SAC/RCS	**ROSE CANYON FAULT MAP** SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN CHULA VISTA, CALIFORNIA	FIGURE 5
Scale: 1"=10,000'	Date: July 2013		Leighton
Drafted By: MAM	Checked By: RCS		

P:\DRAFTING\603541\002\OF_2013-06-13\OF_2013-05-24\FIGURE5.DWG (07-18-13 11:23:02AM) Plotted by: mmurphy



Project: 603541-002	Eng/Geol:SAC/RCS	GEOLOGIC HAZARDS MAP	Figure 6
Scale: 1 " = 6,000 feet	Date: July 2013	SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN CHULA VISTA, CALIFORNIA	Leighton
Base Map: City of Chula Vista, The Chula Vista City General Plan, 2003. Author: mmurphy (mmurphy)			

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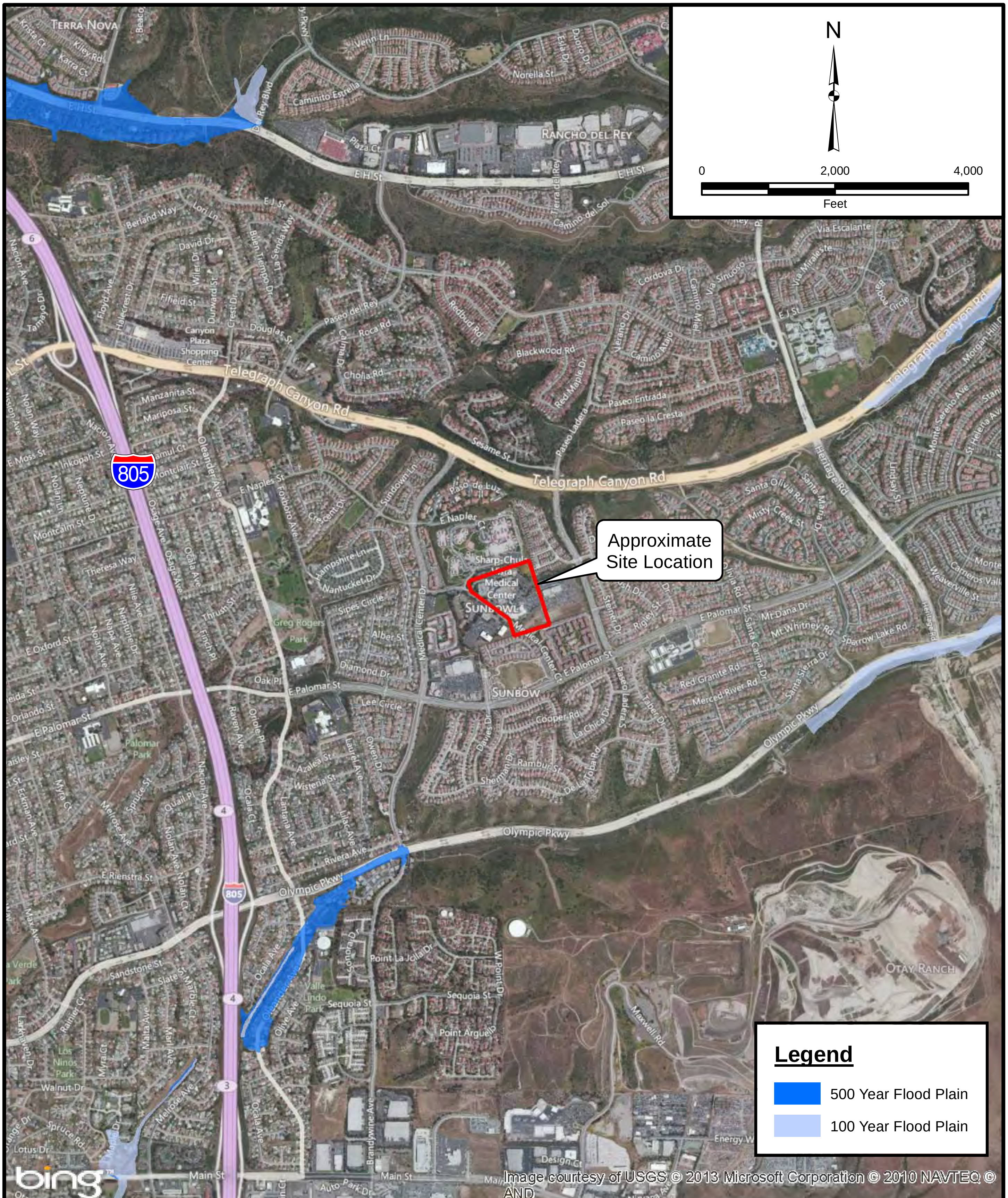

Project: 603541-002	Eng/Geol: SAC/RCS
Scale: 1 " = 2,000 '	Date: July 2013

Reference: FEMA Q3 Flood Data, San Diego County

Thematic Info: Leighton
Author: Mmurphy

FLOOD HAZARD MAP

SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN
CHULA VISTA, CALIFORNIA

Figure 7

Leighton

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Project: 603541-002	Eng/Geol: SAC/RCS
Scale: 1 " = 2 miles	Date: July 2013

Base Map: ESRI Resource Center 2013
Thematic Info: Leighton, San Diego County (Multi-Agency Hazard Mitigation Plan)
Author: (mmurphy)

DAM INUNDATION HAZARD ZONE MAP

SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN
CHULA VISTA, CALIFORNIA

Figure 8

Leighton

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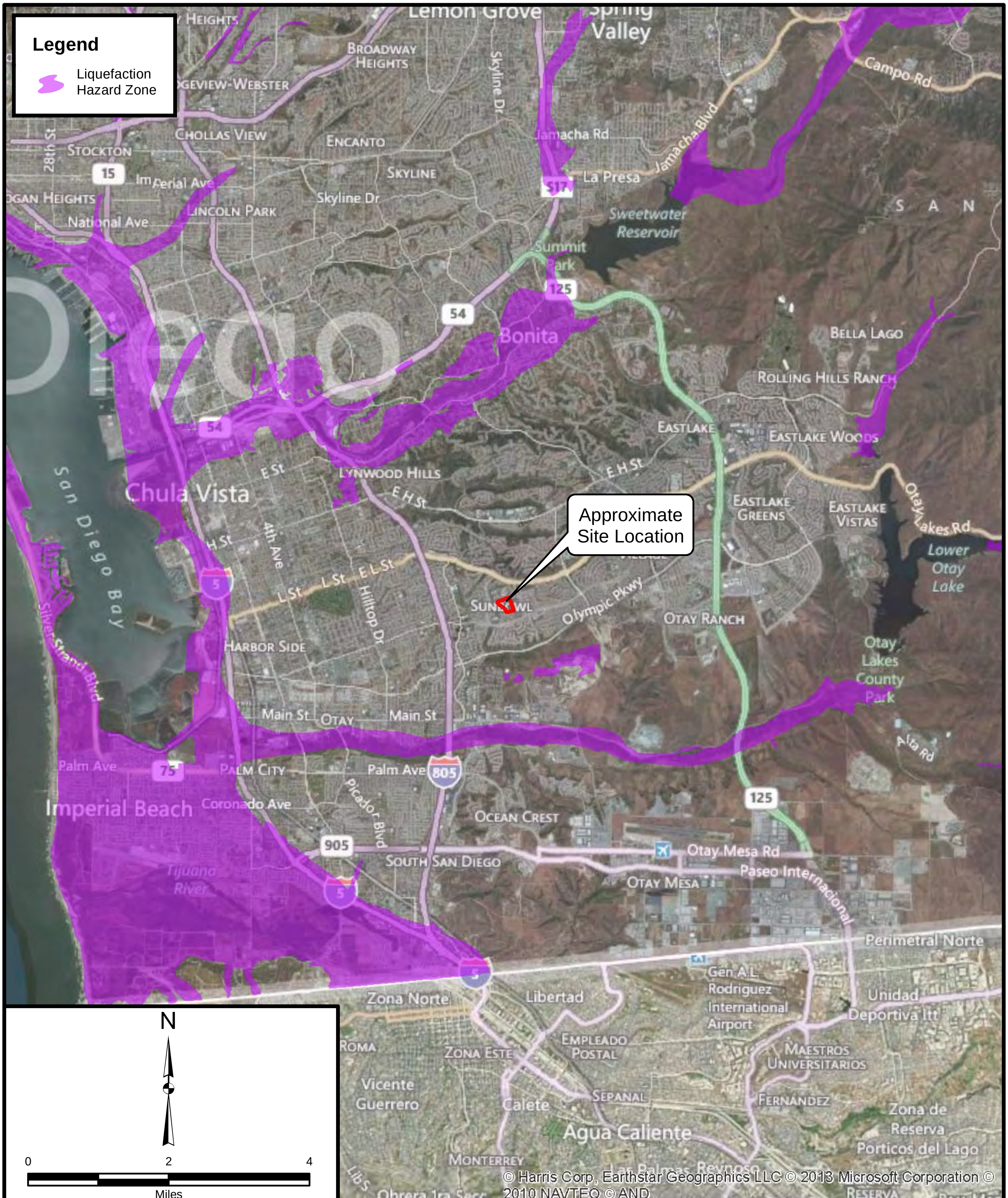

Project: 603541-002	Eng/Geol: SAC/RCS
Scale: 1 " = 2 miles	Date: July 2013

Base Map: ESRI Resource Center 2013
Thematic Info: Leighton, San Diego County (Multi-Agency Hazard Mitigation Plan)
Author: (mmurphy)

LIQUEFACTION HAZARD ZONE MAP

SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN
CHULA VISTA, CALIFORNIA

Figure 9

Leighton

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Plates

LEGEND

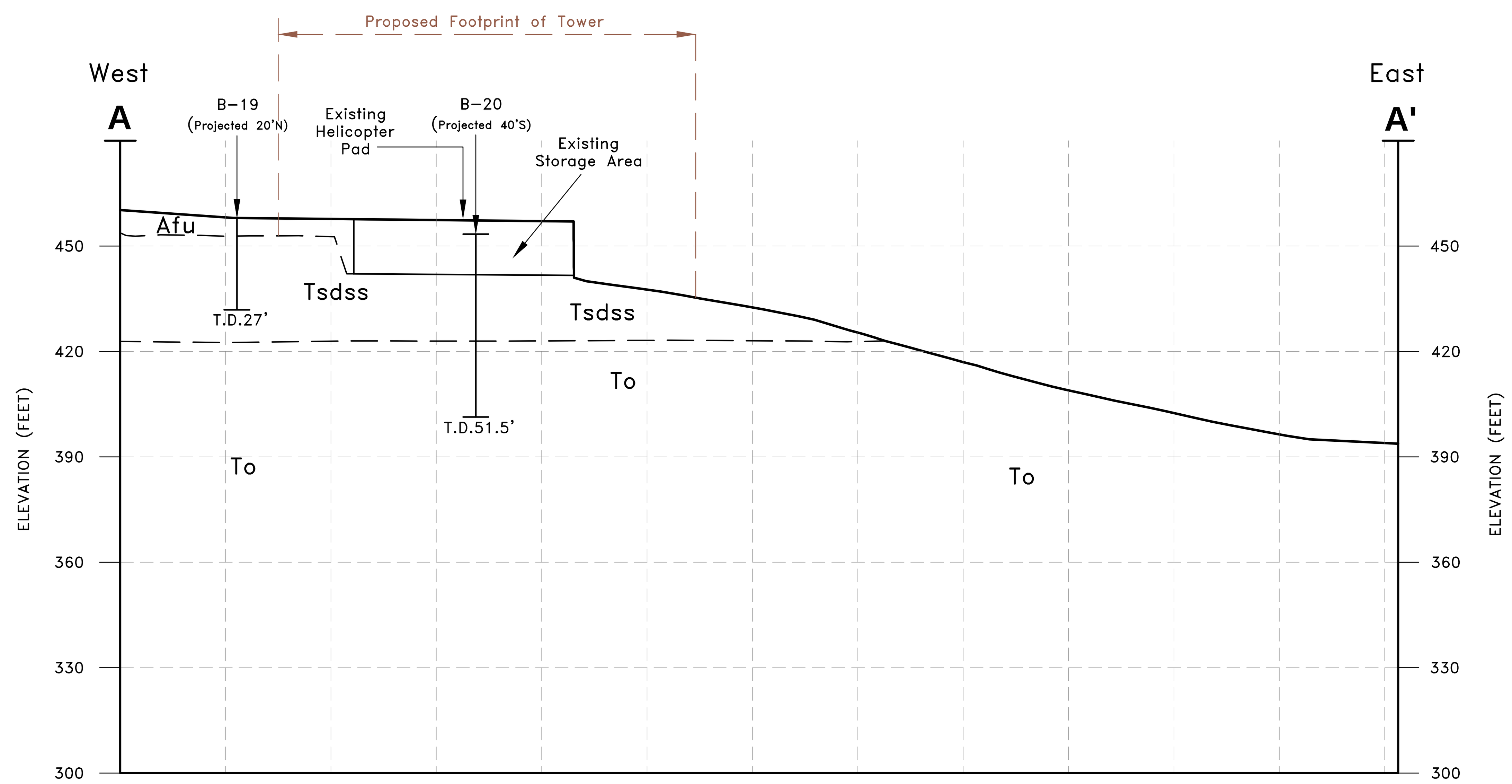
B-20	APPROXIMATE BORING LOCATION
WCB-10	APPROXIMATE BORING LOCATION (WOODWARD-CLYDE, 1989)
T-6	APPROXIMATE FAULT EXPLORATION TRENCH LOCATION (THIS STUDY)
WT-5	APPROXIMATE FAULT EXPLORATION TRENCH LOCATION (WOODWARD-GIZIENSKI & ASSOCIATES, MARCH 15, 1973)
GT-2	APPROXIMATE FAULT EXPLORATION TRENCH LOCATION (GEOCON INC, NOVEMBER 19, 1998)
LT-2	APPROXIMATE FAULT EXPLORATION TRENCH (LEIGHTON AND ASSOCIATES, 1996)
WC-1	APPROXIMATE FAULT EXPLORATION TRENCH LOCATION (WOODWARD-CLYDE, APRIL 25, 1989, REVISED (SEPTEMBER 7, 1989)
	APN 641-010-28
78	MINOR FAULT – POTENTIALLY ACTIVE (11k-1.6k YEARS bp), DASHED WHERE APPROXIMATE, BOX AND NUMBER INDICATE DIRECTION AND AMOUNT OF DIP, WHERE KNOWN
?	APPROXIMATE GEOLOGIC CONTACT, QUERIED WHERE ASSUMED
A A'	CROSS SECTION LINE
	10' STRUCTURAL SETBACK FOR ESSENTIAL BUILDINGS
Afu	UNDOCUMENTED FILL (GREATER THAN 5' IN THICKNESS)
Qvop	UNDIFFERENTIATED LATE PLEISTOCENE-AGE VERY OLD PARALIC DEPOSITS
Tsdss	SAN DIEGO FORMATION – EARLY PLEISTOCENE AND LATE PLIOCENE, MARINE SANDSTONE
To	OTAY FORMATION – LATE OLIGOCENE, MARINE SANDSTONE/CLAYSTONE
	PHASE I – LOOP DRIVEWAY, PARKING STRUCTURE AND SURFACE PARKING
	PHASE II – NEW EAST PATIENT CARE BUILDING
	PHASE III – FUTURE WEST PATIENT CARE BUILDING

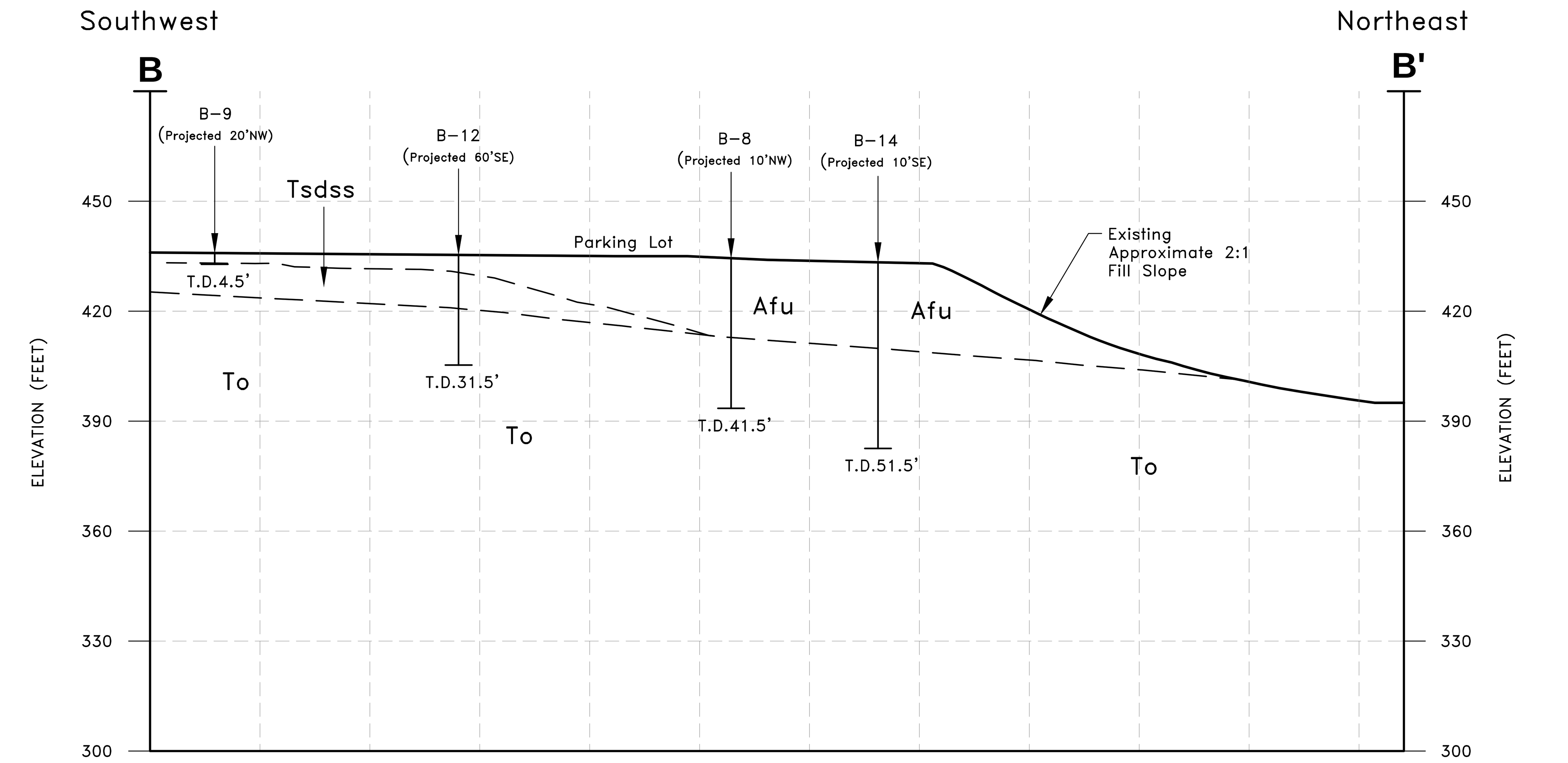
N

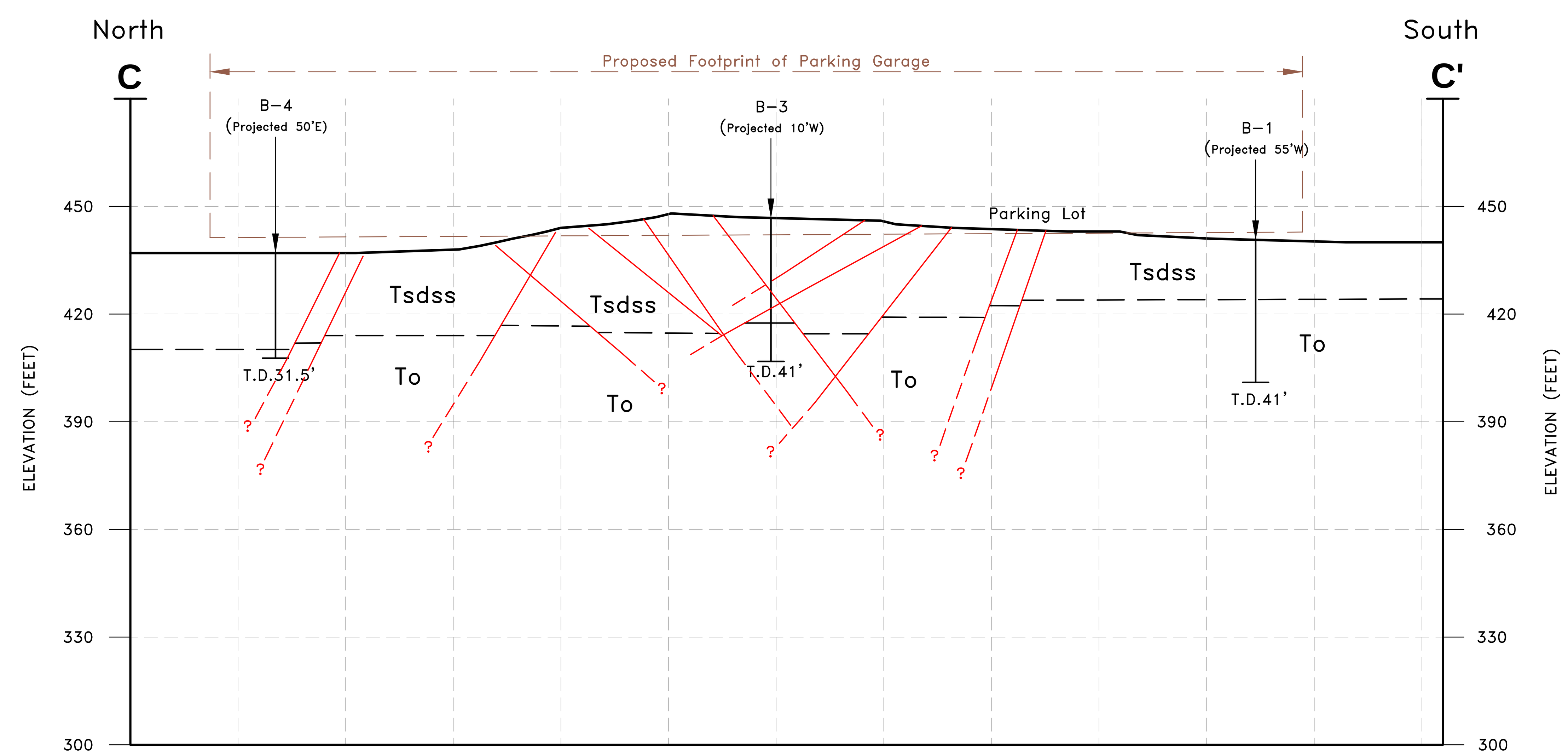
0 40 80
SCALE FEET

REFERENCE: BASE MAP BY NTD HEALTHCARE, CUNINGHAM GROUP, 2013

PLATE 1	GEOTECHNICAL MAP SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN CHULA VISTA, CALIFORNIA	
Leighton	Proj: 603541-001	Eng/Geol: SAC/RCS
	Scale: 1"=40'	Date: 07/2013
	Drafted By: MAM · Checked By: RCS	P:\DRAFTING\603541\002\OF_2013-06-13\OF_2013-05-24\PLATE1.DWG (07-18-13 11:39:18AM) Plotted by: mmurphy







LEGEND

Geologic Contact

Approximate Location of Minor Fault - Potentially Active (11K-1.6K Years B.P.)

Afu Undocumented Fill

Qvop Very Old Paralic Deposits

Tsdss San Diego Formation

To Otay Formation

B-20 T.D.41' Approximate Boring Location With Total Depth

GEOTECHNICAL CROSS SECTIONS A-A' THROUGH C-C'		PLATE 2
SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN CHULA VISTA, CALIFORNIA		
Proj: 603541-002	Eng/Geol: SAC/RCS	
Scale: 1"=30'	Date: 07/2013	Leighton

Appendix A

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APPENDIX A

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APPENDIX A (Continued)

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Appendix B

Boring Logs

GEOTECHNICAL BORING LOG KEY

Project No.		**Date Drilled**	
Project	KEY TO BORING LOG GRAPHICS	**Logged By**	
Drilling Co.		**Hole Diameter**	
Drilling Method		**Ground Elevation**	
Location		**Sampled By**	

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								Asphaltic concrete	
									Portland cement concrete	
								CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay	
								CH	Inorganic clay; high plasticity, fat clays	
								OL	Organic clay; medium to plasticity, organic silts	
	5							ML	Inorganic silt; clayey silt with low plasticity	
								MH	Inorganic silt; diatomaceous fine sandy or silty soils; elastic silt	
								ML-CL	Clayey silt to silty clay	
								GW	Well-graded gravel; gravel-sand mixture, little or no fines	
								GP	Poorly graded gravel; gravel-sand mixture, little or no fines	
	10							GM	Silty gravel; gravel-sand-silt mixtures	
								GC	Clayey gravel; gravel-sand-clay mixtures	
								SW	Well-graded sand; gravelly sand, little or no fines	
								SP	Poorly graded sand; gravelly sand, little or no fines	
								SM	Silty sand; poorly graded sand-silt mixtures	
	15							SC	Clayey sand; sand-clay mixtures	
									Bedrock	
	▽								Ground water encountered at time of drilling	
				B-1					Bulk Sample	
	20			C-1					Core Sample	
				G-1					Grab Sample	
				R-1					Modified California Sampler (3" O.D., 2.5 I.D.)	
				SH-1					Shelby Tube Sampler (3" O.D.)	
				S-1					Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.)	
	25			PUSH					Sampler Penetrates without Hammer Blow	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	TR	THERMAL RESISTIVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

GEOTECHNICAL BORING LOG B-1

Project No.	603541-002	**Date Drilled**	5-1-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	441'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								0-3" Asphalt Concrete	
440								SM	3"-7" Class II Aggregate Base	
								SC	ARTIFICIAL FILL (Afu) @ 7"-1': Light brown silty SAND, moist, medium dense, fine to medium grained	
									SAN DIEGO FORMATION (Tsdss) @ 1': Light olive to light brown clayey SANDSTONE, moist, very dense, fine grained, trace gravel	
435	5									
430	10			R-1 B-1 @10'-15'	28 50/5"	103	12	CL	@ 10': Light brown to light olive-brown sandy CLAYSTONE with some interbedded sandstone, moist, hard	EI, SA, AL
425	15			S-1	14 24 35					
									OTAY FORMATION (To) @ 18': Light brown silty CLAYSTONE, moist, hard, with trace fine sand	
420	20			R-2	15 50/6"			CL		
415	25			S-1	15 25 26			SC-SM	@ 25': Light brown to gray silty clayey SANDSTONE, moist, very dense, fine grained, trace gravel	
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG B-1

Project No.	603541-002	**Date Drilled**	5-1-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	441'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-3	21 50/6"			SC-SM CL	@ 30.5': Light brown to reddish brown, sandy silty CLAYSTONE, damp to moist, hard, trace gravel	
410										
	35			S-3	14 25 36			SM	@ 35': Gray silty SANDSTONE, dry to damp, very dense, friable	
405										
	40			R-4	18 50/5"					
400									**Total Depth = 41 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout on 5/1/13**	
	45									
395										
	50									
390										
	55									
385										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-2

Project No.	603541-002	**Date Drilled**	5-1-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	440'
Location	See Boring Location Map	**Sampled By**	FJW

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
440	0							SM SC-ML	0-2" Asphalt Concrete 2"-5" Class II Aggregate Base ARTIFICIAL FILL (Afu) @ 5"-1': Light brown silty SAND, moist, medium dense SAN DIEGO FORMATION (Tsdss) @ 1': Olive to light brown clayey SANDSTONE to clayey SILTSTONE, damp to moist, dense, friable, micaceous	
435	5									
430	10			R-1	22 50/6"	98	24		@ 10': Moist, very dense	
425	15			S-1	9 22 29			SC	@ 15': Olive to light brown clayey SANDSTONE, moist, very dense, friable, micaceous	
								CL	OTAY FORMATION (To) @ 18': Brown, sandy silty CLAYSTONE, damp to moist, very stiff	
420	20			R-2	29 50/5"			SC	@ 20': Brown clayey SANDSTONE with SILTSTONE, moist, very dense, micaceous	
415	25			S-2	11 25 40			CL	@ 25': Red-brown to light brown sandy CLAYSTONE, moist, hard, micaceous	
410	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-2

Project No.	603541-002	**Date Drilled**	5-1-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	440'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
410	30			R-3	30 50/4"			SC-SM	@ 30.5': Gray silty clayey SANDSTONE, moist, very dense, micaceous	
405	35			S-3					@ 35': Partial sample	
400	40			R-4	50/5"					
									Total Depth = 41 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout on 5/1/13**	
395	45									
390	50									
385	55									
380	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG B-3

Project No.	603541-002	**Date Drilled**	5-1-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	447'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								0-3" Asphalt Concrete	
								SM	3"-6" Class II Aggregate Base	
445								SM-SC	ARTIFICIAL FILL (Afu) @ 6"-1': Gray silty SAND fine grained, dry to damp, friable, micaceous	
									SAN DIEGO FORMATION (Tsdss) @ 1': Grayish to olive-brown, silty clayey SANDSTONE, dense, micaceous, friable	
	5									
440										
	10			R-1	25 50/5"				@ 10': Very dense	DS
435										
	15			S-1	11 14 16				@ 15': Dense	
430										
	20			R-2 B-1 @20'-25'	16 18 23	91	15	SM	@ 20': Light brown to olive silty SANDSTONE, moist, dense, micaceous	
425										
	25			S-2	16 19 21				@ 25': Light brown silty SANDSTONE, moist, very dense	
420										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-3

Project No.	603541-002	**Date Drilled**	5-1-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	447'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-3	22 50/8"			SC	OTAY FORMATION (To) @ 30': Light brown to olive silty clayey SANDSTONE, moist, very dense, micaceous, friable	
415										
	35			S-3	10 11 14				@ 35': Light brown, silty clayey SANDSTONE, moist, dense	
410										
	40			R-4	36 50/4"				@ 40': Very dense	
405									**Total Depth = 41 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout on 5/1/13**	
	45									
400										
	50									
395										
	55									
390										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-4

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	438'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0							SM	0-2" Asphalt Concrete 2"-5" Class II Aggregate Base SAN DIEGO FORMATION (Tsdss) @ 5": Light gray silty SANDSTONE, damp to dry, dense, friable, fine gained	
435										
	5			R-1 B-1 @5'-10'	11 16 31	92	8		@ 5': Light gray to light brown silty SANDSTONE, damp to moist, dense, micaceous, friable	DS
430										
	10			S-1	8 16 17					
425										
	15			R-2	11 15 21					
420										
	20			S-2	8 10 11				@ 20': Medium dense to dense	
415										
	25			R-3	12 21 30				@ 25': Dense	
410								SC	OTAY FORMATION (To) @ 28': Light brown with interbedded orange clayey SANDSTONE, damp, dense to very dense, friable, micaceous	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG B-4

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	438'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			S-3	16 17 21			SC		
405									**Total Depth = 31.5 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/2/13**	
	35									
400										
	40									
395										
	45									
390										
	50									
385										
	55									
380										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-5

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								0-3" Asphalt Concrete	
435				B-1 @1'-4'				SM	3"-7" Class II Aggregate Base ARTIFICIAL FILL (Afu) @ 7"-4': Brown silty SAND with gravel, dry to damp, dense, friable	CR
									SAN DIEGO FORMATION (Tsdss) @ 4': Gray to light brown silty SANDSTONE, damp, dense, friable, micaceous	
430	5			R-1	10 17 26			SM		
425	10			S-1	11 14 15					
420	15			R-2	30 50/5"	97	4		@ 15': Very dense	
415	20			S-2	10 13 17				@ 20': Dense	
410	25			R-3	9 20 34					
								SM-SC	OTAY FORMATION (To) @ 27': Gray to light brown to orange clayey to silty SANDSTONE, damp to moist, dense, friable, micaceous	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-5

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-3	15 15 17			SM-SC		
405										
									Total Depth = 31.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/2/13**	
	35									
400										
	40									
395										
	45									
390										
	50									
385										
	55									
380										
	60									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG B-6

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	435'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
435	0								0-2" Asphalt Concrete	
								SM	2"-5" Class II Aggregate Base ARTIFICIAL FILL (Afu) @ 5"-6': Medium brown silty SAND with gravel, moist, medium dense	
430	5			R-1	8 15 17					
								SM	@ 6': Gray to brown with orange silty SAND with trace gravel, moist, medium dense	
425	10			S-1	8 9 12					
420	15			R-3	10 14 20	108	13			
415	20			S-2	7 7 8					
									SAN DIEGO FORMATION (Tsdss) @ 22': Light brown to reddish brown silty SANDSTONE, moist, medium dense, micaceous, fine grained	
410	25			R-3	10 16 23			SM		
405	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-6

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	435'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
405	30			S-3	7 8 9			SM	@ 30': Light brown to gray silty SANDSTONE, damp, medium dense, fine grained, friable	
400	35			R-4	8 20 26				@ 35': Dense	
395	40			S-4	10 12 13				@ 40': Dense	
									Total Depth = 41.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/2/13**	
390	45									
385	50									
380	55									
375	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-7

Project No.	603541-002	**Date Drilled**	5-7-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	435'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
435	0							SM	0-4" Asphalt Concrete 4"-8" Class II Aggregate Base ARTIFICIAL FILL (Afu) @ 8": Medium brown silty SAND, damp to moist, medium dense, with trace gravel	
430	5			R-1	7 16 31	114	14	SM-ML	SAN DIEGO FORMATION (Tsdss) @ 5': Olive to light brown silty SANDSTONE to sandy SILTSTONE, damp, dense, friable, micaceous, fine	
425	10			S-1 B-1 @10'-13'	7 16 18			SC-CL	@ 10': Gray sandy silty CLAYSTONE to clayey SANDSTONE, moist, dense to very dense, hard	
420	15			R-2	12 26 50			SC	@ 15': Gray to light brown clayey SANDSTONE, moist, very dense, friable, micaceous	
415	20			S-2	10 13 11			SC-SM	@ 20': Gray to light reddish brown clayey to silty SANDSTONE, moist, medium dense, micaceous, friable	
									Total Depth = 21.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/7/13**	
410	25									
405	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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GEOTECHNICAL BORING LOG B-8

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	435'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
435	0							SM	0-2" Asphalt Concrete 2"-5" Class II Aggregate Base ARTIFICIAL FILL (Afu) @ 5": Medium brown silty SAND with clay and trace gravel, moist, medium dense	
430	5			R-1	8 14 16	102	13	SC	@ 6': Medium brown to dark gray clayey SAND with trace gravel, moist, medium dense, micaceous	
425	10			S-1	7 8 9			SM	@ 10': Gray to medium brown silty SAND with trace gravel, moist, medium dense, micaceous, friable	
420	15			R-2	3 4 4	108	15		@ 15': Loose	
415	20			S-2 B-1 @20'-25'	3 2 2					
								SM	OTAY FORMATION (To)	EI
410	25			R-3	7 11 16				@ 26': Light brown silty SANDSTONE with trace gravel, moist, medium dense, micaceous, friable	
405	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG B-8

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	435'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
405	30			S-3	8 8 8			SM		
400	35			R-4	15 27 33				@ 35': Gray to light brown silty SANDSTONE, moist, dense, micaceous, friable, fine grained	
395	40			S-4	8 14 16					
									Total Depth = 41.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/8/13**	
390	45									
385	50									
380	55									
375	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-9

Project No.	603541-002	**Date Drilled**	5-8-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	438'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0								0-4" Asphalt Concrete	
									4-9" Class II Aggregate Base	
				B-1 @1.5'-2'				SM	ARTIFICIAL FILL (Afu) @ 9"-1.5': Medium brown silty SAND with gravel, damp, medium dense	
435									@ 1.5': Gray silty SAND, damp to moist, micaceous, friable, trace clay and gravel	
									@ 4.5': Refusal on concrete	
	5								**Total Depth = 4.5 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/8/13**	
430										
	10									
425										
	15									
420										
	20									
415										
	25									
410										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-10

Project No.	603541-002	**Date Drilled**	5-7-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	439'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								0-5" Asphalt Concrete	
									5"-9" Class II Aggregate Base	
								SM	ARTIFICIAL FILL (Afu) @ 9": Medium to dark brown silty SAND with gravel and cobbles, crushed aggregate, damp to moist, loose (trench or wall backfill)	
435	5			R-1	3 3 3	94	9			
430	10			S-1 B-1 @10'-12'	9 17 20			SM	SAN DIEGO FORMATION (Tsdss) @ 10': Olive to light brown fine silty SANDSTONE, damp, medium dense, friable, micaceous	SA, CR
425	15			R-2	34 37 50/5"	114	5		@ 15': Very dense	
420	20			S-2	9 11 13				@ 20': Dense	
415	25			R-3	26 50/6"			SM/CL	OTAY FORMATION (To) @ 25': Olive to light brown to gray silty SANDSTONE to sandy silty CLAYSTONE, moist, very dense to hard, micaceous	
410	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-10

Project No.	603541-002	**Date Drilled**	5-7-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	439'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			S-3	8 16 22			SM/CL	@ 30': Very dense to hard	
									Total Depth = 31.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/7/13**	
405										
	35									
400										
	40									
395										
	45									
390										
	50									
385										
	55									
380										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-11

Project No.	603541-002	**Date Drilled**	5-6-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								0-5" Asphalt Concrete	
435									5"-9" Class II Aggregate Base	
								SM	ARTIFICIAL FILL (Afu) @ 9"-2.5': Medium brown, silty SAND with gravel, damp, medium dense @ 2.5'-4': Medium brown to olive silty SAND, damp to moist, medium dense, trace gravel, fine grained	
								SM	SAN DIEGO FORMATION (Tsdss) @ 4': Olive to light brown silty SANDSTONE, damp to moist, very dense, micaceous	
	5			R-1	21 50/4"					
430										
				B-1 @8'-10'						
	10			S-1	13 13 18					
425									@ 11': Olive to gray to light brown silty SANDSTONE, moist, dense, calcite deposits, fine grained, friable	
	15			R-2	11 29 50/4"	98	13		@ 15': Very dense	
420										
	20			S-2	13 16 20				@ 20': Very dense	
415										
	25			R-3	19 33 50/3"	98	13	SM	@ 25': Olive to light brown silty SANDSTONE, moist, very dense, micaceous, friable, fine grained, with some interbedded SILTSTONE	
410										
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG B-11

Project No.	603541-002	**Date Drilled**	5-6-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
405	30			S-3	14 28 40			SM	OTAY FORMATION (To) @ 30': Gray silty SANDSTONE, moist, very dense, friable, fine grained	
400	35			R-4	18 50/5"	93	12		@ 35': Gray to light brown	
395	40			S-4	13 25 31					
390	45			R-5	13 43 50/2"	95	7		@ 45': Gray silty SANDSTONE, damp to moist, very dense, micaceous, friable, fine grained	
385	50			S-5 B-2 @50'-55'	14 20 26				@ 50': Gray to light brown, fine to medium grained	
380	55			S-6	16 22 27				@ 55': Interbedded gray to light brown to orange, silty SANDSTONE, damp to moist, very dense, friable, fine grained	
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-11

Project No.	603541-002	**Date Drilled**	5-6-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	60			S-7	12 20 22			SM		
375										
	65			S-8	16 19 34					
370										
	70			S-9	10 19 19				@ 70': Gray to yellowish brown silty SANDSTONE with trace of interbedded sandy CLAYSTONE, moist, very dense to hard, friable	
365										
	75			S-10	15 20 20					
360										
	80			S-11	15 25 50/6"			CL	@ 80': Gray sandy silty CLAYSTONE, moist, hard	
355										
	85			S-12	50/2"				@ 85': No sample recovered	
350										
									@ 88': Harder drilling	
	90									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-11

Project No.	603541-002	**Date Drilled**	5-6-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	90			S-13	38 50/4"			SM	@ 90': Reddish brown to orange-brown silty SANDSTONE, moist, very dense, fine to medium grained	
345										
	95			S-14	39 50/3"			CL	@ 95': Gray to reddish brown CLAYSTONE, moist, hard	
340										
	100			S-15	30 50/3"					
335									**Total Depth = 101 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and cement on 5/6/13**	
	105									
330										
	110									
325										
	115									
320										
	120									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-12

Project No. 603541-002
Project Sharp Chula Vista/Geotechnical Investigation
Drilling Co. Baja Exploration
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Boring Location Map

Date Drilled 5-7-13
Logged By FJW
Hole Diameter 8"
Ground Elevation 436'
Sampled By FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								0-4" Asphalt Concrete	
435									4"-8" Class II Aggregate Base	
								SM	ARTIFICIAL FILL (Afu) @ 8"-5': Medium brown silty SAND with gravel, dry to moist, medium dense	
430	5			R-1 B-1 @5'-10'	19 41 32	115	9	ML	SAN DIEGO FORMATION (Tsdss) @ 5': Light brown to gray sandy SILTSTONE, with trace gravel, dense to very dense , micaceous	SA
425	10			S-1	7 11 11			SM-ML	@ 10': Gray to olive fine silty SANDSTONE to sandy SILTSTONE, dry to damp, dense, micaceous	
420	15			R-2	16 25 50			SM	OTAY FORMATION (To) @ 15': Gray silty SANDSTONE, moist, very dense, micaceous	
415	20			S-2	13 22 25					
410	25			R-3	19 50/6"					
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-12

Project No.	603541-002	**Date Drilled**	5-7-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
405	30			S-3	16 22 20			SM	@ 30': Gray to light brown, silty clayey SANDSTONE, moist, very dense, fine grained	
									Total Depth = 31.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/7/13**	
400	35									
395	40									
390	45									
385	50									
380	55									
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-13

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Graphic Log: N S

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							0-2" Asphalt Concrete	
								2"-6" Class II Aggregate Base	
435							SM	ARTIFICIAL FILL (Afu) @ 6": Medium brown silty SAND with trace gravel, moist micaceous, medium dense	
430	5		R-1	7 13 22	111	14			
425	10		R-2	7 5 7	110	10		@ 10': Loose	
420	15		S-1	11 10 7			SM	OTAY FORMATION (To) @ 15': Gray to light medium brown silty SANDSTONE, moist, medium dense	
415	20		R-3	10 22 40				@ 20': Gray to light brown, silty SANDSTONE with trace clay, dense to very dense, moist, micaceous, friable, fine-grained	
410	25		S-2	10 17 18				@ 25': Very dense	
	30								

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-13

Project No.	603541-002	**Date Drilled**	5-2-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-4	50/5"			SM	@ 30': Gray to light brown silty SANDSTONE, moist, very dense, micaceous, friable	
405										
400	35			S-3	10 19 22				@ 35': Very dense	
395	40			R-5	13 36 50/5"				@ 40': Very dense	
									Total Depth = 41.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/2/13**	
390	45									
385	50									
380	55									
	60									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-14

Project No.	603541-002	**Date Drilled**	5-7-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	435'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
435	0								0-4" Topsoil	
								SM	ARTIFICIAL FILL (Afu) @ 4"-5': Medium brown silty SAND with gravel, moist, medium dense	
430	5			R-1	26 41 50/3"				@ 5': Gray to light brown silty SAND with gravel, moist, very dense, micaceous	DS
425	10			S-1	10 12 16				@ 10': Dense	
				B-1 @12'-15'						MD
420	15			R-2	12 28 43	102	10		@ 15': Very dense	
415	20			S-2	1 2 2				@ 20': Light to medium reddish brown, silty SAND with trace gravel, moist, loose, micaceous	AL, SA, H
410	25			R-3	5 9 18	96	8	SM	OTAY FORMATION (To) @ 25': Light brown to olive silty SANDSTONE, damp, medium dense	DS
405	30									

SAMPLE TYPES:

- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-14

Project No.	603541-002	**Date Drilled**	5-7-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	435'
Location	See Boring Location Map	**Sampled By**	FJW

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
405	30			S-3	9 15 15			SM	@ 30.5': Light brown to gray clayey silty SANDSTONE, damp, dense	
400	35			R-4 B-2 @35'-40'	8 16 28					
395	40			S-4	10 14 19				@ 40': Light brown to gray silty SANDSTONE, damp to moist, dense, micaceous	
390	45			R-5	18 32 50/3"				@ 45': Light brown to olive, very dense	
385	50			S-5	16 19 22				@ 50': Very dense	
									Total Depth = 51.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/7/13**	
380	55									
375	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-15

Project No.	603541-002	**Date Drilled**	5-8-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	443'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								0-2" Topsoil with organics	
								SM	ARTIFICIAL FILL (Afu) @ 2": Olive to light brown to gray, fine silty SAND with clay, damp, dense, micaceous, friable	
440				B-1 @2'-5'						SE
	5			R-1	12 17 29					
435									**Total Depth = 6.5 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/8/13**	
	10									
430										
	15									
425										
	20									
420										
	25									
415										
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-16

Project No.	603541-002	**Date Drilled**	5-8-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	436'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0								0-3" Topsoil	
435								SM	SAN DIEGO FORMATION (Tsdss) @ 3": Olive to light brown to gray silty SANDSTONE, very dense, fine grained, micaceous, friable	
				B-1 @2'-5'						RV, SE
	5			R-1	15 32 50/5"					
430										
									Total Depth = 6.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/8/13**	
	10									
425										
	15									
420										
	20									
415										
	25									
410										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-17

Project No.	603541-002	**Date Drilled**	5-8-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	426'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								0-3" Topsoil	
425								SM	SAN DIEGO FORMATION (Tsdss) @ 3": Olive to light brown silty SANDSTONE, damp, dense, micaceous, friable, fine grained	
				B-1 @2'-5'						SE
	5			R-1	12 15 26					
420										
									Total Depth = 6.5 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/8/13**	
	10									
415										
	15									
410										
	20									
405										
	25									
400										
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-18

Project No.	603541-002	**Date Drilled**	5-8-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	407'
Location	See Boring Location Map	**Sampled By**	FJW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								0-3" Asphalt Concrete	
									3"-6" Class II Aggregate Base	
								SM	ARTIFICIAL FILL (Afu)	
405				B-1 @2'-5'					@ 6': Olive to light brown silty SAND, damp to moist, medium dense, with clay chunks, trace gravel	SE
	5			R-1	7 9 17				@ 5': Olive to gray silty SAND, damp to moist, medium dense, micaceous, trace gravel	
400									**Total Depth = 6.5 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/8/13**	
	10									
395										
	15									
390										
	20									
385										
	25									
380										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

GEOTECHNICAL BORING LOG B-19

Project No.	603541-002	**Date Drilled**	5-3-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	456'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	VERY OLD PARALIC DEPOSITS (Qvop) @ 0': Light to medium brown silty SANDSTONE with GRAVEL-COBBLE CONGLOMERATE, dry to damp, very dense, micaceous, medium grained	
455										
	5			B-1 @4'-8' R-1	50/3"	76	7			
450								SM	SAN DIEGO FORMATION (Tsdss) @ 6': Gray to light brown silty SANDSTONE, damp to moist, very dense, micaceous, friable, fine-grained	
	10			S-1	24 50/5"					
445										
	15			R-2	50/6"	93	11		@ 15': Moist	
440										
	20			S-2	50/6"					
435										
	25			R-3	24 50/6"					
430									@ 27': Refusal on very dense SANDSTONE	
									Total Depth = 27 Feet **No groundwater encountered at time of drilling** **Backfilled with bentonite grout and concrete on 5/3/13**	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG B-20

Project No.	603541-002	**Date Drilled**	5-3-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	452'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								0-6" Topsoil	
450								SM	SAN DIEGO FORMATION (Tsdss) @ 6": Light brown to grayish brown silty SANDSTONE with trace gravel, dry to damp, very dense, friable, micaceous	
	5			R-1	50/5"				@ 5': Damp to moist	
445										
	10			S-1 B-1 @10'-15'	28 50/5"					
440										
	15			R-2	28 50/1"					DS
435										
	20			S-2	15 21 27					
430										
	25			R-3	30 50/3"	97	2		@ 25': Dry	
425										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG B-20

Project No.	603541-002	**Date Drilled**	5-3-13
Project	Sharp Chula Vista/Geotechnical Investigation	**Logged By**	FJW
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	452'
Location	See Boring Location Map	**Sampled By**	FJW

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	30			S-3	20 29 50/5"			SM	OTAY FORMATION (To) @ 30': Light brown to olive fine silty SANDSTONE with trace clay, damp to moist, very dense, friable, micaceous	
420										
	35			R-4	16 23 30				@ 35': Gray to olive to light brown	
415										
	40			S-4	11 20 28					
410										
	45			R-5	16 23 50	98	12		@ 45': Gray to olive fine silty SANDSTONE, moist, very dense, micaceous, friable	
405										
	50			S-5	15 18 20					
400									**Total Depth = 51.5 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite grout on 5/3/13**	
	55									
395										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Appendix B

Woodward-Clyde Borings, 1989

Project: CHULA VISTA HOSPITAL	KEY TO LOGS

Date Drilled:	Water Depth:	Measured:
Type of Boring:	Type of Drill Rig:	Hammer:

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation:			
0			**DISTURBED SAMPLE LOCATION** Sample was obtained by collecting auger cuttings in a plastic bag.			
			DRIVE SAMPLE LOCATION Sample with recorded blows per foot was obtained by using a Modified California drive sampler (2" inside diameter, 2.5" outside diameter). The sampler was driven into the soil at the bottom of the hole with a 140 pound hammer falling 30 inches.			
5						
			Fill			
10			Sand			
			Clay			
			Silt			
15			Sand/Clay			
20			*GS - Grain Size Distribution Analysis DS - Direct Shear Test 'R' - R-Value Test			
25						
30						

Project: CHULA VISTA HOSPITAL	Log of Boring No: 1

Date Drilled: 3-27-89 | Water Depth: Dry | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 431.5'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** 1.5" Asphalt concrete over moist, greenish gray, very silty fine sand with some gravel			
	1-1					
5	1-2	25	Increased gravels	17	100	
			Moist, greenish gray and brown mottled, silty fine sand			
10	1-3	28		13	106	
			Some gravels			
15	1-4	24		21	103	
20	1-5	35		19	100	
			RESIDUAL SOIL Very stiff to hard, moist, dark brown, sandy lean clay (CL) with some gravels and roots (porous)			
25	1-6	13		15	107	UCS= 1466psf
			SAN DIEGO FORMATION Very dense, moist, yellowish brown, silty fine sand with orange laminated staining (SM)			
30	1-7	59		13	107	

Project: CHULA VISTA HOSPITAL	Log of Boring No: 1 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	1-7	59	(Continued) very dense, moist, yellowish brown, silty fine sand with orange laminated staining (SM)			
35	1-8	82				
			Bottom of Boring at 36.5 feet			
40						
45						
50						
55						
60						
65						

Project No: 8951127W-SI01	Woodward-Clyde Consultants	Figure: A-8

Project: CHULA VISTA HOSPITAL	Log of Boring No: 2

Date Drilled: 3-27-89 | Water Depth: Dry | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 426.5'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** 1.5" Asphalt concrete over moist, dark brown to red brown, silty fine sand with some gravels			
5	2-1	24	Moist, greenish brown, silty fine sand (micaceous)	11	97	
10			Moist, brown-gray, silty fine sand with gravels and localized pockets of rusty brown silty sand			
	2-2	57/6"		12	94	
15			Moist, red-brown and green-brown mottled, silty to clayey sand with gravel			
	2-3	29		16	110	
20	2-4	28		21	98	
			Moist, yellowish brown and dark brown mottled, silty sand			
25			Moist, yellow-brown, silty sand (mottled)			
	2-5	36		13	95	
			SAN DIEGO FORMATION Very dense, moist, yellow- brown, silty fine sand (SM) with orange laminated staining			
30						

Project No: 8951127W-SI01 | **Woodward-Clyde Consultants** | Figure: A-4

Project: CHULA VISTA HOSPITAL

Log of Boring No: 2 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	2-6	92	(Continued) very dense, moist, yellowish brown, silty fine sand with orange laminated staining (SM)	11	94	
35	2-7	83				
			Bottom of Boring at 36.5 feet			
40						
45						
50						
55						
60						
65						

Project: CHULA VISTA HOSPITAL	**Log of Boring No: 3**

Date Drilled: 3-27-89 | Water Depth: Dry | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
			Surface Elevation: Approximately 450.5'			
0			**FILL** Moist, red-brown and gray mottled, silty sand with gravels			
	3-1					GS
5	3-2	29		15	102	
10	3-3	24		11	100	
15	3-4	25		13	103	
			Moist, yellow brown to gray, poorly graded medium sand with gravel and localized clay balls			
20	3-5	12				
25	3-6	58	Increased gravel	14	85	
			Refusal on gravel at 25.5 feet			
30						

The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL	Log of Boring No: 4

Date Drilled: 3-27-89 | Water Depth: Dry | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 450'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, yellow brown and dark brown mottled, silty fine sand with gravels and mica			
			Increased gravel			
5	4-1	24		14	102	
			Moist, greenish gray and dark brown, silty sand with localized black, clay balls and gravel			
10	4-2	15		13	99	
			Moist, yellowish brown, silty sand with gravel			
15	4-3	41		10	106	
20	4-4	32	Moist, greenish brown, silty sand with mica and poorly graded sand pockets and gravels	16	104	
25	4-5	15	**RESIDUAL SOIL** Hard, moist, dark brown, clayey fine sand to lean clay with some gravels (SC-CL)	18	110	UNC= 384psf
			SAN DIEGO FORMATION Dense, moist, greenish gray, silty sand with yellow gray staining (SM), micaceous			
30						

The data developed and conclusions reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL				Log of Boring No: 4 (Cont'd)		
Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	4-6	30	(Continued) dense, moist, greenish gray, silty sand with yellow gray staining (SM), micaceous			
35	4-7	32				
40	4-8	55	Very dense, moist, greenish gray, silty fine sand (SM) with mica and calcium carbonates			
	4-9	68	Gravel			
45			Bottom of Boring at 44 feet			
50						
55						
60						
65						

Project No: 8951127W-SI01 **Woodward-Clyde Consultants** Figure A-8

The data developed and c[illegible] recommendations [illegible] correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL | **Log of Boring No: 5**

Date Drilled: 3-28-89 | Water Depth: Dry | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 446'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, yellow brown and dark brown mottled, silty fine sand with gravels			
	5-1					GS,"R'
5	5-2	39		10	101	
			Moist, green brown and green gray, silty fine sand with medium grained sand pockets, gravel and mica			
10	5-3	44		13	106	
15	5-4	35		13	98	
			Moist, green gray, light and dark brown mottled, silty fine sand with gravels, orange staining and mica			
20	5-5	34		12	100	
25	5-6	36		16	99	
			Very moist to wet, green gray and brown, silty fine sand with gravels and orange stained			
30						

Project: CHULA VISTA HOSPITAL	Log of Boring No: 5 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	5-7	52	(Continued) very moist to wet, green gray and brown, silty fine sand with gravels and orange stained	18	104	
			Moist, dark brown, silty fine sand with wood debris and organic odor and gravels			
35	5-8	9				
			RESIDUAL SOIL Hard, moist, dark gray brown, sandy lean clay (CL) some gravels			
	5-9	50/5.5"	**SAN DIEGO FORMATION** Very dense, moist, gray green, silty fine sand (SM) with abundant gravel and some orange staining			
40						
			Dense to very dense yellow brown silty fine sand (micaceous)			
45	5-10	40				
50	5-11	68				
			Bottom of Boring at 50.5 feet			
55						
60						
65						

The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL	Log of Boring No: 6

Date Drilled: 3-28-89 | Water Depth: 24' (perched) | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 441'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, dark and light brown and gray mottled, silty fine sand with orange sandy pockets and some gravels, micaceous			
5	6-1	48		12	100	
10	6-2	36		12	95	
15	6-3	33	Moist, yellow brown, light brown mottled, silty fine sand with gravels and orange pockets(micaceous)	10	97	
20	6-4	38	Moist, light yellow and dark brown, silty sand with dark brown, clayey sand pockets, gravel and micas	16	104	
			▼ increased gravels	20	105	
25	6-5	26	Wet, green-gray and brown mottled, silty sand with dark brown and green pockets, some gravels and wood			
30			**RESIDUAL SOIL** Dense, moist, dark brown, clayey fine sand with gravel and root fibers (SC)			

Project No: 8951127W-SI01 | **Woodward-Clyde Consultants** | Figure: A-11

Project: CHULA VISTA HOSPITAL	Log of Boring No: 6 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	6-6	34	(Continued) dense, moist, dark brown, clayey fine sand with gravel and root fibers (SC)	15	105	UCS= 1002psf
			SAN DIEGO FORMATION Dense, moist, yellow brown, sandy silt with brown staining (ML)			
35	6-7	37	Very hard drilling at 37 feet			
			Medium dense, moist, green-gray, silty fine sand (SM). (micaceous)			
40	6-8	24		22	95	
45	6-9	80				
			Bottom of Boring at 46.5 feet			
50						
55						
60						
65						

Project: CHULA VISTA HOSPITAL	**Log of Boring No: 7**

Date Drilled: 3-28-89 | Water Depth: Dry | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 423'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** 1.5" asphalt concrete over moist yellow-gray, silty sand with gravels and shell fragments (micaceous)			
5	7-1	40				
			Grading to			
			Moist, greenish brown and yellow brown mottled, silty fine sand with orange medium grained sand pockets, gravel and shell fragments			
10	7-2	30		18	99	
			Moist, yellow brown, silty fine sand with gravel, mica and shell fragments			
15	7-3	27		15	100	
			Moist, green-brown and yellow brown, silty sand with dark brown, lean clay pockets with gravel and wood			
20	7-4	44		16	103	
			Moist, red-brown, silty fine sand to sandy silt	20	105	
25	7-5	42	Becomes very hard drilling at 26.5 feet			
			SAN DIEGO FORMATION Very dense, moist, yellow brown silt with orange staining (ML)			
	7-6	65/6"				
			Refusal at 28.5 feet			
30						

Project No: 8951127WSI01 **Woodward-Clyde Consultants** Figure: A13

This was developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL	Log of Boring No: 8

Date Drilled: 3-29-89	Water Depth:Dry	Measured:At time of drilling
Type of Boring: 8" HSA	Type of Drill Rig: CME-55	Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 441'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, green-brown and brown mottled, silty sand with orange medium grained sand pockets and gravels			
5	8-1	24		13	103	
10	8-2	29		15	105	
			RESIDUAL SOIL Stiff to hard, moist, dark brown, sandy lean clay (CL) with gravels			
15	8-3	23	**TERRACE DEPOSITS** Medium dense, moist, red-brown, poorly graded medium sand with silt (SM/ML)			
			Dense gravels			
20	8-4	76/ 5.5"				
			SAN DIEGO FORMATION Very dense, moist, gray, silty very fine sand with cemented zones and micas (SM) with some orange staining			
25	8-5	53				
30						

Project: CHULA VISTA HOSPITAL	Log of Boring No: 8 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	8-6	53	(Continued) very dense, moist, gray, silty very fine sand with cemented zones and mica (SM)			
35	8-7	51				
40	8-8	52				
			Bottom of Boring at 41.5 feet			
45						
50						
55						
60						
65						

Project No: 8951127W-SI01 | Woodward-Clyde Consultants | Figure: A-15

The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL | **Log of Boring No: 9**

Date Drilled: 3-29-89 | Water Depth: Dry | Measured: At time of drilling

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 451'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, dark brown, clayey fine sand with roots and gravel			
	9-1					GS
5	9-2	35		17	106	
			Moist, brown, silty sand with yellow-brown pockets and gravel			
10	9-3	33		12	99	
			Moist, green-gray, silty fine sand with some gravels and micas			
15	9-4	32		8	93	
			Very hard drilling at 17.5 feet			
			TERRACE DEPOSITS Very dense, moist, reddish brown, medium to coarse poorly graded sand (SP)	4	109	
20	9-5	63				
25	9-6	93				
			SAN DIEGO FORMATION Dense, moist, gray, silty fine sand with some orange staining and micas (SM)			
30						

Project No: 8951127W-SI01 | **Woodward-Clyde Consultants** | Figure: A-16

The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL | **Log of Boring No: 9 (Cont'd)**

th, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	9-7	42	(Continued) dense, moist, gray, silty fine sand some orange staining and micas (SM)			
35	9-8	40				
40	9-9	34				
45	9-10	60				
			Bottom of Boring at 46.5 feet			
50						
55						
60						
65						

Project No: 8951127W-SI01 | **Woodward-Clyde Consultants** | Figure: A-17

The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL | **Log of Boring No: 10**

Date Drilled: 3-29-89 | Water Depth:DRY | Measured:AT TIME OF DRILLING

Type of Boring: 8" HSA | Type of Drill Rig: CME-55 | Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 446'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, yellow brown and red brown mottled, silty sand with black spots			
5	10-1	53		11	93	
			Moist, green brown, silty fine sand, micaceous			
10	10-2	20		12	91	
			TERRACE DEPOSITS Very dense, moist, reddish brown, medium to coarse poorly graded sand with gravel (SP)			
15	10-3	69				
			SAN DIEGO FORMATION Very dense, moist, yellow brown, silty fine sand with micas (SM)			
20	10-4	99				
			Grades to Very dense, moist, green gray, silty fine sand with micas (SM)			
25	10-5	53				
			Bottom of Boring at 26.5 feet			
30						

Project: CHULA VISTA HOSPITAL	**Log of Boring No: 11**

Date Drilled: 3-29-89	Water Depth:Dry	Measured:At time of drilling
Type of Boring: 8" HSA	Type of Drill Rig: CME-55	Hammer: 140 lbs at 30" drop

* see Key to Logs, Fig. A-1

Surface Elevation: Approximately 450.5'

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
0			**FILL** Moist, yellow brown, silty fine sand with gravels			
5						
10						
15						
			RESIDUAL SOIL Dense, moist, dark brown, sandy lean clay (CL) with gravels			
			TERRACE DEPOSITS Very dense, moist, red brown, poorly graded medium sand (SP) with gravels			
20	11-1	36				
	11-2	71				
25			**SAN DIEGO FORMATION** Very dense, moist, yellow brown, silty fine sand with orange staining and calcium carbonate and micas (SM)			
	11-3	80				
			Very dense, moist, green gray, silty fine sand with cemented zones and micas (SM)			
30						

Project No: 8951127W-SI01 **Woodward-Clyde Consultants** Figure: A-19

The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Project: CHULA VISTA HOSPITAL	Log of Boring No: 11 (Cont'd)

Depth, ft	Samples	Blows/ft	Material Description	Moisture Content, %	Dry Density, pcf	Other Tests*
30	10-4	83	(Continued) very dense, moist, green gray, silty fine sand with cemented zones and micas (SM)			
	10-5	74				
35			Bottom of Boring at 34.5 feet			
40						
45						
50						
55						
60						
65						

Project No: 8951127W-SI01 **Woodward-Clyde Consultants** Figure: A-20

Appendix C

Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Moisture Determination Tests: Moisture content determinations were performed on relatively undisturbed samples obtained from the boring excavations. The results of these tests are presented on the boring logs.

Expansion Index Tests: The expansion potential of selected materials was evaluated by the Expansion Index Text, ASTM Test Method 4829. Specimens are molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

Sample Location	Description	Expansion Index	Expansion Potential
B-1, 10-15 feet	Light Brown to Light Olive Brown to sandy lean CLAY	62	Medium
B-8, 20-25 feet	Medium Brown to Brown silty SAND with a trace of GRAVEL	9	Very Low

Maximum Dry Density and Optimum Moisture Content Tests: The maximum dry density and optimum moisture content of selected representative soil samples were evaluated in general accordance with ASTM D 1557. The test results are presented in the table below and the plotted curve is presented in the test data.

Sample Location	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-14, 12-15 feet	Light Brown to Medium Reddish Brown clayey silty SAND with a trace of GRAVEL	123.2	12.0

Direct Shear/Soil Strength Tests: Direct shear test was performed on selected remolded sample which was soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing

APPENDIX C (Continued)

force. The samples were tested under various normal loads, using a motor-driven, strain-controlled, direct-shear testing apparatus. The test results are presented in the test data.

Sample			Peak Shear		Ultimate Shear	
Sample Location	Unit	Sample Description	Friction Angle (degrees)	Apparent Cohesion (psf)	Friction Angle (degrees)	Apparent Cohesion (psf)
B-3 @ 10-11 feet	Tsdss	Grayish to Olive-Brown silty clayey SANDSTONE	37.0	158.5	32.5	157.5
B-4 @ 5-6 feet	Tsdss	Light Gray to Light Brown silty SANDSTONE	37.4	47	36.8	0
B-14 @ 5-6 feet	Afu	Gray to Light Brown silty SAND	42.6	3.5	28.1	390
B-14 @ 25-26 feet	To	Light Brown to Olive silty SANDSTONE	38.3	639	35.8	130.5
B-20 @ 15-16 feet	Tsdss	Light Brown to Grayish Brown silty SANDSTONE	40.4	105	39.5	114.5

Soluble Sulfates: The soluble contents of selected samples were determined by standard geochemical methods. The test results are presented in the table below:

Sample Location	Sulfate Content (%)
B-5 @ 1 to 4 feet	0.0375
B-10 @ 10 to 12 feet	0.0150

APPENDIX C (Continued)

Chloride Content: Chloride content was tested in accordance with DOT Test Method No. 422. The results are presented below:

Sample Location	Chloride Content, ppm
B-5 @ 1 to 4 feet	24
B-10 @ 10 to 12 feet	12

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with California Test Method 643. The results are presented in the table below:

Sample Location	pH	Minimum Resistivity (ohms-cm)
B-5 @ 1 to 4 feet	7.71	878
B-10 @ 10 to 12 feet	8.01	3,044

Particle/Grain Size Analysis (ASTM D422): Particle size analysis was performed by mechanical sieving, wash sieving, and hydrometer methods according to ASTM D422, D 1140, D4318, and D6913. The percent fine particles from these analyses are summarized below. Plots of the sieve and hydrometer results are provided on the figures at the end of this Appendix.

Sample	Percent Passing No. 200 Sieve
B-1 @ 10-15 feet	60
B-10 @ 10-12 feet	27
B-12 @ @ 5-10 feet	52
B-14 @ 20-21 feet	45

APPENDIX C (Continued)

Atterberg Limits (ASTM D 4318): The Atterberg Limits were determined in accordance with ASTM Test Method D4318 for engineering classification of the fine-grained materials and presented in the table below:

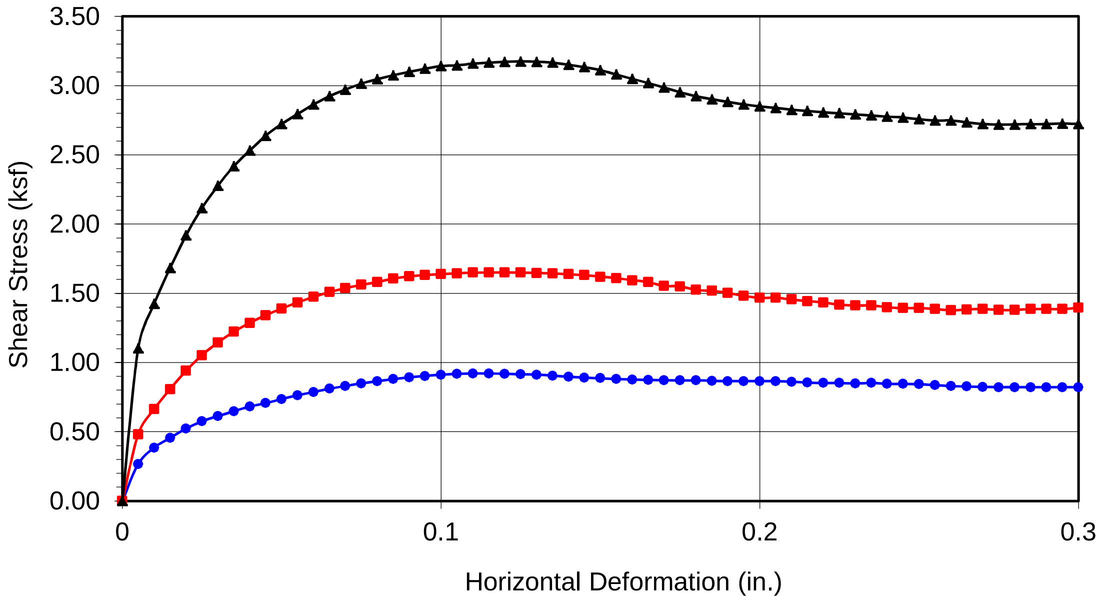
Sample	Plasticity Index	Liquid Limit (%)	Plastic Limit (%)	USCS Soil Classification
B-1 @ 10-15 feet	17	31	14	CL
B-14 @ 20-21.5 feet	3	23	20	ML

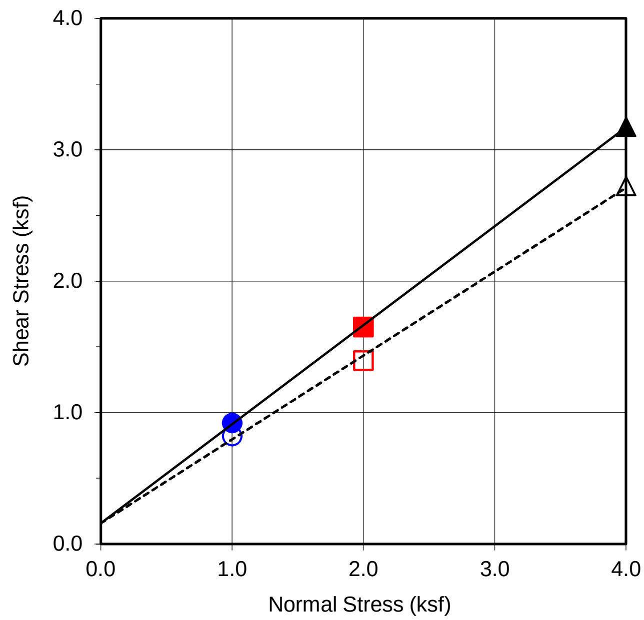
"R"-Value: The resistance "R"-value was determined by the California Materials Method CT301 for base, subbase, and basement soils. The samples were prepared and exudation pressure and "R"-value determined. The graphically determined "R"-value at exudation pressure of 300 psi is reported.

Sample Location	Sample Description	R-Value
B-16 @ 2 to 5 feet	Olive to Light Brown to Gray silty SANDSTONE	63

Sand Equivalent Test (ASTM D 2419): Sand equivalent (SE) tests were performed on selected representative samples. The SE value is the ratio of the coarse- to fine-grained particles in the selected samples.

Sample	Average SE
B-15 @ 2 to 5 feet	25
B-16 @ 2 to 5 feet	34
B-17 @ 2 to 5 feet	45
B-18 @ 2 to 5 feet	18





Boring No.	**B-3**
Sample No.	**R-1**
Depth (ft)	**10-11**
Sample Type:	Ring
Soil Identification: Pale olive silty sand (SM)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	158.5	37.0
Ultimate	157.5	32.5

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.921	■ 1.650	▲ 3.175
Shear Stress @ End of Test (ksf)	○ 0.821	□ 1.396	△ 2.723
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	6.47	6.47	6.47
Dry Density (pcf)	84.7	90.1	88.6
Saturation (%)	17.6	20.0	19.3
Soil Height Before Shearing (in.)	0.9883	0.9862	0.9780
Final Moisture Content (%)	30.7	30.2	30.0

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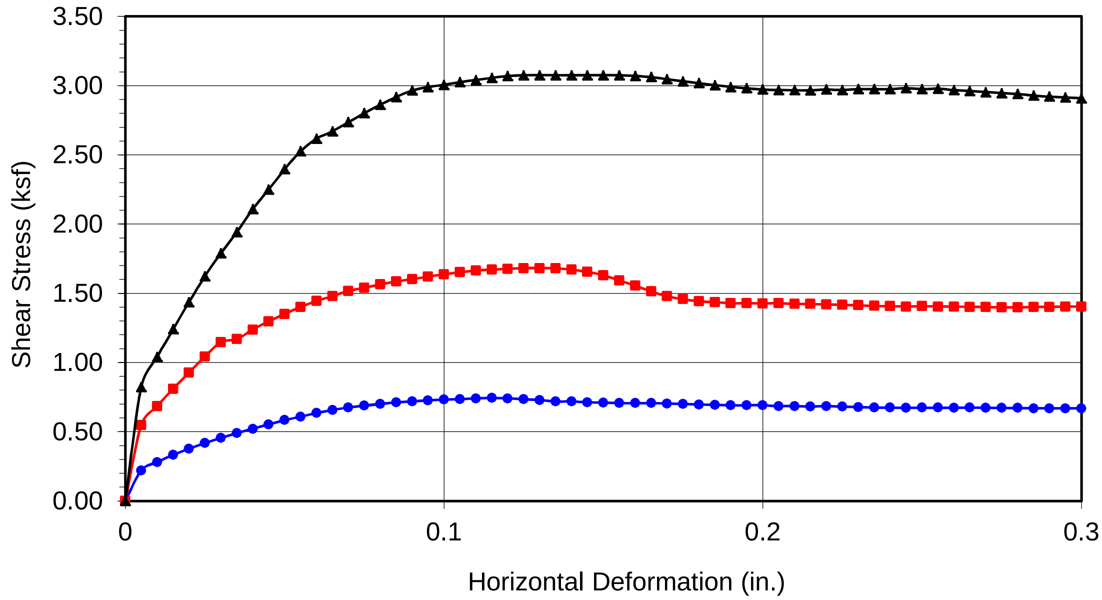
DIRECT SHEAR TEST RESULTS

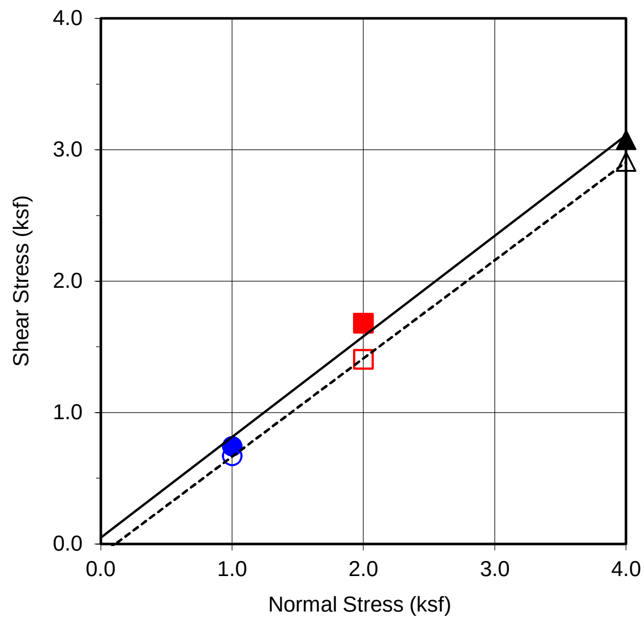
Consolidated Drained - ASTM D 3080

Project No.: 603541-002

SHARP CHULA VISTA MEDICAL CENTER
MASTER PLAN

05-13





Boring No.	**B-4**
Sample No.	**R-1**
Depth (ft)	**5-6.0**
Sample Type:	RING

Soil Identification:

POORLY GRADED SAND WITH SILT (SP-SM), light grayish brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	47.0	37.4
Ultimate	-83.5	36.8

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.744	■ 1.681	▲ 3.075
Shear Stress @ End of Test (ksf)	○ 0.669	□ 1.404	△ 2.909
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	10.78	10.86	10.33
Dry Density (pcf)	91.1	90.4	87.7
Saturation (%)	34.2	33.9	30.3
Soil Height Before Shearing (in.)	0.9824	0.9825	0.9618
Final Moisture Content (%)	29.4	28.5	29.0

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DIRECT SHEAR TEST RESULTS

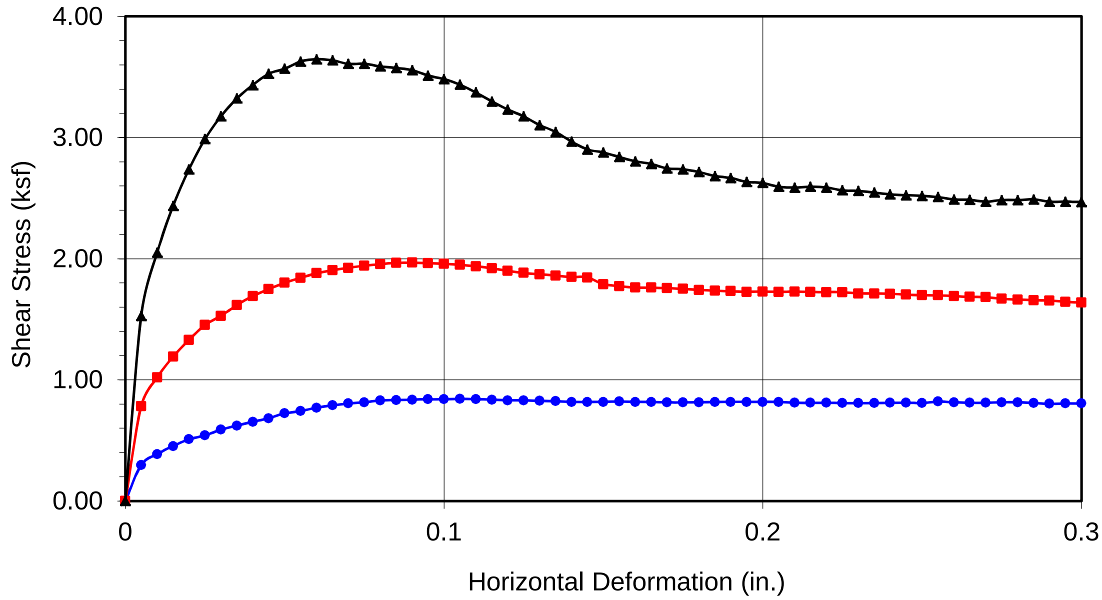
Consolidated Drained ASTM D-3080

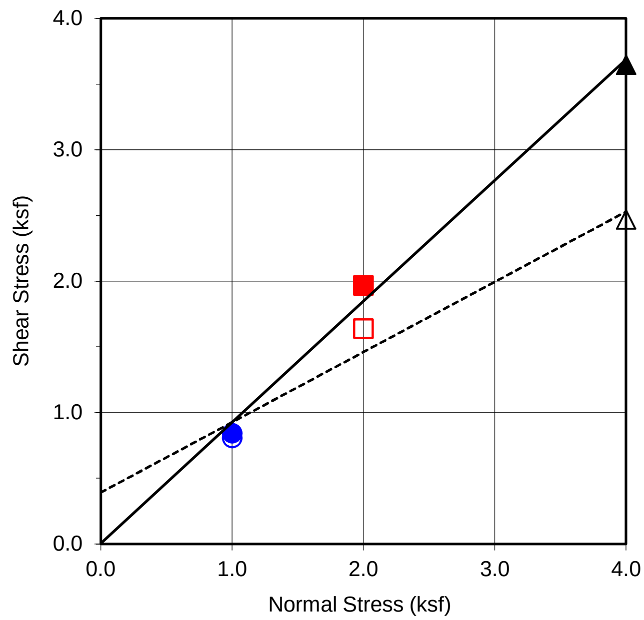
Project No.: 603541-002

SHARP CHULA VISTA MEDICAL CENTER

MASTER PLAN

05-13





Boring No.	**B-14**
Sample No.	**R-1**
Depth (ft)	**5-6**
Sample Type:	Ring

Soil Identification:
Yellowish brown silty, clayey sand with gravel (SC-SM)g

Strength Parameters

	C (psf)	ϕ (°)
Peak	3.5	42.6
Ultimate	390.0	28.1

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.843	■ 1.968	▲ 3.647
Shear Stress @ End of Test (ksf)	○ 0.805	□ 1.638	△ 2.468
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	7.50	7.50	7.50
Dry Density (pcf)	99.2	105.2	112.7
Saturation (%)	29.0	33.6	40.9
Soil Height Before Shearing (in.)	0.9646	0.9775	0.9861
Final Moisture Content (%)	17.3	18.6	17.6

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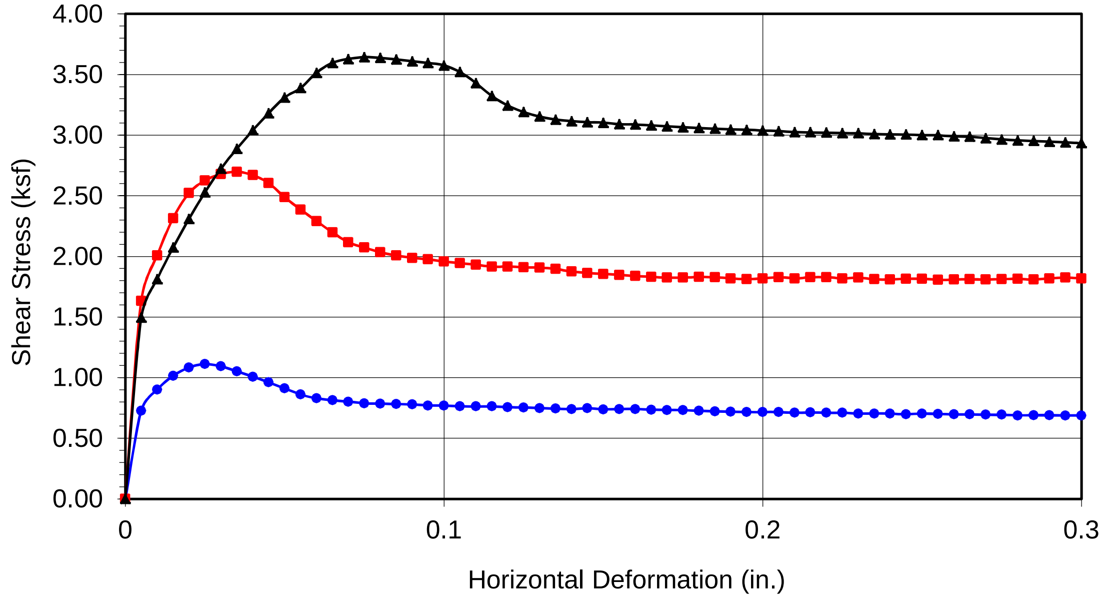
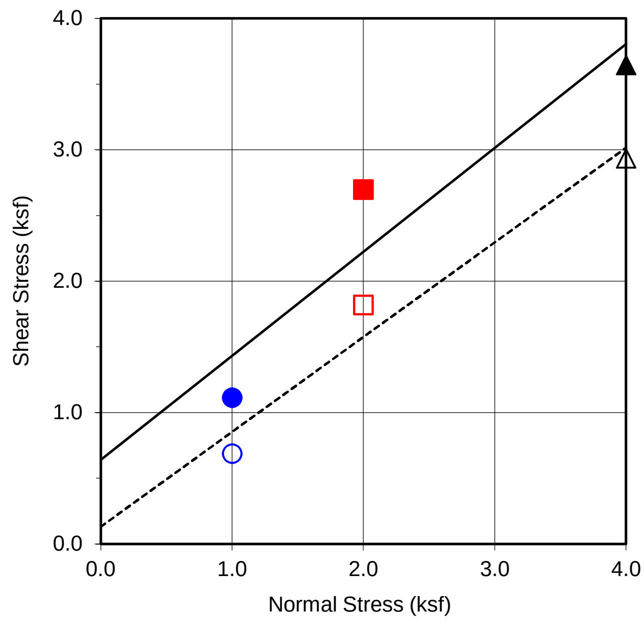
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 603541-002
SHARP CHULA VISTA MEDICAL CENTER
MASTER PLAN
05-13





Boring No.	**B-14**
Sample No.	**R-3**
Depth (ft)	**25-26.0**
Sample Type:	RING

Soil Identification:
POORLY GRADED SAND WITH SILT (SP-SM), light grayish brown.

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	639.0	38.3
Ultimate	130.5	35.8

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.112	■ 2.698	▲ 3.644
Shear Stress @ End of Test (ksf)	○ 0.688	□ 1.819	△ 2.934
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	13.29	14.01	11.03
Dry Density (pcf)	93.2	95.6	91.2
Saturation (%)	44.4	49.5	35.1
Soil Height Before Shearing (in.)	0.9962	0.9935	0.9706
Final Moisture Content (%)	29.3	26.8	28.3

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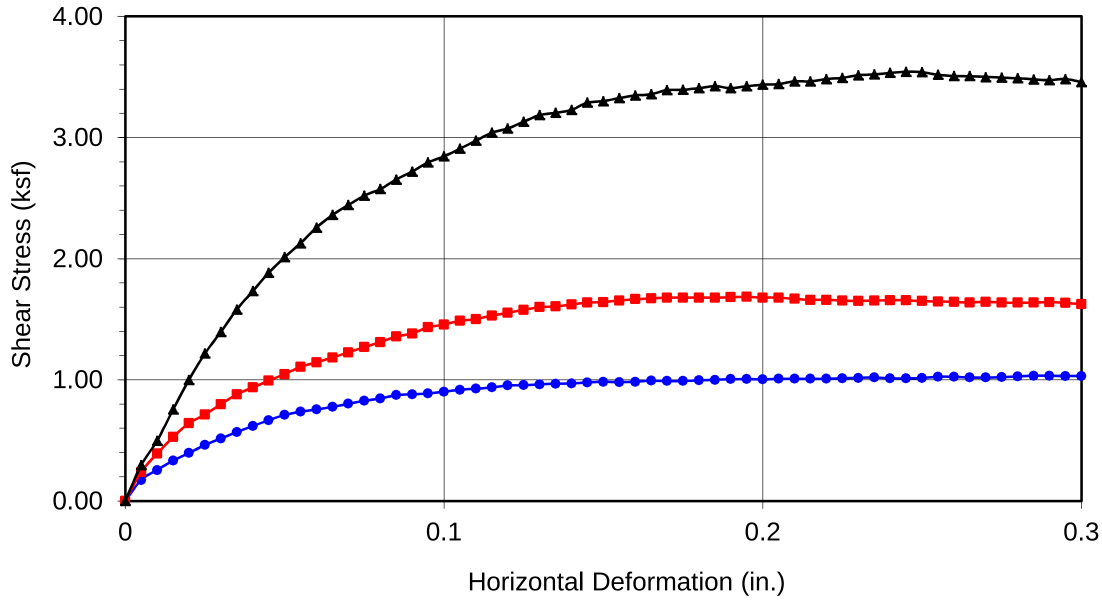
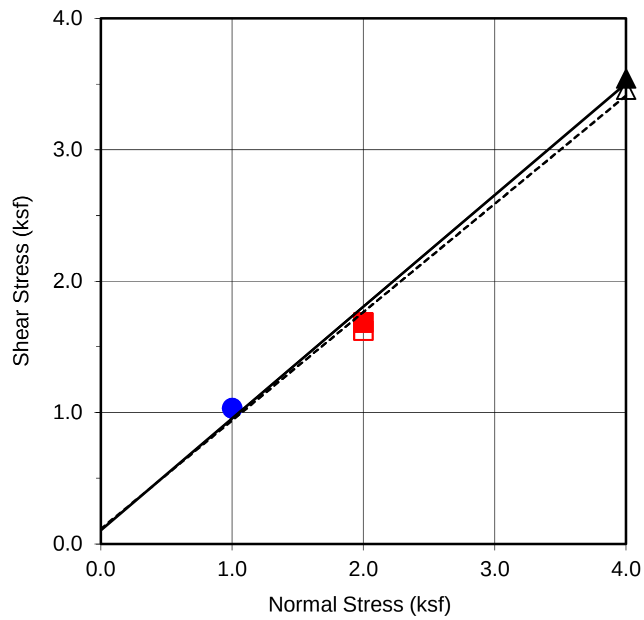
DIRECT SHEAR TEST RESULTS
Consolidated Drained ASTM D-3080

Project No.: 603541-002
SHARP CHULA VISTA MEDICAL CENTER
MASTER PLAN

06-13





Boring No.	**B-20**
Sample No.	**R-2**
Depth (ft)	**15-16**
Sample Type:	Ring

Soil Identification:
Light olive brown sandy silt
s(ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	105.0	40.4
Ultimate	114.5	39.5

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.034	■ 1.685	▲ 3.543
Shear Stress @ End of Test (ksf)	○ 1.031	□ 1.625	△ 3.458
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	7.39	7.39	7.39
Dry Density (pcf)	86.5	86.4	87.9
Saturation (%)	21.0	21.0	21.7
Soil Height Before Shearing (in.)	0.9773	0.9539	0.9584
Final Moisture Content (%)	24.5	25.5	24.9

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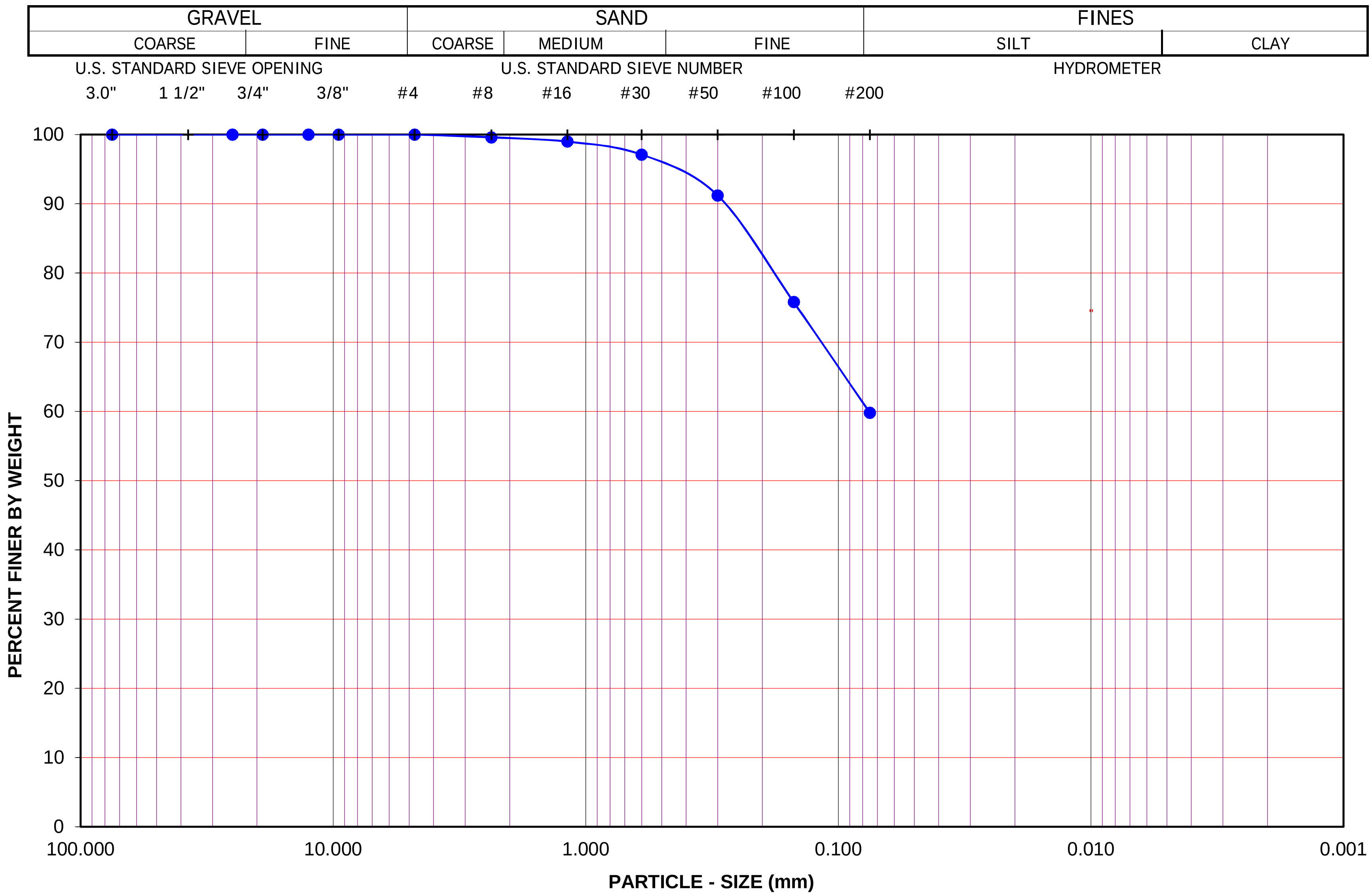
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 603541-002
SHARP CHULA VISTA MEDICAL CENTER
MASTER PLAN
05-13



Project Name: SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN

Project No.: 603541-002

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Exploration No.: B-1

Sample No.: B-1

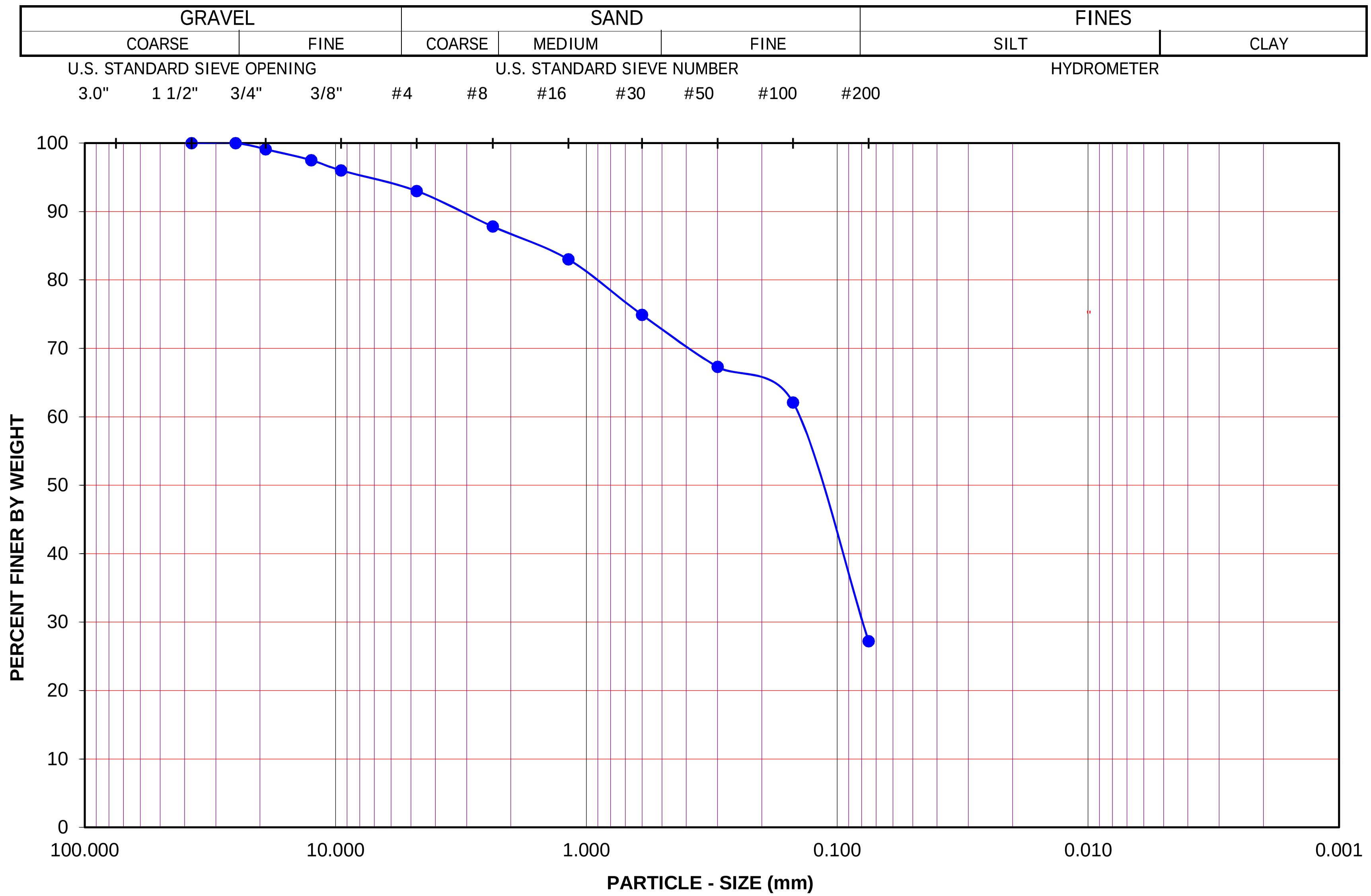
Depth (feet): 10-15.0

Soil Type : s(CL)

Soil Identification: SANDY LEAN CLAY s(CL), light olive brown.

GR:SA:FI : (%) 0 : 40 : 60

May-13



Project Name: SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN

Project No.: 603541-002

Leighton

PARTICLE - SIZE DISTRIBUTION
ASTM D 6913

Exploration No.: B-10 Sample No.: B-1

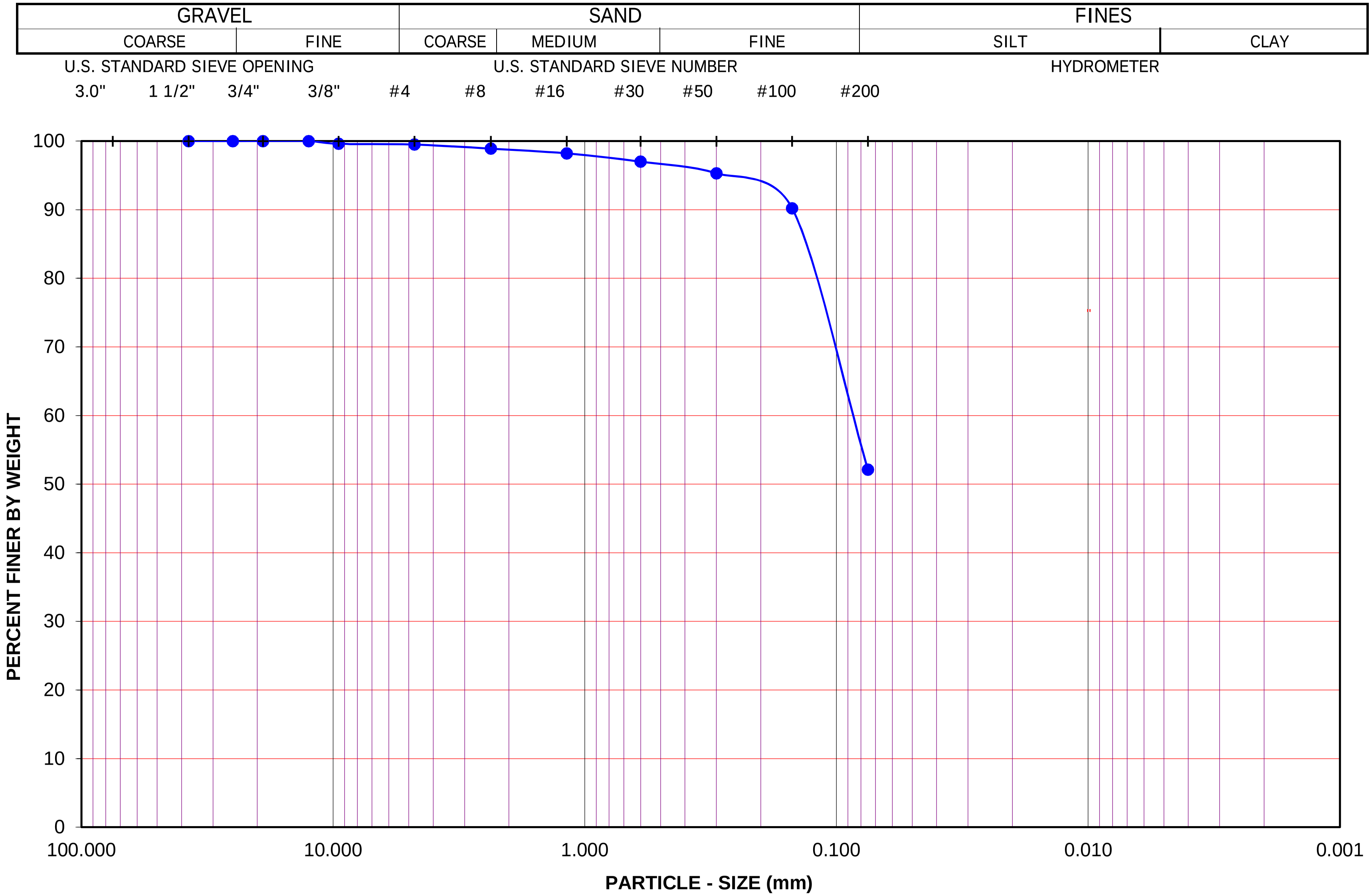
Depth (feet): 10-12.0 Soil Type : SM

Soil Identification: SILTY SAND WITH FEW GRAVEL (SM), olive brown.

GR:SA:FI : (%) 7 : 66 : 27

May-13

Split Sieve Landscape; B-10, B-1 (5-1-13)



Project Name: SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN

Project No.: 603541-002

Exploration No.: B-12 Sample No.: B-1

Depth (feet): 5-10.0 Soil Type : s(ML)

Soil Identification: SANDY SILT WITH TRACE GRAVEL s(ML), yellowish brown.

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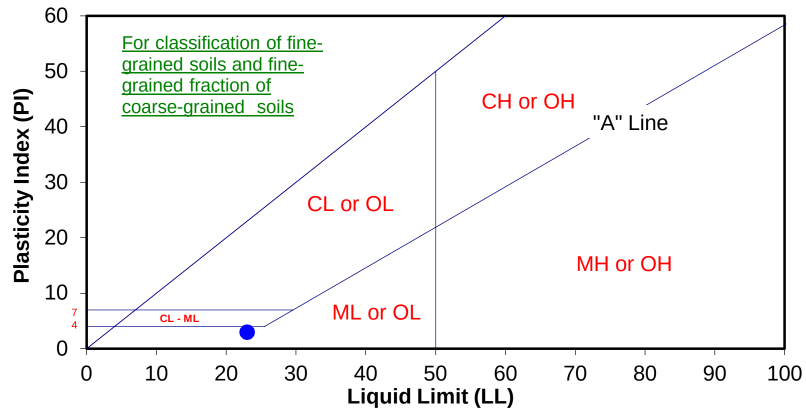
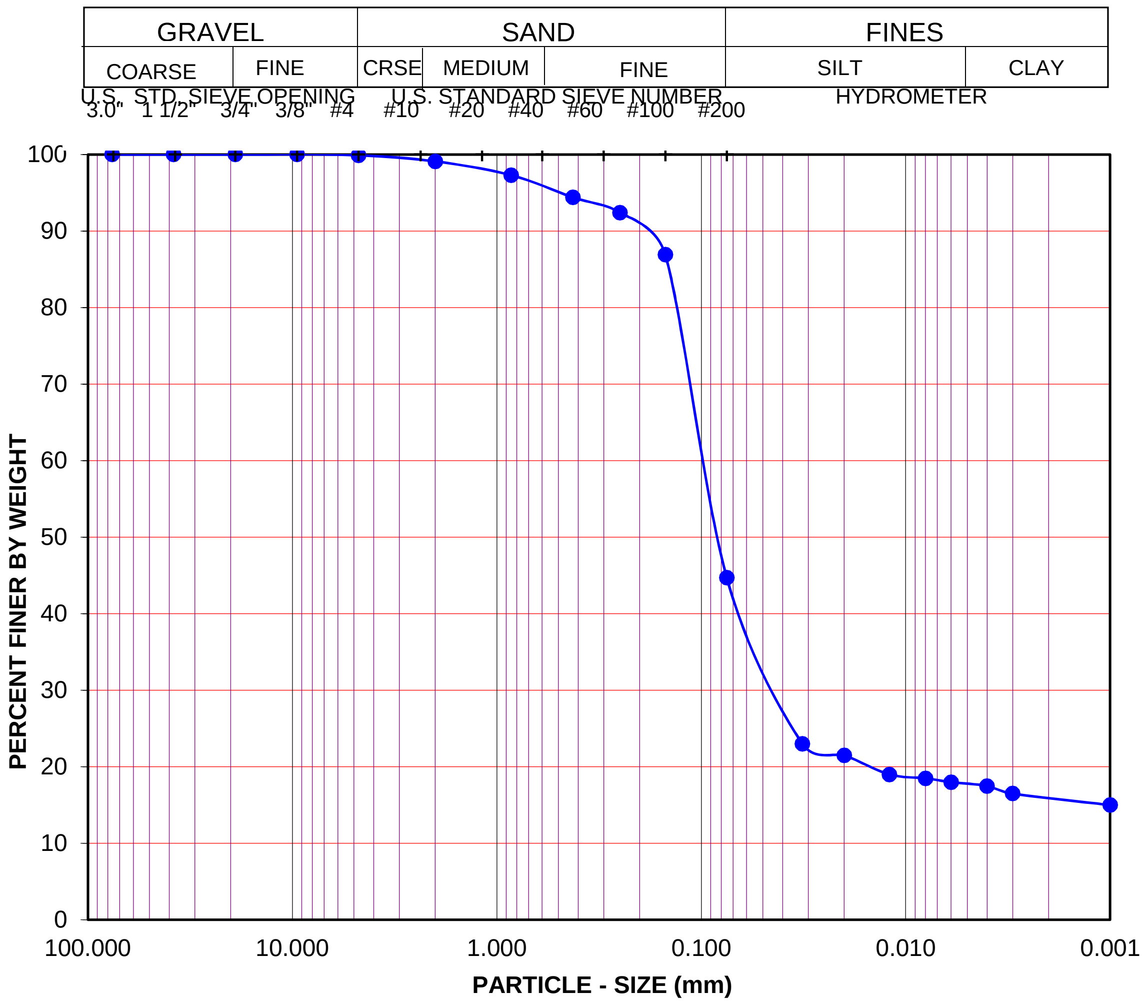
PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 1 : 47 : 52

May-13

Split Sieve Landscape; B-12, B-1 (5-1-13)





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
B-14	**S-2**	20-21.5	SM	0:55:45	23,20,3

Sample Description:
SILTY SAND WITH TRACE GRAVEL (SM), brown.

Leighton

Project No.: 603541-002
SHARP CHULA VISTA MEDICAL CENTER MASTER PLAN

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

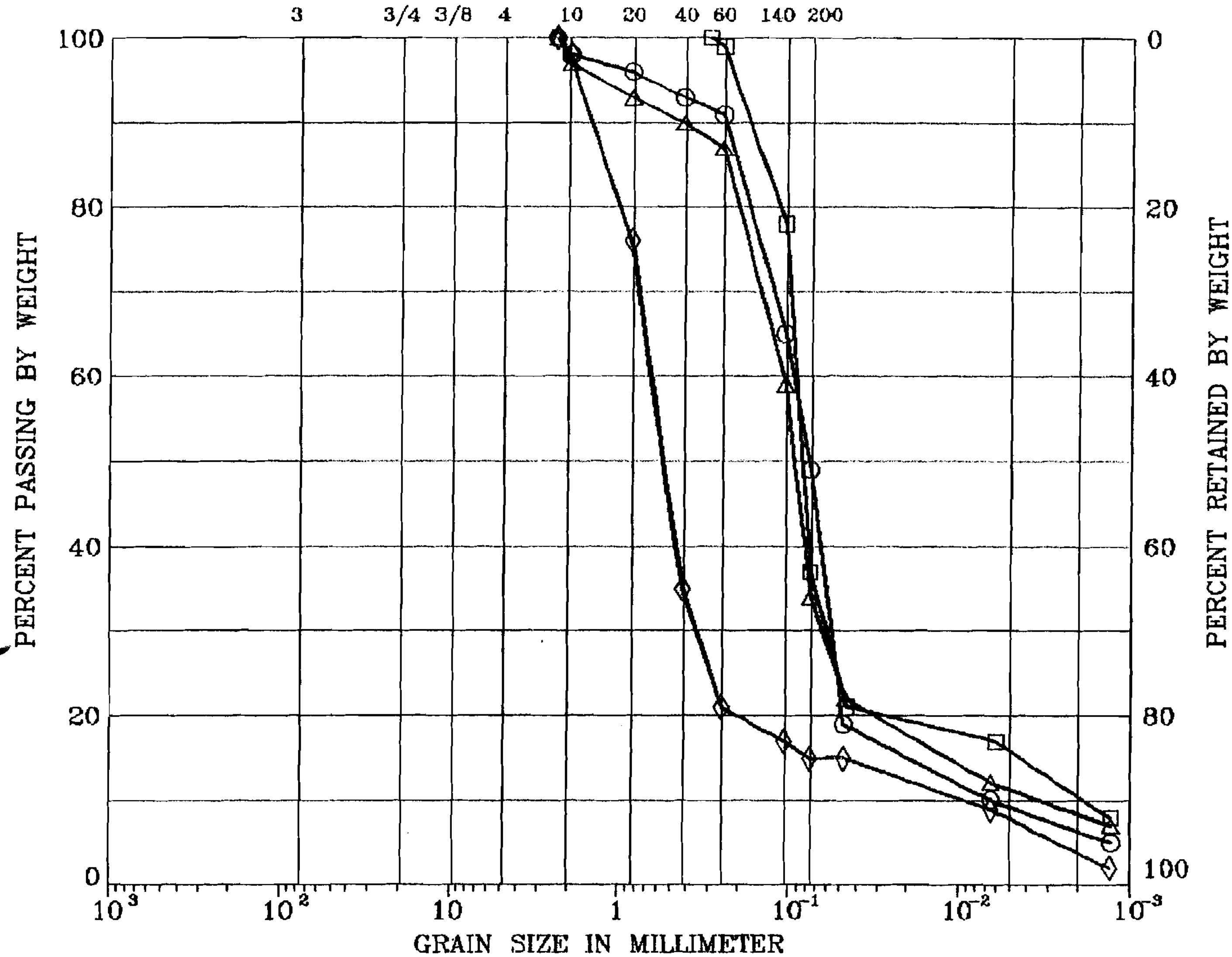
Rev. 08-04

Appendix C

Woodward-Clyde Laboratory Testing, 1989

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
U.S. SIEVE SIZE IN INCHES			U.S. STANDARD SIEVE No.			HYDROMETER

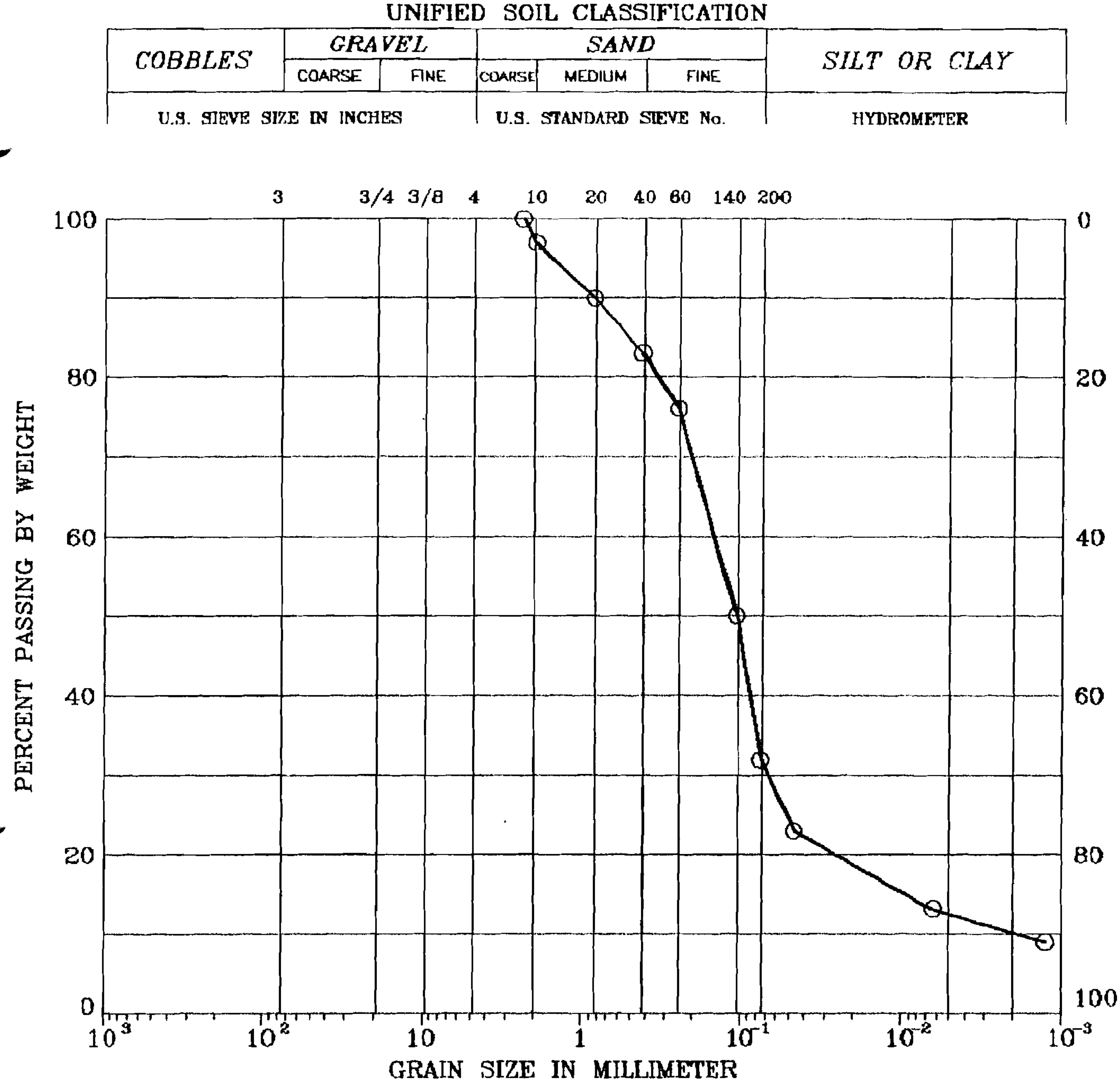


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	3–1				SILTY FINE SAND (SM)
□	4–6–4				SILTY FINE SAND (SM)
△	5–1				SILTY FINE SAND (SM)
◊	8–3–4				SILTY SAND (SM)

Remark :

?951127W SI01	CHULA VISTA COMMUNITY HOSPITAL
Woodward Clyde Consultants San Diego, CA	GRAIN SIZE DISTRIBUTION Figure No. B-1

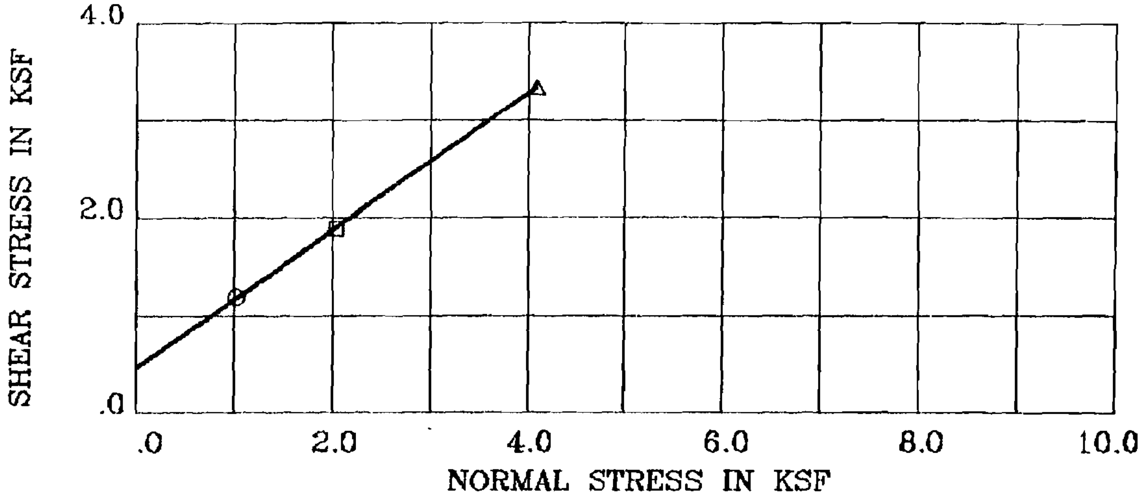
The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

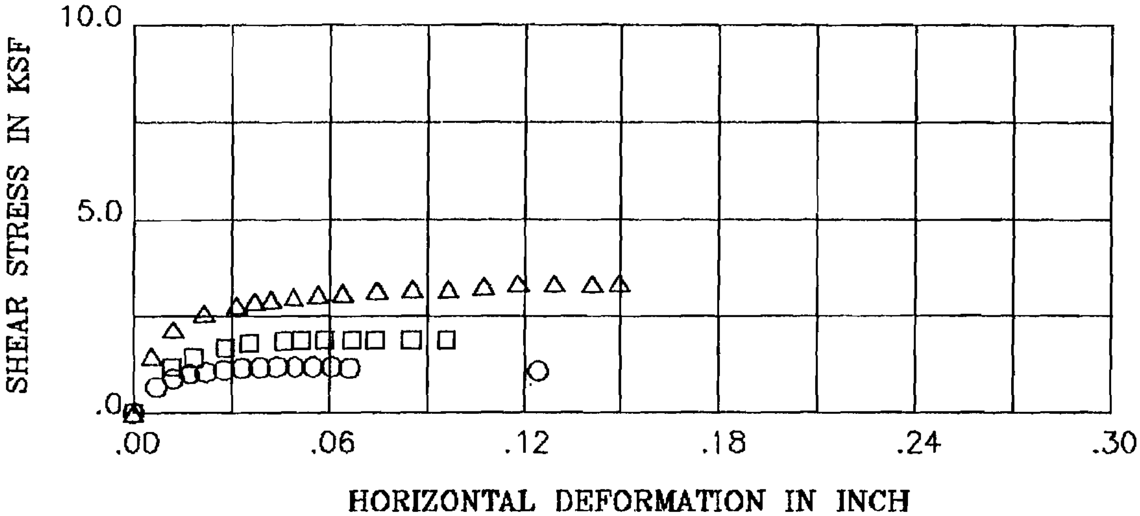


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	9-1				SILTY FINE SAND (SM)

Remark :

8951127W SI01	CHULA VISTA COMMUNITY HOSPITAL
Woodward Clyde Consultants San Diego, CA	**GRAIN SIZE DISTRIBUTION** Figure No. B-2





BORING/SAMPLE : 4-6-4 DEPTH (ft) :

DESCRIPTION : SILTY FINE SAND (SM)

STRENGTH INTERCEPT (C) : .467 KSF

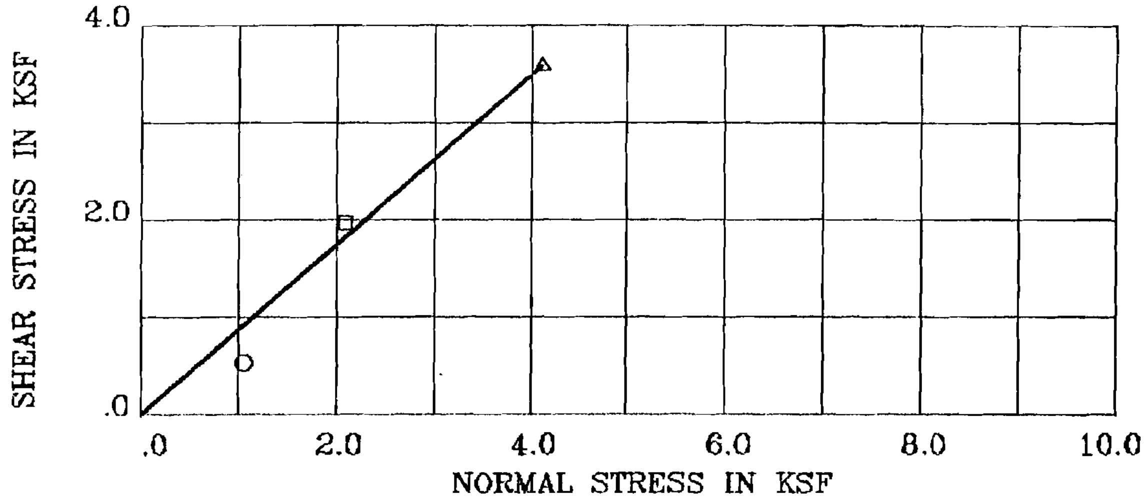
FRICTION ANGLE (PHI) : 35.0 DEG

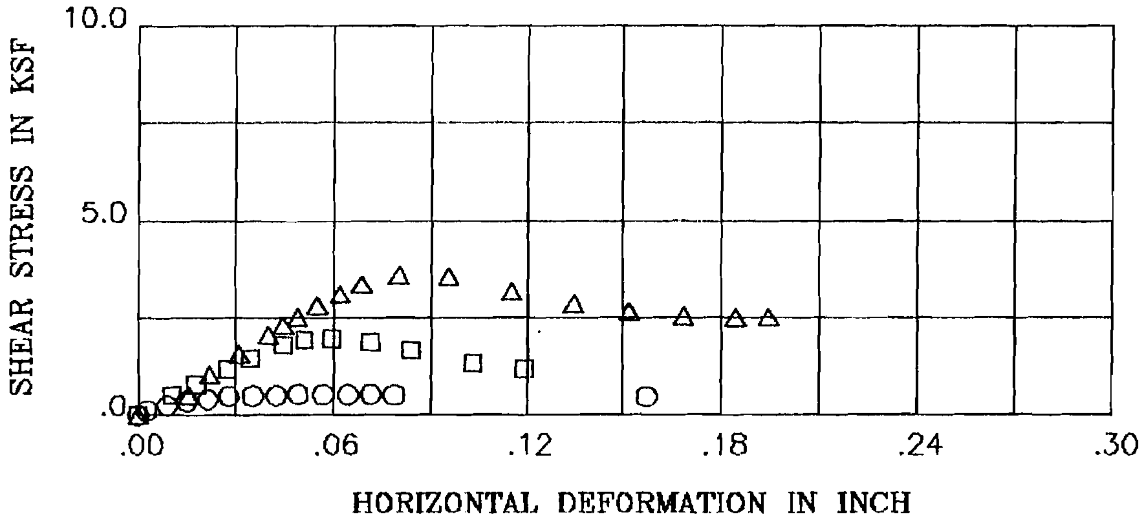
(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	27.8	96.3	.724	1.02	1.19	1.07
□	25.6	98.4	.687	2.05	1.89	1.87
△	28.3	95.8	.732	4.10	3.34	3.32

Remark : AVERAGE;INITIAL MC : 18.6 %; INITIAL DD : 96.2 PCF

8951127W SI01	CHULA VISTA COMMUNITY HOSPITAL
Woodward Clyde Consultants San Diego, CA	DIRECT SHEAR TEST Figure No. B-3





BORING/SAMPLE : 8-3-4 DEPTH (ft) :

DESCRIPTION : SILTY SAND (SM)

STRENGTH INTERCEPT (C) : .000 KSF

FRICTION ANGLE (PHI) : 41.1 DEG

(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	17.0	114.4	.446	1.05	.53	.46
□	16.0	112.4	.471	2.09	1.97	1.19
△	16.3	113.0	.463	4.12	3.61	2.50

Remark : AVERAGE;INITIAL MC : 8.1%; INITIAL DD : 111.4PCF

3951127W SI01	CHULA VISTA COMMUNITY HOSPITAL
Woodward Clyde Consultants San Diego, CA	DIRECT SHEAR TEST Figure No. B-4

The data developed and conclusions and recommendations reached in this/these correspondence(s) was/were for a specific project. Additionally, we wish to advise you that since this/these correspondence(s) was/were issued, there have been substantial changes in the state of the art and state of practice of the profession, as well as in the degree of risk considered acceptable by society and the profession. Thus, the opinion, recommendations, and conclusions contained therein must be reevaluated. We recommend that you retain a qualified firm to do this before proceeding with any plans that might be influenced by the contents of this/these correspondence(s).

Testing Engineers—San Diego
3467 Kurtz Street, P.O. Box 80985, San Diego, CA 92138 (619) 225-9641
2956 Industry St., Oceanside, CA 92054 (619) 757-0248

Job No: 001285 00
Job Name: WOODWARD - CLYDE CONSUL.
Job Address: 1550 HOTEL CIRCLE NORTH
SAN DIEGO
CA 92108

WOODWARD - CLYDE CONSUL.
1550 HOTEL CIRCLE NORTH
SAN DIEGO
CA 92108

WOODWARD - CLYDE CONSUL.
Testing Engineers - San Diego

Project: WOODWARD - CLYDE CONSUL.
Engineer: RENDINI, DAVID

Report: 56243
Date: 4/11/89

R VALUE DATA

	A	B	C	D
Compactor Pressure - P.S.I.	350	350	350	
Moisture @ Compaction - Percent	13.4	13.8	14.2	
Density - Pounds/Cubic Foot	117.9	116.0	116.2	
R-Value - Stabilometer	70	59	52	
Exude. Pressure - P.S.I.	430	270	220	
Stabilometer Thickness - Feet	.43	.59	.69	
Expansion Pressure Thickness - Feet	0	0	0	
T. I. (Assumed)	4.5			
By Stabilometer @ 300 PSI, Exud.	61			
By Expansion Pressure	/			
At Equilibrium	61			
Sand Equivalent	/			

Material Supplied by: Client

Submitted to Laboratory On: 4/04/89

Described As: Medium brown fine silty sand
R-Value #254/Lab #89-420

Sampled From: Sample #SAK/ 5-1 Depth 0.5
PROJECT: Chula vista Community Hospital 8955127W SI01

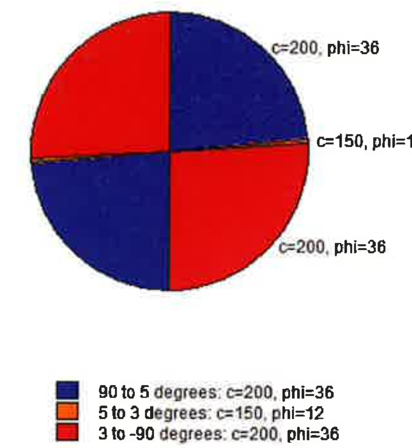
Figure 5

Appendix D

Slope Stability Calculations

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Anisotropic Function	Water Surface
To		110	Anisotropic function			Otay Formation Aniso	None
Tsdss		100	Mohr-Coulomb	100	39		None
Afu		120	Mohr-Coulomb	350	28		None

Otay Formation Aniso Model



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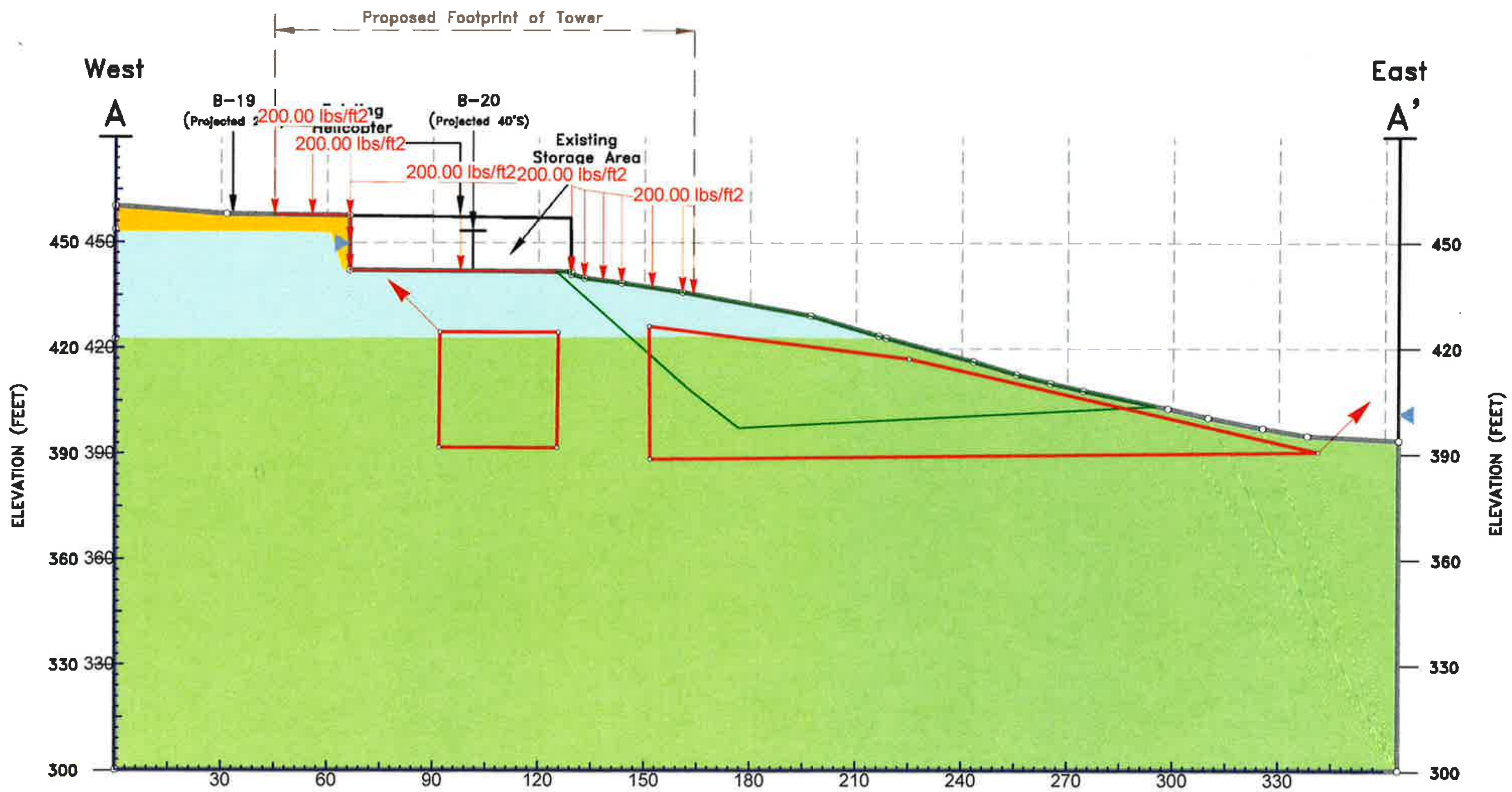


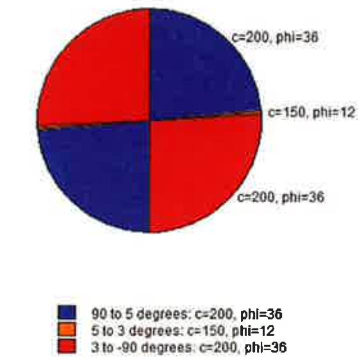
Project	Sharp CV Medical Center Section A-A'		
Analysis Description	Proj No. 603541-002		
Drawn By	FJW	Scale	1:500
Company	Leighton Consulting		
Date	7/15/2013, 11:00:43 AM	File Name	A-A' Static.slim

Leighton

SLIDEINTERPRET 6.023

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Anisotropic Function	Water Surface
To		110	Anisotropic function			Otay Formation Aniso	None
Tsdss		100	Mohr-Coulomb	100	39		None
Afu		120	Mohr-Coulomb	350	28		None

Otay Formation Aniso Model



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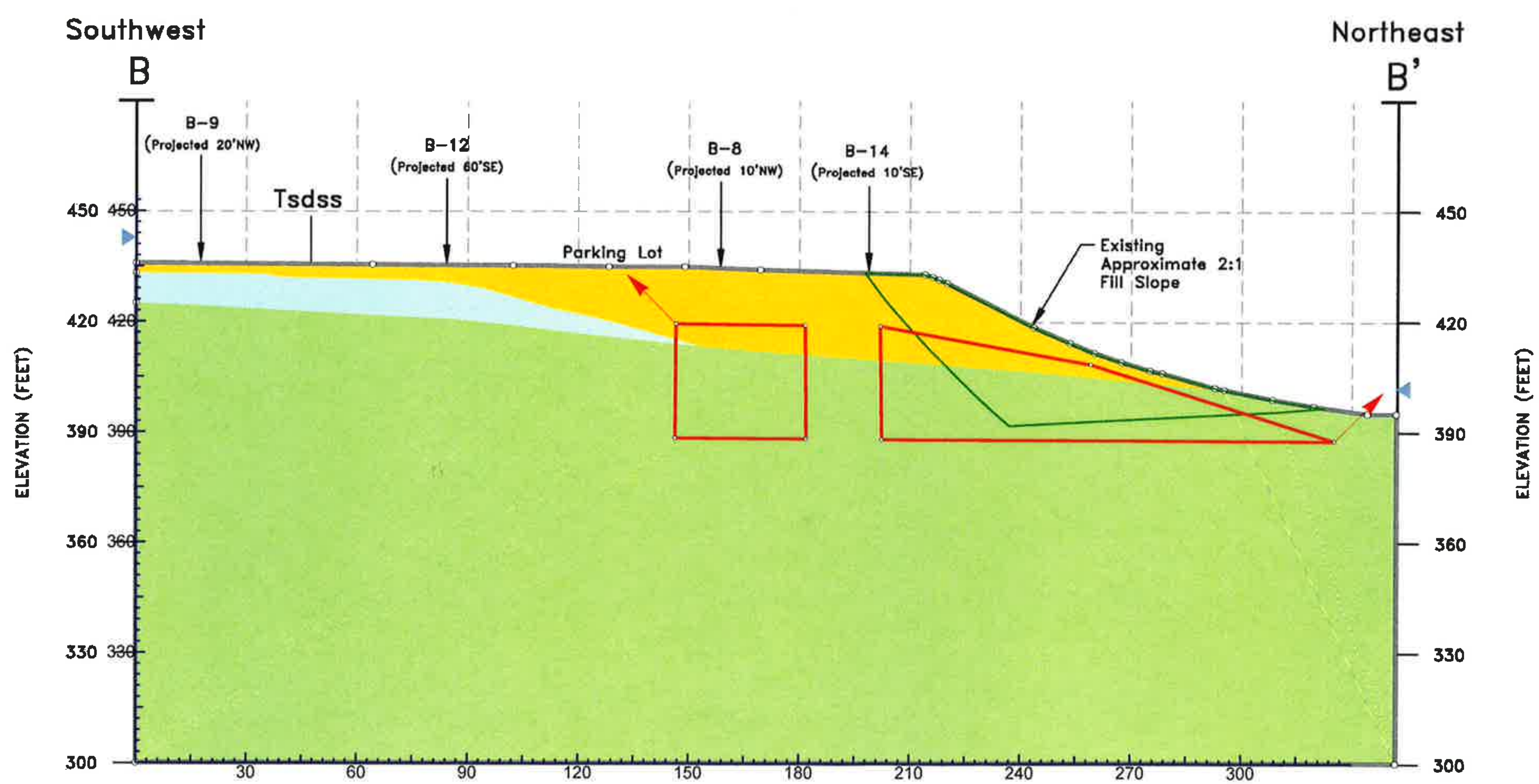


Leighton

Project	Sharp CV Medical Center Section B-B'
Analysis Description	Proj No. 603541-002
Drawn By	FJW
Scale	1:500
Company	Leighton Consulting
Date	7/15/2013, 2:00:37 PM
File Name	Section B-B' Static.slim

SLIDEINTERPRET 6.023

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geoenvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.262		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.14	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.21	seconds	
Moment Magnitude (Mw)	6.9		
Spectral Acceleration (Sa(1.5Ts))	0.752	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	5	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	4.98	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

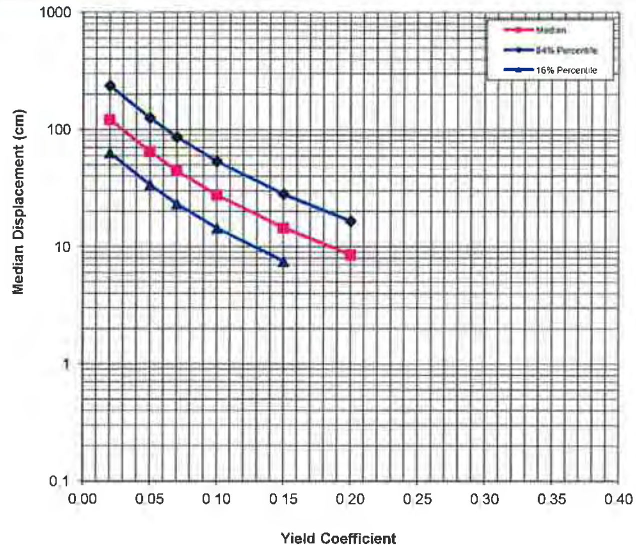
Results			
Probability of Negligible Displ. (P(D=0))	0.051		eq. (3)
D1	2.26	cm	calc. using eq. (7)
D2	4.77	cm	calc. using eq. (7)
D3	9.39	cm	calc. using eq. (7)
P(D>d_threshold)	0.473		eq. (7)

Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement)
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below
9. Examples of how Ts is estimated are shown below
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	121.9	121.9	234.9	63.2
0.05	0.00	64.7	64.7	124.7	33.6
0.07	0.00	44.6	44.6	85.9	23.1
0.1	0.00	27.6	27.6	53.3	14.3
0.15	0.00	14.5	14.5	27.9	7.5
0.2	0.01	8.6	8.5	16.5	4.4
0.3	0.12	3.7	3.3	6.8	1.3
0.4	0.40	1.9	1.0	2.9	<1



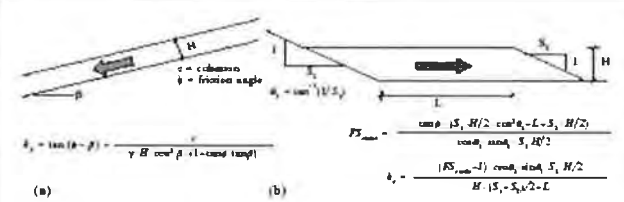


Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

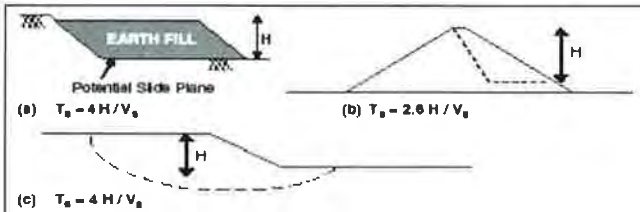
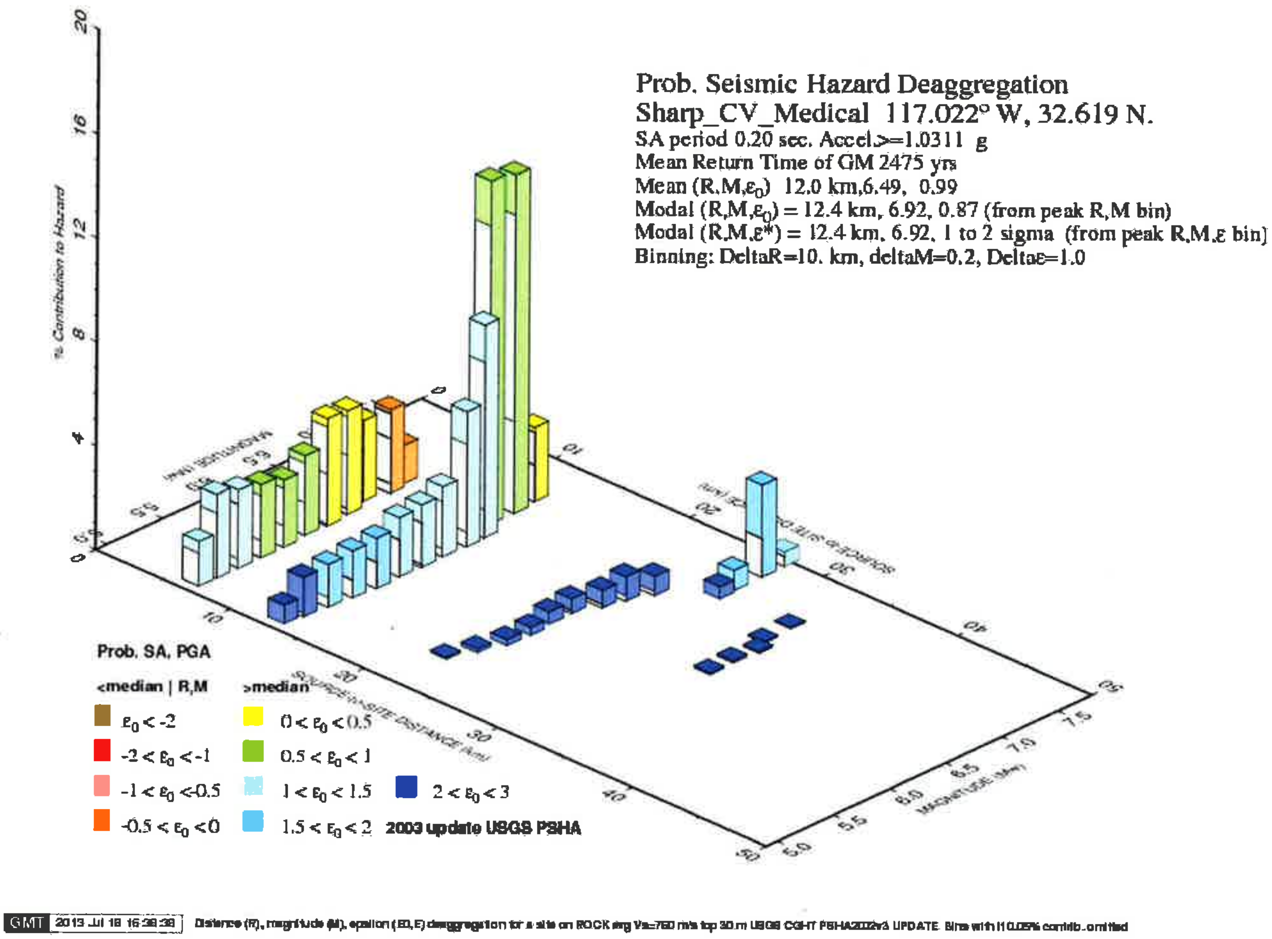
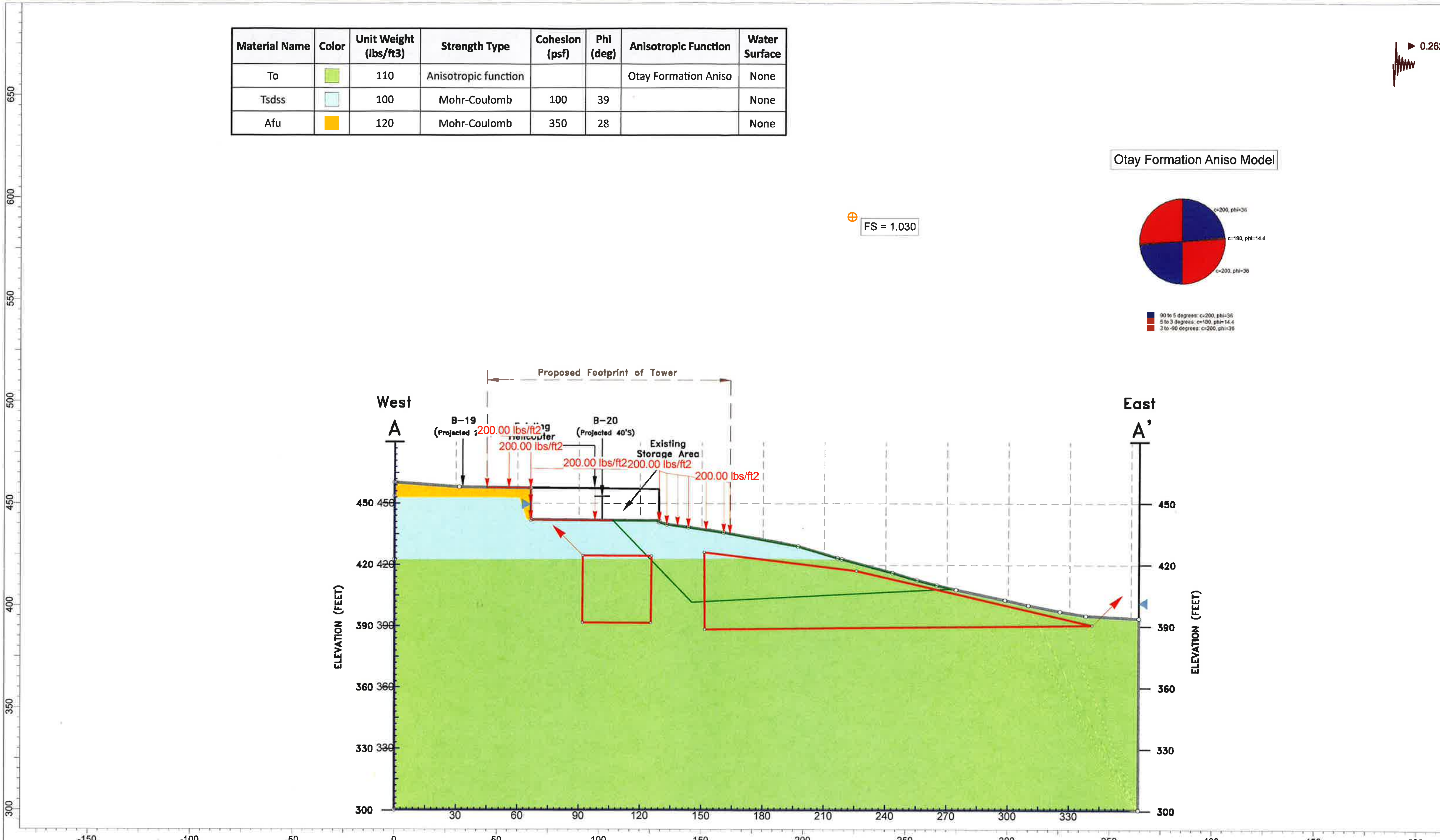


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

Figures from Bray, J.D. (2007) "Chapter 14: Simplified Seismic Slope Displacement Procedures," Earthquake Geotechnical Engineering, 4th Inter. Conf. on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, and Earthquake Engineering Series, Vol. 6, Pitilakis, Kyriazis D., Ed., Springer, Vol. 6, pp. 327-353


Prob. Seismic Hazard Deaggregation
Sharp_CV_Medical 117.022° W, 32.619 N.
SA period 0.20 sec. Accel.>=1.0311 g
Mean Return Time of GM 2475 yrs
Mean (R,M,ε0) 12.0 km,6.49, 0.99
Modal (R,M,ε0) = 12.4 km, 6.92, 0.87 (from peak R,M bin)
Modal (R,M,ε*) = 12.4 km, 6.92, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
SOURCE-to-SITE DISTANCE (km)
MAGNITUDE (Mw)
Prob. SA, PGA
<median | R,M
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 <0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 1.5
1.5 < ε0 < 2
2 < ε0 < 3
2003 update USGS PSHA
GMT 2013 Jul 18 16:38:38
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on ROCK avg Vs=760 m/s top 30 m USGS CGHT PSHA2002v3 UPDATE Bins with lt 0.05% contrib. omitted

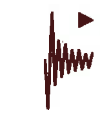
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Anisotropic Function	Water Surface
To		110	Anisotropic function			Otay Formation Aniso	None
Tsdss		100	Mohr-Coulomb	100	39		None
Afu		120	Mohr-Coulomb	350	28		None



Project	Sharp CV Medical Center Section A-A'		
Analysis Description	Proj No. 603541-002		
Drawn By	FJW	Scale	1:500
Company	Leighton Consulting		
Date	7/15/2013, 11:00:43 AM	File Name	A-A' Seismic a.slim

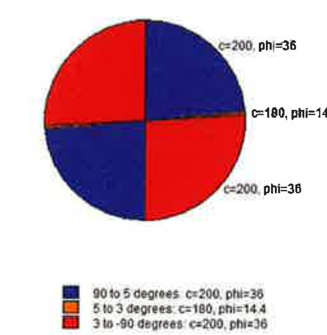
Leighton

SLIDEINTERPRET 6.023



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Anisotropic Function	Water Surface	Ru
To		110	Anisotropic function			Otay Formation Aniso	None	0
Tsdss		100	Mohr-Coulomb	100	39		None	0
Afu		120	Mohr-Coulomb	350	28		None	0

Otay Formation Aniso Model



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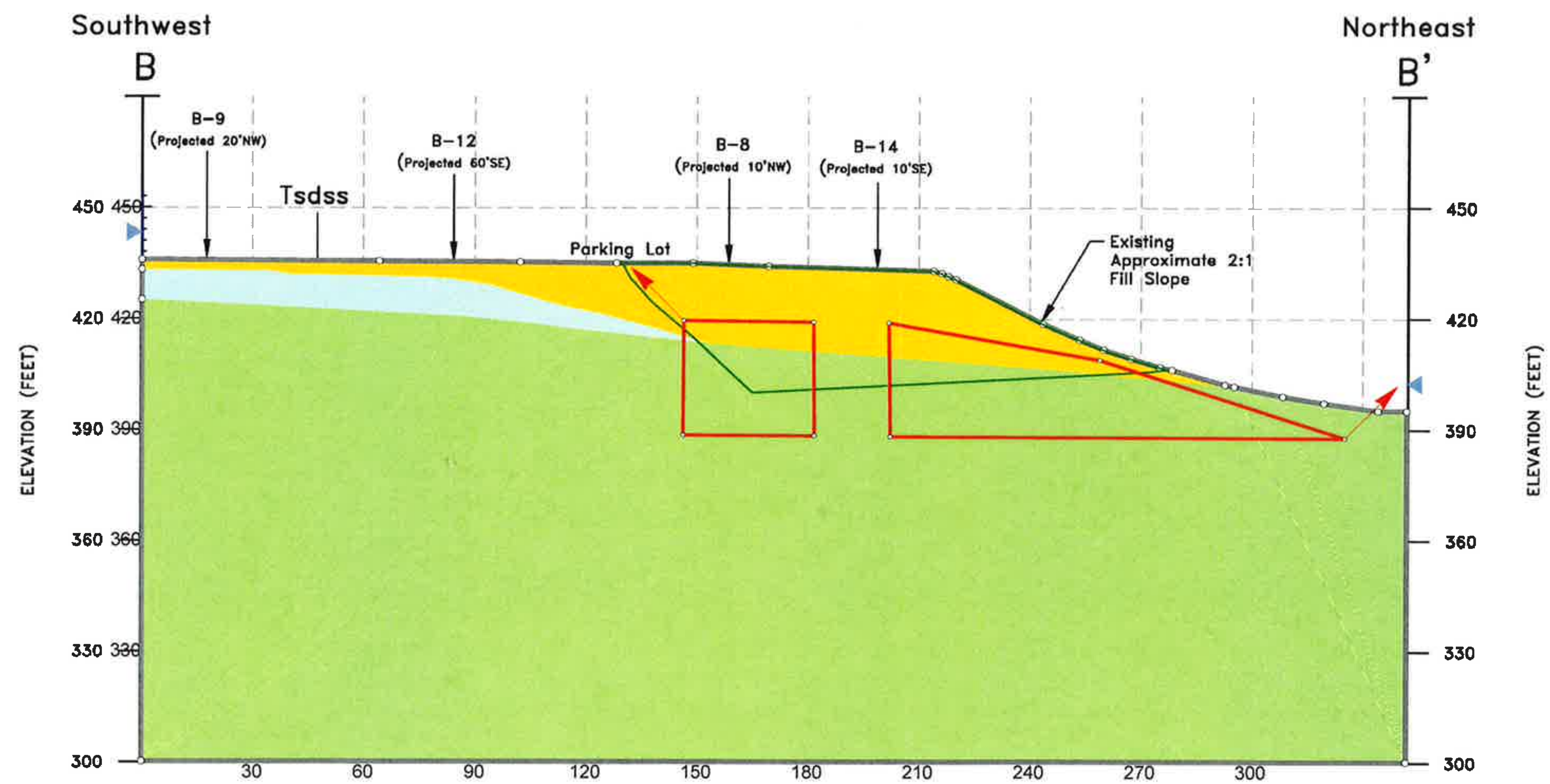


Project	Sharp CV Medical Center Section B-B'		
Analysis Description	Proj No. 603541-002		
Drawn By	FJW	Scale	1:500
Company	Leighton Consulting		
Date	7/15/2013, 2:00:37 PM	File Name	Section B-B' Seismic.slim

Leighton

SLIDEINTERPRET 6.023

Appendix E

Seismic Hazard Analysis

```
            ***********************
            *                     *
            *   E Q F A U L T     *
            *                     *
            *   Version 3.00      *
            *                     *
            ***********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 603541-002

DATE: 07-12-2013

JOB NAME: Sharp Chula Vista Master Plan

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 32.6191
SITE LONGITUDE: 117.0228

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 3) Boore et al. (1997) Horiz. - NEHRP D (250)
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: cd_2drp
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	7.5(12.1)	7.2	0.347	IX
CORONADO BANK	16.8(27.0)	7.6	0.244	IX
ELSINORE (JULIAN)	42.2(67.9)	7.1	0.093	VII
NEWPORT-INGLEWOOD (Offshore)	44.0(70.8)	7.1	0.090	VII
ELSINORE (COYOTE MOUNTAIN)	45.3(72.9)	6.8	0.075	VII
EARTHQUAKE VALLEY	45.7(73.6)	6.5	0.063	VI
ELSINORE (TEMECULA)	52.4(84.3)	6.8	0.067	VI
SAN JACINTO-COYOTE CREEK	62.4(100.4)	6.6	0.053	VI
SAN JACINTO - BORREGO	62.8(101.0)	6.6	0.052	VI
SAN JACINTO-ANZA	65.3(105.1)	7.2	0.070	VI
LAGUNA SALADA	66.8(107.5)	7.0	0.062	VI
SUPERSTITION MTN. (San Jacinto)	69.1(111.2)	6.6	0.049	VI
PALOS VERDES	70.3(113.2)	7.3	0.069	VI
ELSINORE (GLEN IVY)	73.3(117.9)	6.8	0.052	VI
ELMORE RANCH	73.7(118.6)	6.6	0.046	VI
SUPERSTITION HILLS (San Jacinto)	74.1(119.3)	6.6	0.046	VI
SAN JOAQUIN HILLS	75.4(121.3)	6.6	0.055	VI
SAN JACINTO-SAN JACINTO VALLEY	77.7(125.0)	6.9	0.052	VI
NEWPORT-INGLEWOOD (L.A.Basin)	86.1(138.5)	7.1	0.053	VI
IMPERIAL	87.1(140.2)	7.0	0.050	VI
CHINO-CENTRAL AVE. (Elsinore)	88.3(142.1)	6.7	0.051	VI
SAN ANDREAS - SB-Coach. M-1b-2	89.4(143.9)	7.7	0.071	VI
SAN ANDREAS - Whole M-1a	89.4(143.9)	8.0	0.083	VII
SAN ANDREAS - Coachella M-1c-5	89.4(143.9)	7.2	0.055	VI
SAN ANDREAS - SB-Coach. M-2b	89.4(143.9)	7.7	0.071	VI
BRAWLEY SEISMIC ZONE	90.3(145.3)	6.4	0.036	V
WHITTIER	92.5(148.9)	6.8	0.043	VI
SAN ANDREAS - San Bernardino M-1	95.2(153.2)	7.5	0.061	VI
SAN JACINTO-SAN BERNARDINO	97.4(156.7)	6.7	0.039	V
BURNT MTN.	98.1(157.8)	6.5	0.035	V

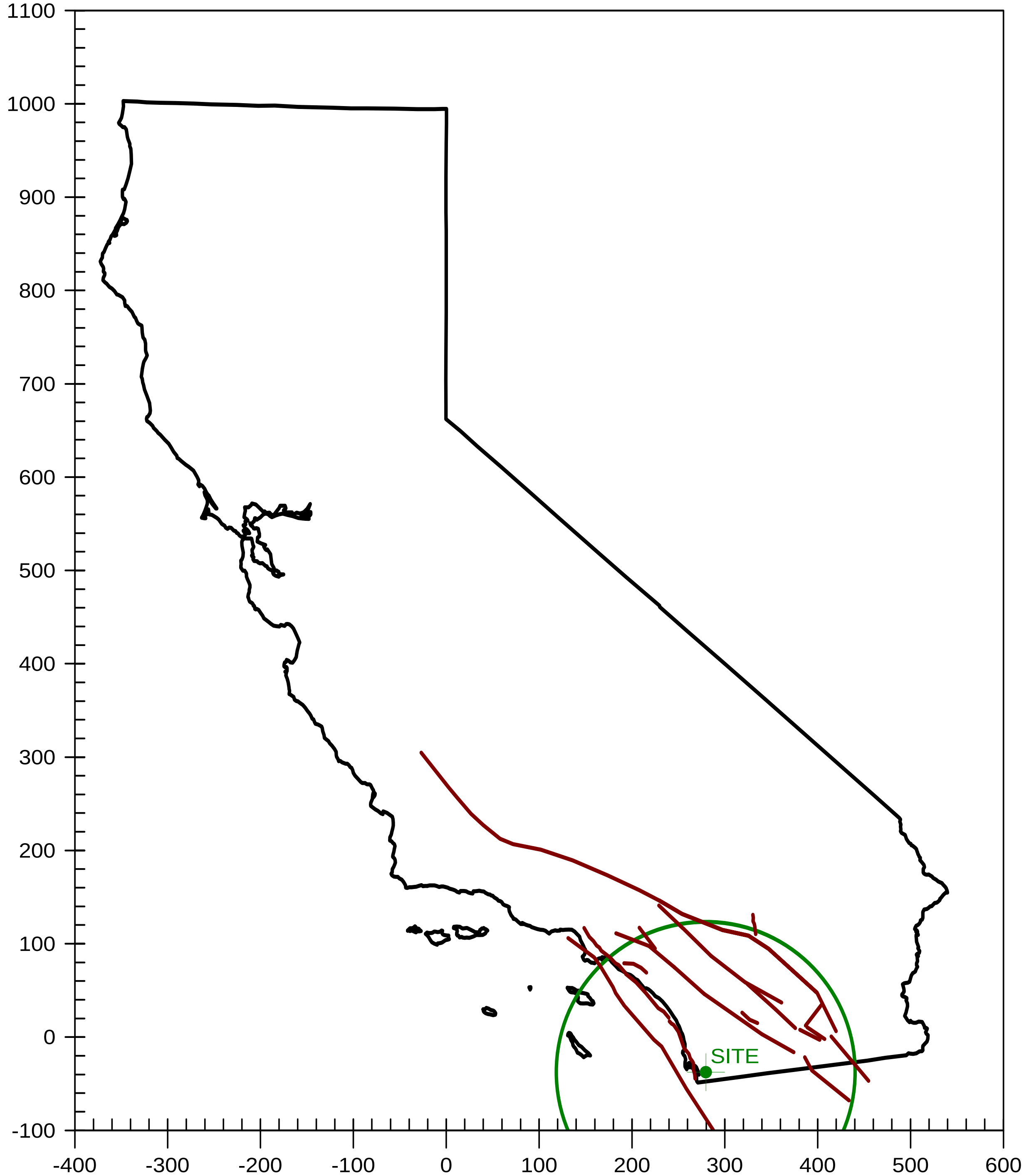
-END OF SEARCH- 30 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 7.5 MILES (12.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3475 g

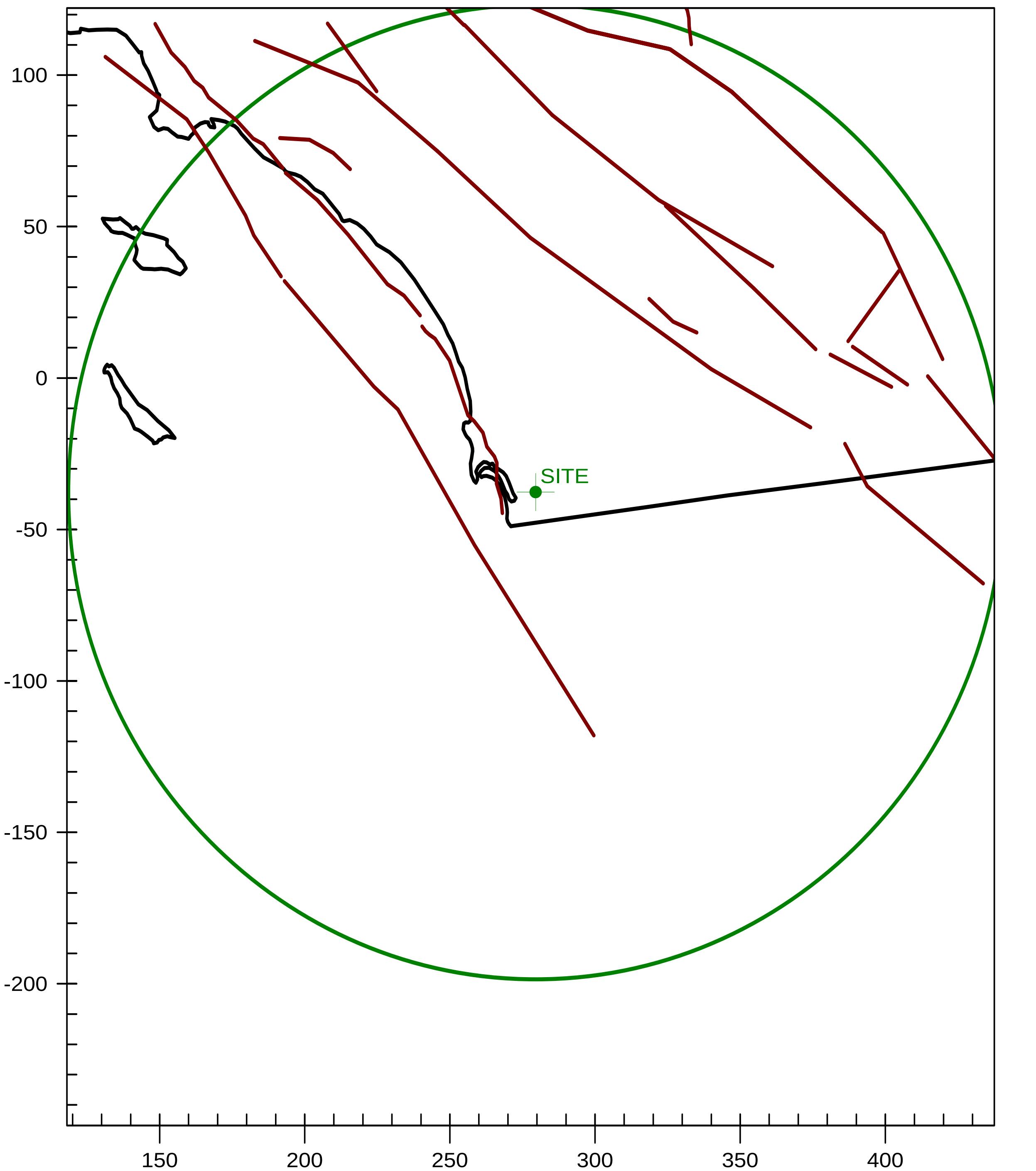
CALIFORNIA FAULT MAP

Sharp Chula Vista Master Plan



CALIFORNIA FAULT MAP

Sharp Chula Vista Master Plan



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                        *************************
                        *                       *
                        *    E Q S E A R C H    *
                        *                       *
                        *     Version 3.00      *
                        *                       *
                        *************************

                             ESTIMATION OF
                        PEAK ACCELERATION FROM
                    CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 603541-002
                                                     DATE: 07-15-2013

JOB NAME: Sharp Chula Vista Master Plan

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  32.6191
   SITE LONGITUDE:  117.0228

SEARCH DATES:
           START DATE:   1800
           END DATE:   2013

SEARCH RADIUS:
           100.0 mi
           160.9 km

ATTENUATION RELATION:   3) Boore et al. (1997) Horiz. - NEHRP D (250)
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   0  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:        Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  0.0
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EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.116	VII	9.2(14.9)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.116	VII	9.2(14.9)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.116	VII	9.2(14.9)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.157	VIII	11.7(18.8)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.090	VII	13.3(21.3)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.103	VII	18.0(28.9)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.054	VI	26.3(42.4)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.105	VII	30.8(49.6)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.041	V	37.7(60.7)
DMG	32.2000	116.5500	11/05/1949	43524.0	0.0	5.10	0.041	V	40.0(64.3)
DMG	32.2000	116.5500	11/04/1949	204238.0	0.0	5.70	0.056	VI	40.0(64.3)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.091	VII	42.4(68.2)
DMG	32.0830	116.6670	11/25/1934	818 0.0	0.0	5.00	0.037	V	42.4(68.3)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.039	V	43.2(69.5)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.036	V	44.3(71.2)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.040	V	47.0(75.6)
DMG	32.0000	117.5000	06/24/1939	1627 0.0	0.0	5.00	0.032	V	51.0(82.1)
DMG	32.0000	117.5000	05/01/1939	2353 0.0	0.0	5.00	0.032	V	51.0(82.1)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.036	V	54.8(88.3)
GSP	32.3290	117.9170	06/15/2004	222848.2	10.0	5.30	0.035	V	55.8(89.8)
DMG	31.8110	117.1310	12/22/1964	205433.2	2.3	5.60	0.041	V	56.1(90.4)
DMG	31.8670	116.5710	02/27/1937	12918.4	10.0	5.00	0.029	V	58.2(93.7)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.055	VI	62.3(100.3)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.035	V	63.3(101.9)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.041	V	63.5(102.2)
DMG	32.9670	116.0000	10/22/1942	181326.0	0.0	5.00	0.027	V	64.0(103.1)
DMG	32.9670	116.0000	10/21/1942	162654.0	0.0	5.00	0.027	V	64.0(103.1)
DMG	32.9670	116.0000	10/21/1942	162519.0	0.0	5.00	0.027	V	64.0(103.1)
DMG	32.9670	116.0000	10/21/1942	162213.0	0.0	6.50	0.059	VI	64.0(103.1)
GSG	32.7000	115.9210	06/15/2010	042658.5	5.0	5.80	0.041	V	64.3(103.5)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.056	VI	65.1(104.8)
DMG	32.9830	115.9830	05/23/1942	154729.0	0.0	5.00	0.027	V	65.4(105.2)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.038	V	66.1(106.3)
DMG	33.1130	116.0370	04/09/1968	3 353.5	5.0	5.20	0.029	V	66.6(107.1)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.049	VI	66.8(107.6)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.027	V	66.8(107.6)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.034	V	66.8(107.6)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.026	V	66.8(107.6)
DMG	31.7500	116.5000	04/29/1935	20 8 0.0	0.0	5.00	0.026	V	67.3(108.4)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.034	V	67.7(108.9)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.026	V	67.9(109.3)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.029	V	68.0(109.5)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.027	V	68.1(109.5)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.051	VI	68.2(109.8)
GSP	32.6520	115.8350	05/19/2010	003900.0	7.0	5.10	0.027	V	69.1(111.2)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.043	VI	70.1(112.8)
GSG	32.6750	115.8060	05/08/2010	183311.0	6.0	5.00	0.025	V	70.8(114.0)
GSP	32.6400	115.8010	04/05/2010	133305.4	0.0	5.10	0.026	V	71.1(114.3)
DMG	31.7960	116.2690	06/11/1963	152338.3	-2.0	5.80	0.038	V	71.9(115.7)
GSP	32.6340	115.7820	04/05/2010	031525.2	3.0	5.00	0.025	V	72.2(116.1)
DMG	33.2310	116.0040	05/26/1957	155933.6	15.1	5.00	0.024	V	72.6(116.8)
GSG	32.6160	115.7730	05/22/2010	173058.8	3.0	5.10	0.026	V	72.7(117.0)
PAS	33.0130	115.8390	11/24/1987	131556.5	2.4	6.00	0.041	V	73.9(118.9)

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.0000	115.8330	01/08/1946	185418.0	0.0	5.40	0.030	V	73.9(118.9)
DMG	33.0330	115.8210	09/30/1971	224611.3	8.0	5.10	0.025	V	75.4(121.3)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.024	IV	75.5(121.5)
GSG	31.8060	116.1280	03/23/1994	025916.2	22.0	5.00	0.023	IV	76.7(123.4)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.023	IV	77.7(125.1)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.023	IV	77.7(125.1)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.039	V	77.7(125.1)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.023	IV	78.1(125.7)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.060	VI	78.1(125.7)
DMG	31.8000	116.1000	10/10/1953	1849 6.0	0.0	5.00	0.023	IV	78.1(125.7)
DMG	32.8170	118.3500	12/26/1951	04654.0	0.0	5.90	0.037	V	78.3(126.0)
DMG	33.1830	115.8500	04/25/1957	222412.0	0.0	5.10	0.024	V	78.3(126.1)
DMG	32.2500	115.7500	12/01/1958	6 2 0.0	0.0	5.50	0.030	V	78.4(126.2)
DMG	32.2500	115.7500	12/01/1958	350 0.0	0.0	5.00	0.023	IV	78.4(126.2)
DMG	32.2500	115.7500	12/01/1958	32118.0	0.0	5.80	0.035	V	78.4(126.2)
DMG	32.9830	115.7330	01/24/1951	717 2.6	0.0	5.60	0.031	V	79.0(127.1)
PAS	33.0820	115.7750	11/24/1987	15414.5	4.9	5.80	0.035	V	79.1(127.3)
DMG	32.9500	115.7170	06/14/1953	41729.9	0.0	5.50	0.030	V	79.2(127.4)
DMG	32.9000	115.7000	10/02/1928	19 1 0.0	0.0	5.00	0.023	IV	79.2(127.5)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.030	V	79.7(128.3)
DMG	31.8330	116.0000	05/10/1956	114854.0	0.0	5.00	0.023	IV	80.7(129.9)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.047	VI	81.5(131.2)
DMG	33.2160	115.8080	04/25/1957	215738.7	-0.3	5.20	0.025	V	81.6(131.3)
DMG	31.5000	116.5000	10/17/1954	225718.0	0.0	5.70	0.032	V	83.1(133.7)
DMG	31.6250	116.2110	06/10/1969	34132.7	-2.0	5.00	0.022	IV	83.4(134.3)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.024	IV	86.3(138.8)
PAS	31.8900	115.8210	05/08/1985	234020.8	6.0	5.00	0.021	IV	86.4(139.0)
DMG	33.2330	115.7170	10/22/1942	15038.0	0.0	5.50	0.028	V	86.7(139.6)
PAS	31.9270	115.7770	07/17/1975	182447.0	17.3	5.00	0.021	IV	87.0(140.0)
PAS	33.0980	115.6320	04/26/1981	12 928.4	3.8	5.70	0.031	V	87.2(140.3)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.042	VI	87.9(141.5)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.021	IV	88.1(141.8)
DMG	31.7500	115.9170	02/10/1956	15 929.0	0.0	5.00	0.021	IV	88.2(141.9)
DMG	31.7500	115.9170	02/09/1956	165953.0	0.0	5.70	0.030	V	88.2(141.9)
DMG	31.7500	115.9170	03/09/1956	03240.0	0.0	5.00	0.021	IV	88.2(141.9)
DMG	31.7500	115.9170	02/09/1956	152426.0	0.0	6.10	0.038	V	88.2(141.9)
DMG	31.7500	115.9170	02/11/1956	25746.0	0.0	5.10	0.022	IV	88.2(141.9)
DMG	31.7500	115.9170	02/09/1956	184845.0	0.0	5.70	0.030	V	88.2(141.9)
DMG	31.7500	115.9170	02/11/1956	61124.0	0.0	5.00	0.021	IV	88.2(141.9)
DMG	31.7500	115.9170	02/09/1956	143238.0	0.0	6.80	0.054	VI	88.2(141.9)
DMG	31.7500	115.9170	02/10/1956	181254.0	0.0	5.50	0.027	V	88.2(141.9)
GSP	33.1600	115.6370	09/02/2005	012719.8	9.0	5.10	0.022	IV	88.6(142.6)
PAS	32.9270	115.5400	10/16/1979	54910.2	10.4	5.10	0.022	IV	88.7(142.7)
DMG	31.6000	116.1000	11/26/1955	1736 0.0	0.0	5.40	0.026	V	88.7(142.7)
MGI	32.7000	115.5000	01/01/1927	13 0 0.0	0.0	5.30	0.025	V	88.7(142.7)
PAS	32.9280	115.5390	10/16/1979	61948.7	9.2	5.10	0.022	IV	88.7(142.8)
DMG	32.7330	115.5000	05/19/1940	43640.9	0.0	6.70	0.051	VI	88.8(143.0)
DMG	32.5000	115.5000	11/05/1923	22 7 0.0	0.0	5.00	0.021	IV	89.0(143.2)
DMG	32.5000	115.5000	04/19/1906	030 0.0	0.0	6.00	0.035	V	89.0(143.2)
DMG	32.5000	115.5000	09/08/1921	1924 0.0	0.0	5.00	0.021	IV	89.0(143.2)
DMG	32.5000	115.5000	11/07/1923	2357 0.0	0.0	5.50	0.027	V	89.0(143.2)
DMG	32.5000	115.5000	05/01/1918	432 0.0	0.0	5.00	0.021	IV	89.0(143.2)
MGI	32.5000	115.5000	04/16/1925	520 0.0	0.0	5.30	0.024	V	89.0(143.2)

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	32.5000	115.5000	01/01/1927	91330.0	0.0	5.50	0.027	V	89.0(143.2)
DMG	32.5000	115.5000	01/01/1927	81645.0	0.0	5.75	0.031	V	89.0(143.2)
MGI	32.5000	115.5000	04/16/1925	330 0.0	0.0	5.00	0.021	IV	89.0(143.2)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.035	V	89.0(143.3)
DMG	32.5000	118.5500	02/24/1948	81510.0	0.0	5.30	0.024	V	89.2(143.6)
DMG	32.8000	115.5000	06/23/1915	359 0.0	0.0	6.25	0.040	V	89.3(143.8)
DMG	32.8000	115.5000	06/23/1915	456 0.0	0.0	6.25	0.040	V	89.3(143.8)
PAS	33.0140	115.5550	10/16/1979	65842.8	9.1	5.50	0.027	V	89.4(143.9)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.022	IV	89.7(144.4)
DMG	32.7670	115.4830	05/19/1940	63320.0	0.0	5.00	0.021	IV	90.0(144.9)
DMG	32.7670	115.4830	05/19/1940	63540.0	0.0	5.50	0.027	V	90.0(144.9)
DMG	32.7670	115.4830	05/19/1940	455 0.0	0.0	5.50	0.027	V	90.0(144.9)
DMG	32.7670	115.4830	05/19/1940	55134.0	0.0	5.50	0.027	V	90.0(144.9)
GSP	31.7030	115.9100	12/03/1991	175435.8	5.0	5.30	0.024	V	90.7(146.0)
DMG	33.1170	115.5670	07/28/1950	175048.0	0.0	5.40	0.025	V	91.2(146.7)
DMG	33.1170	115.5670	07/29/1950	143632.0	0.0	5.50	0.027	V	91.2(146.7)
DMG	31.7000	115.9000	02/09/1956	1434 0.0	0.0	5.60	0.028	V	91.3(146.9)
DMG	31.7000	115.9000	02/11/1956	519 0.0	0.0	5.00	0.020	IV	91.3(146.9)
DMG	33.0000	115.5000	12/17/1955	6 729.0	0.0	5.40	0.025	V	92.2(148.4)
DMG	33.0000	115.5000	02/26/1930	230 0.0	0.0	5.00	0.020	IV	92.2(148.4)
DMG	32.2500	115.5000	12/30/1934	1352 0.0	0.0	6.50	0.045	VI	92.3(148.6)
T-A	33.5000	115.8200	05/00/1868	0 0 0.0	0.0	6.30	0.040	V	92.4(148.7)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.020	IV	92.4(148.7)
PAS	32.7660	115.4410	10/15/1979	231930.0	9.3	5.20	0.023	IV	92.5(148.8)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.026	V	94.5(152.0)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.021	IV	95.3(153.3)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.021	IV	96.0(154.5)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.021	IV	96.0(154.5)
DMG	33.1670	115.5000	12/20/1935	745 0.0	0.0	5.00	0.020	IV	96.0(154.6)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.038	V	96.2(154.9)
PAS	31.7130	115.7670	01/25/1988	131710.6	6.0	5.60	0.027	V	96.4(155.2)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.023	IV	96.6(155.5)
GSP	33.8760	116.2670	06/29/1992	160142.8	1.0	5.20	0.022	IV	97.1(156.3)
DMG	31.5000	116.0000	10/24/1954	944 8.0	0.0	6.00	0.033	V	97.7(157.3)
DMG	31.5000	116.0000	11/12/1954	131642.0	0.0	5.00	0.019	IV	97.7(157.3)
DMG	31.5000	116.0000	11/14/1954	53619.0	0.0	5.40	0.024	V	97.7(157.3)
DMG	31.5000	116.0000	10/24/1954	112124.0	0.0	5.40	0.024	V	97.7(157.3)
DMG	31.5000	116.0000	11/12/1954	122647.0	0.0	6.30	0.039	V	97.7(157.3)
PAS	32.0840	115.4710	01/10/1976	125815.8	12.3	5.00	0.019	IV	97.8(157.3)
DMG	33.9330	116.3830	12/04/1948	234317.0	0.0	6.50	0.043	VI	97.9(157.6)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.027	V	98.2(158.0)
GSP	33.9020	116.2840	07/24/1992	181436.2	9.0	5.00	0.019	IV	98.3(158.2)
GSG	32.4680	115.3340	02/12/2008	043237.9	13.0	5.00	0.019	IV	98.8(159.1)
PAS	32.6140	115.3180	10/15/1979	231653.4	12.3	6.60	0.045	VI	99.1(159.5)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.055	VI	99.2(159.7)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.019	IV	99.2(159.7)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.020	IV	99.2(159.7)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.019	IV	99.2(159.7)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.022	IV	99.2(159.7)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.020	IV	99.2(159.7)
GSG	32.4120	115.3330	11/20/2008	192303.3	3.0	5.30	0.022	IV	99.4(160.0)
GSG	32.4680	115.3170	02/19/2008	224130.7	10.0	5.10	0.020	IV	99.8(160.6)

****

-END OF SEARCH- 158 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2013

LENGTH OF SEARCH TIME: 214 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 9.2 MILES (14.9 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

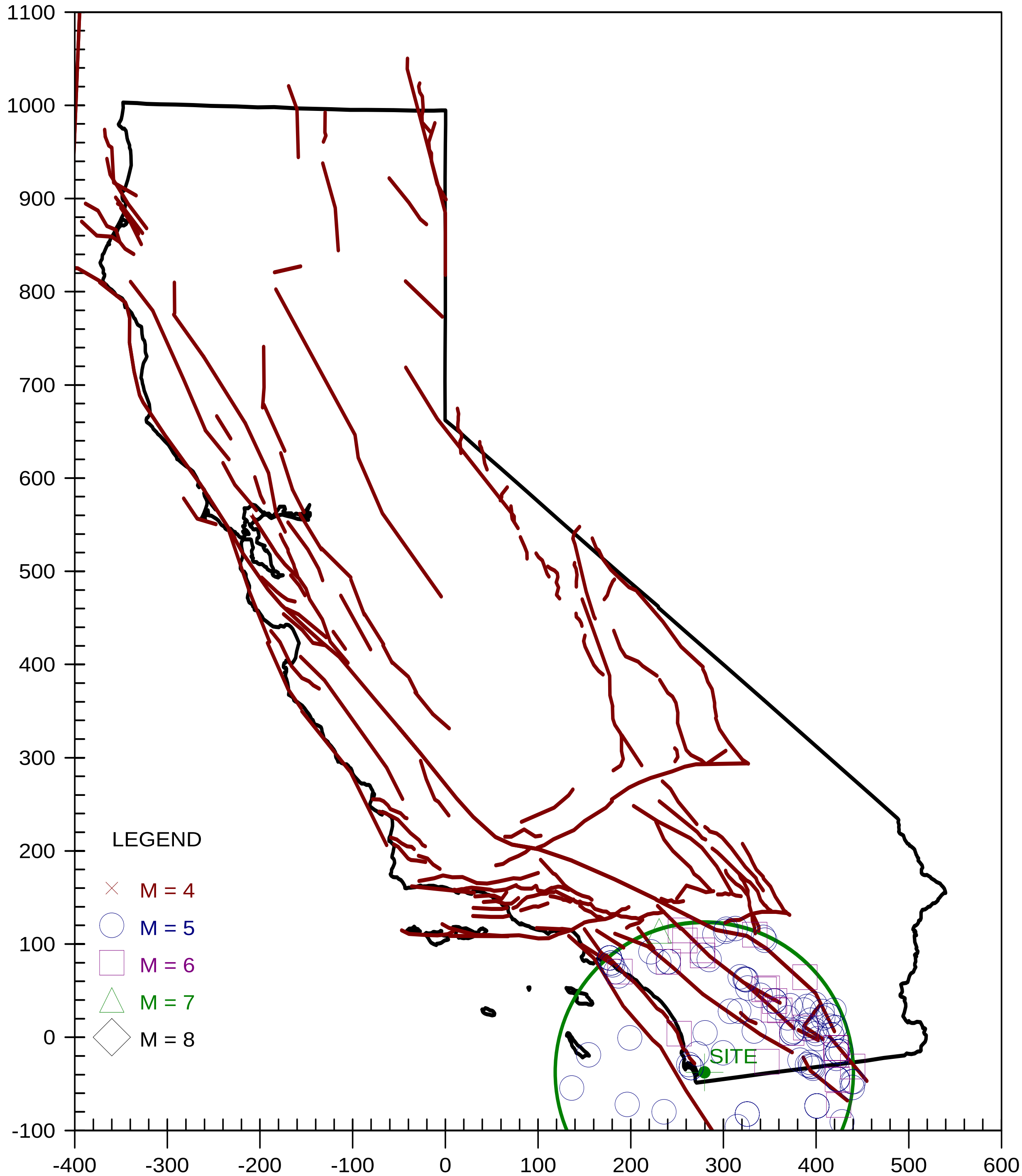
LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.157 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 1.535
b-value= 0.379
beta-value= 0.873

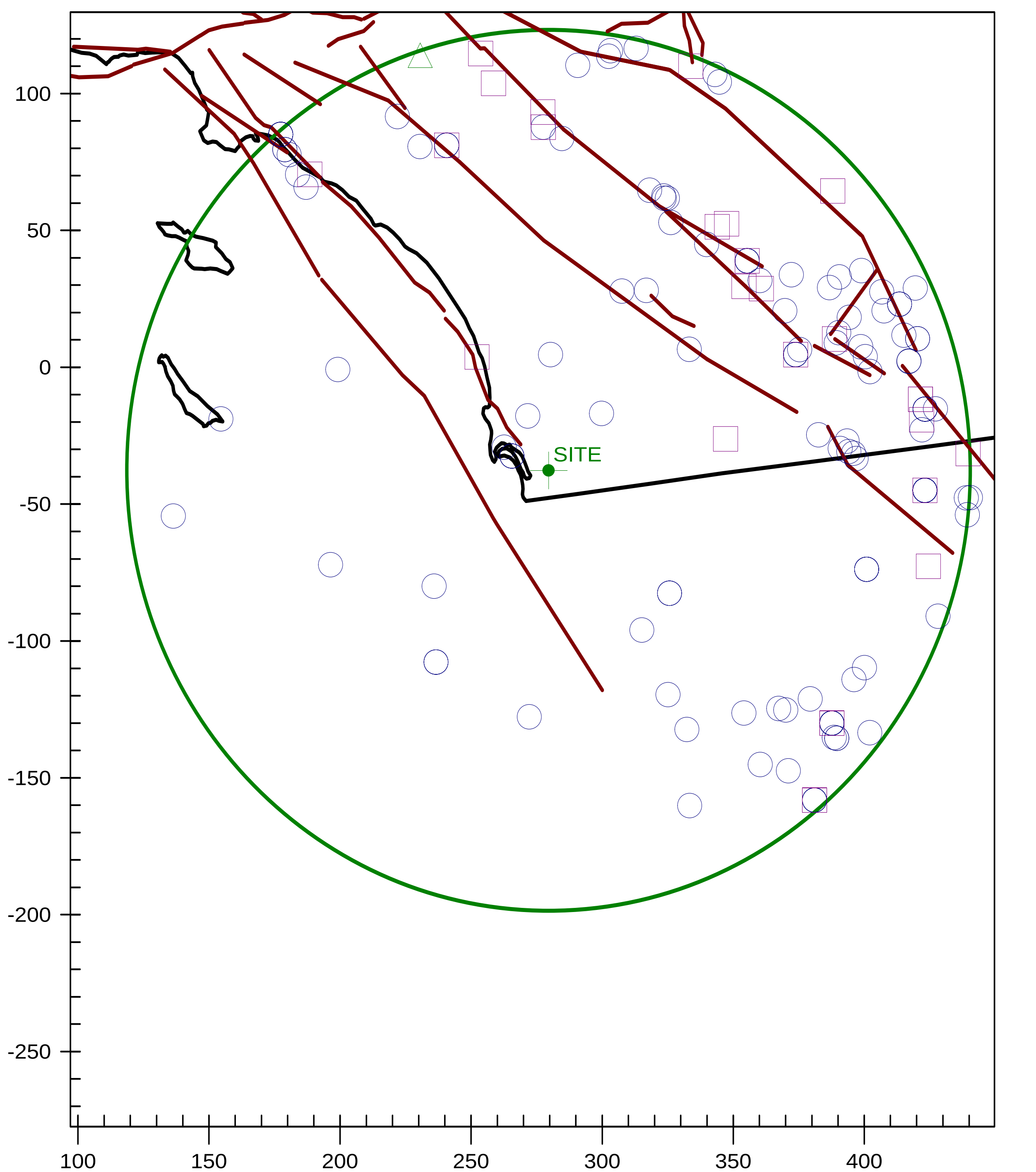
TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	158	0.74178
4.5	158	0.74178
5.0	158	0.74178
5.5	64	0.30047
6.0	28	0.13146
6.5	10	0.04695
7.0	1	0.00469

EARTHQUAKE EPICENTER MAP



EARTHQUAKE EPICENTER MAP

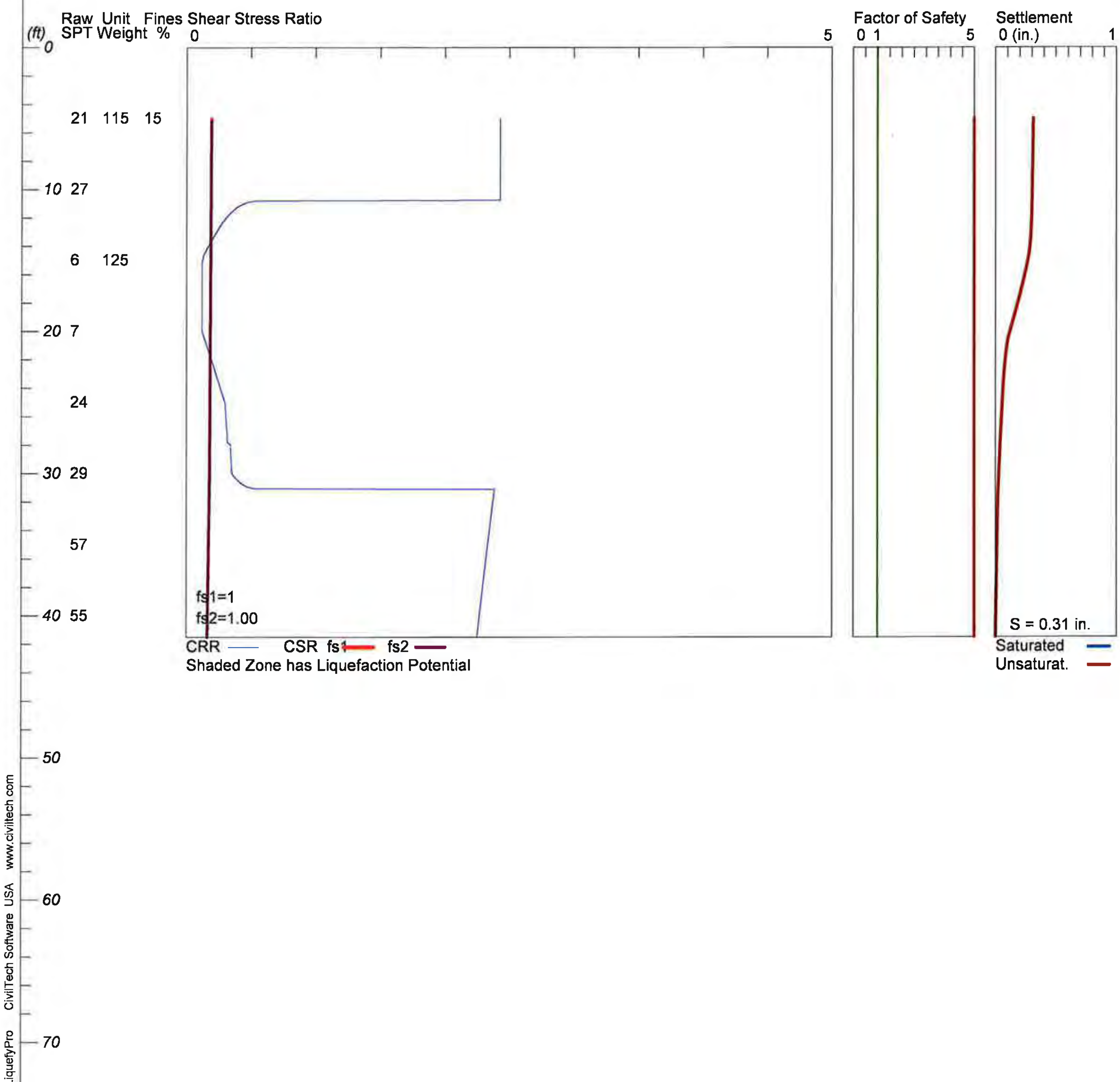


LIQUEFACTION ANALYSIS

Sharp CV Medical Center

Hole No.=B-8 Water Depth=150 ft Surface Elev.=435

Magnitude=6.95
Acceleration=.3g

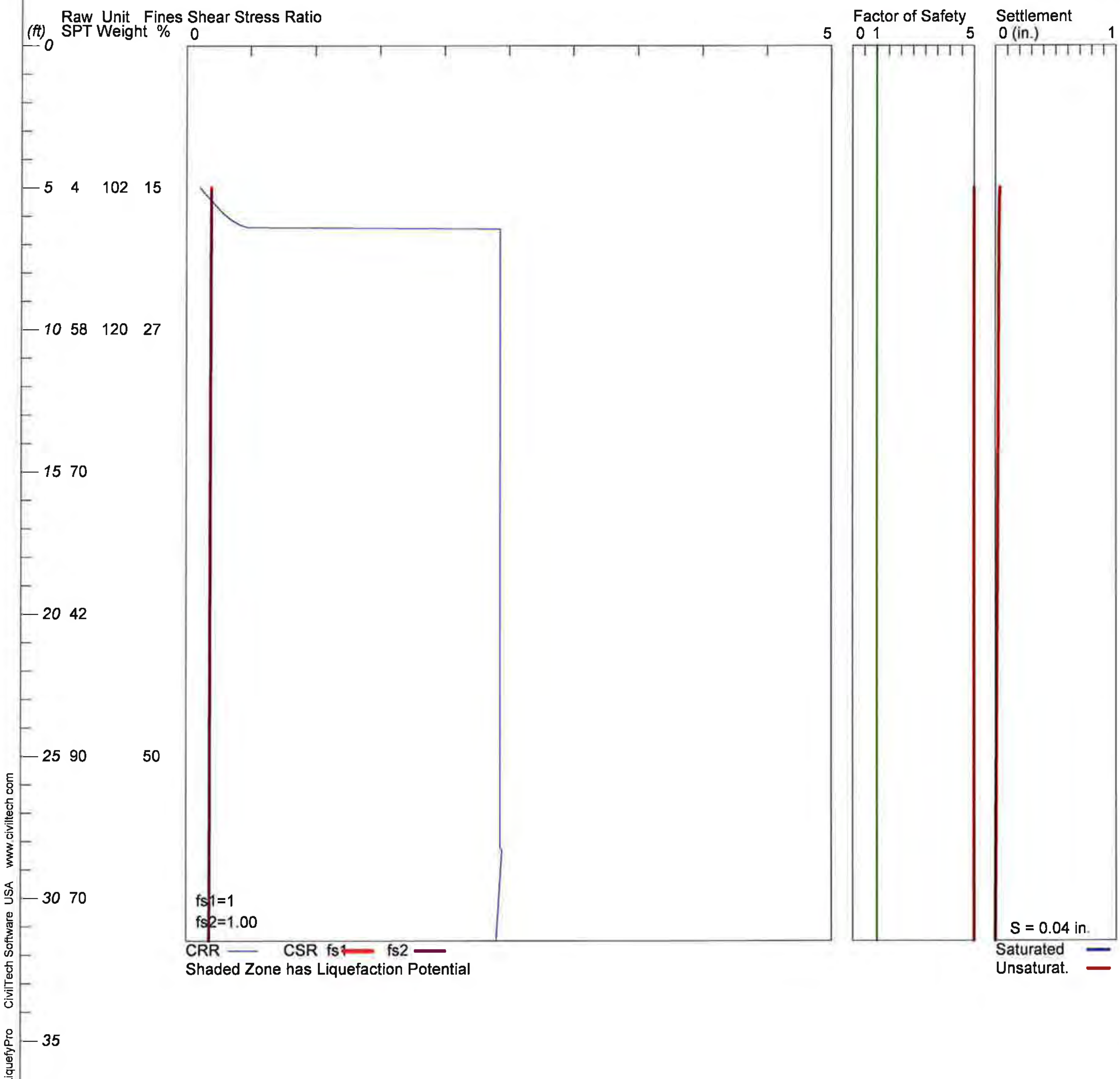


LIQUEFACTION ANALYSIS

Sharp CV Medical Center

Hole No.=B-10 Water Depth=150 ft Surface Elev.=439

Magnitude=6.95
Acceleration=.3g

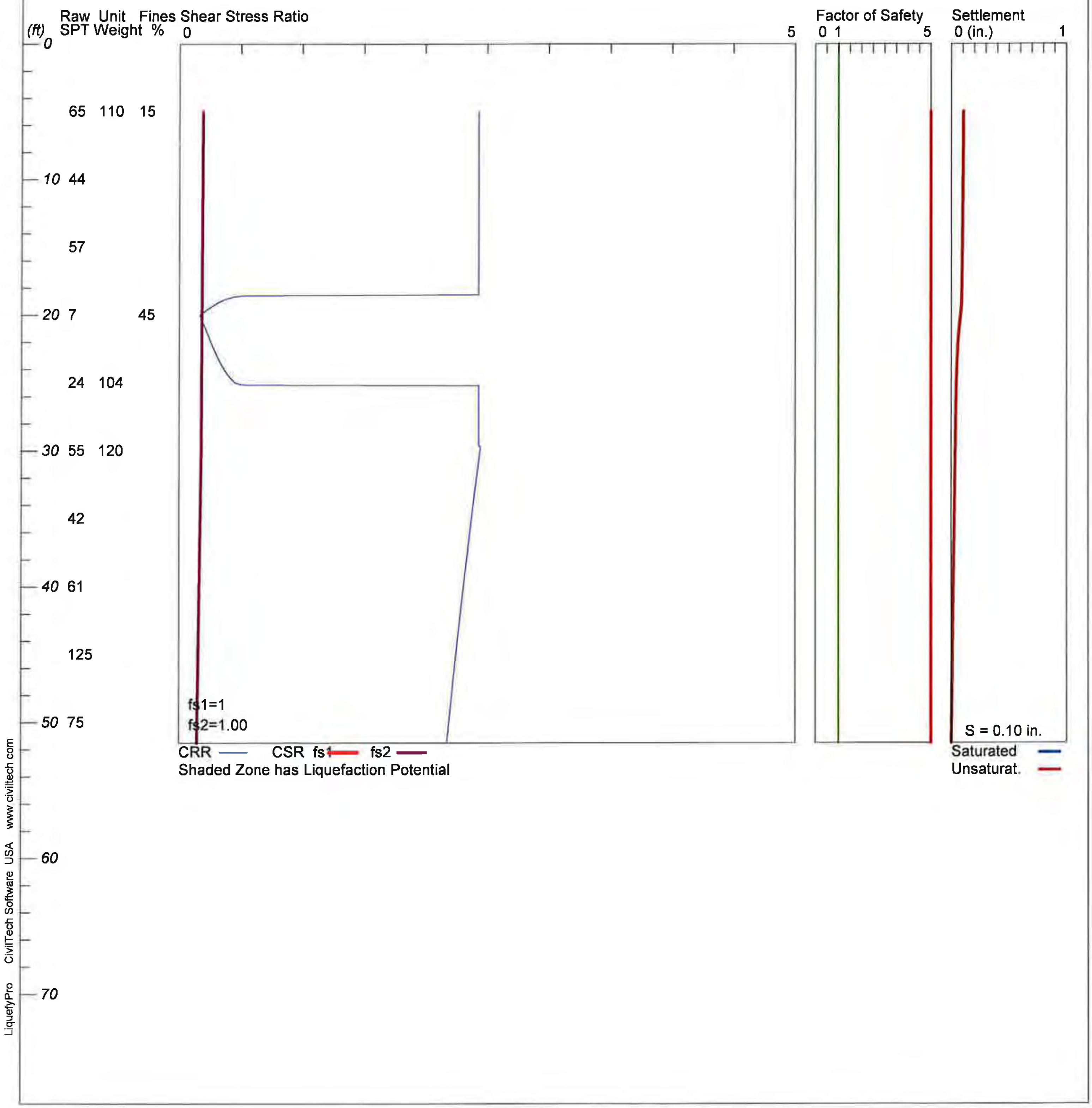


LIQUEFACTION ANALYSIS

Sharp CV Medical Center

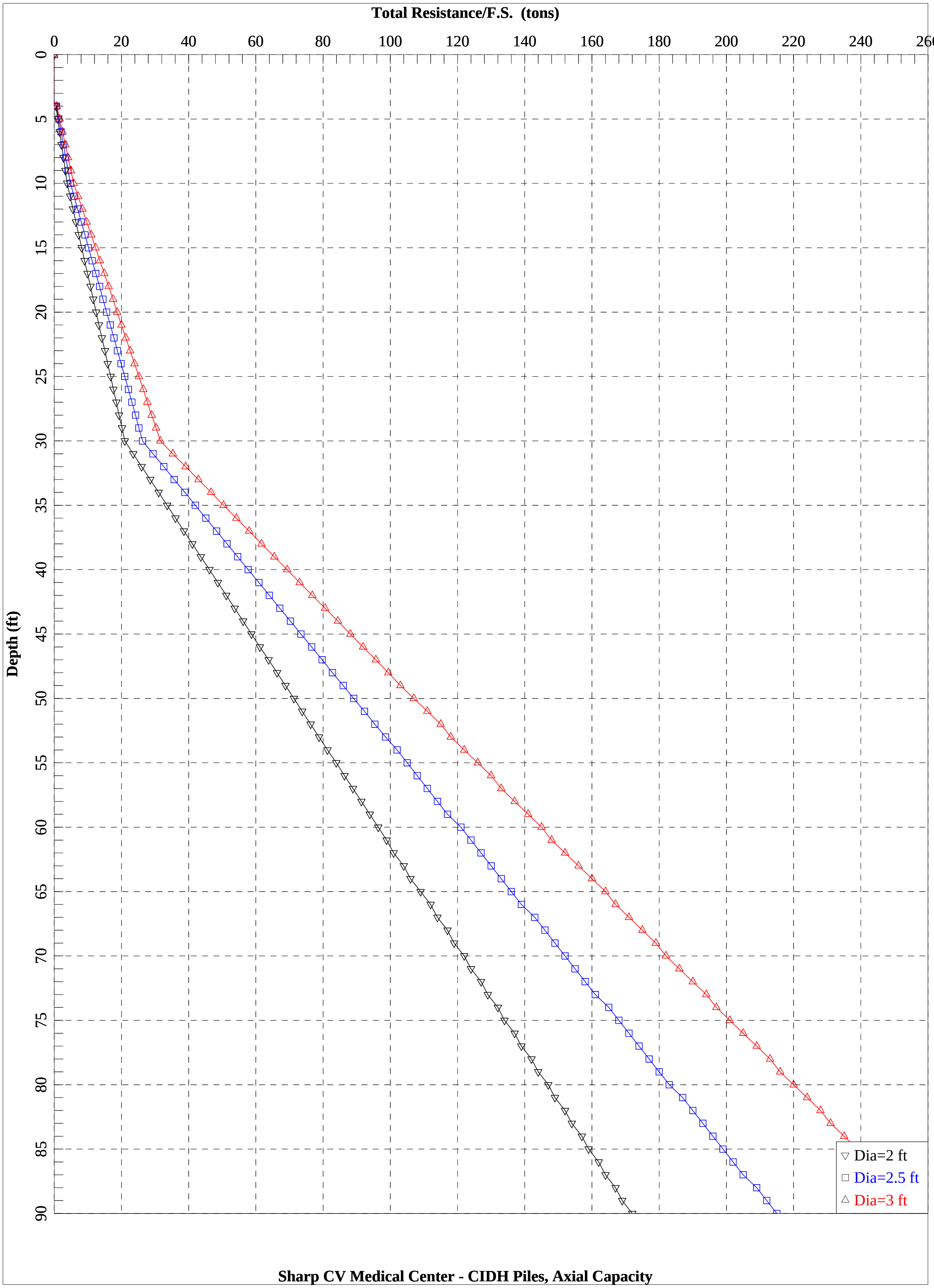
Hole No.=B-14   Water Depth=150 ft   Surface Elev.=435

Magnitude=6.95
Acceleration=.3g



Appendix F

Design Curves for CIDH Piles


Total Resistance/F.S. (tons)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
Dia=2 ft
Dia=2.5 ft
Dia=3 ft
Sharp CV Medical Center - CIDH Piles, Axial Capacity

Appendix G

General Earthwork and Grading Specifications

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

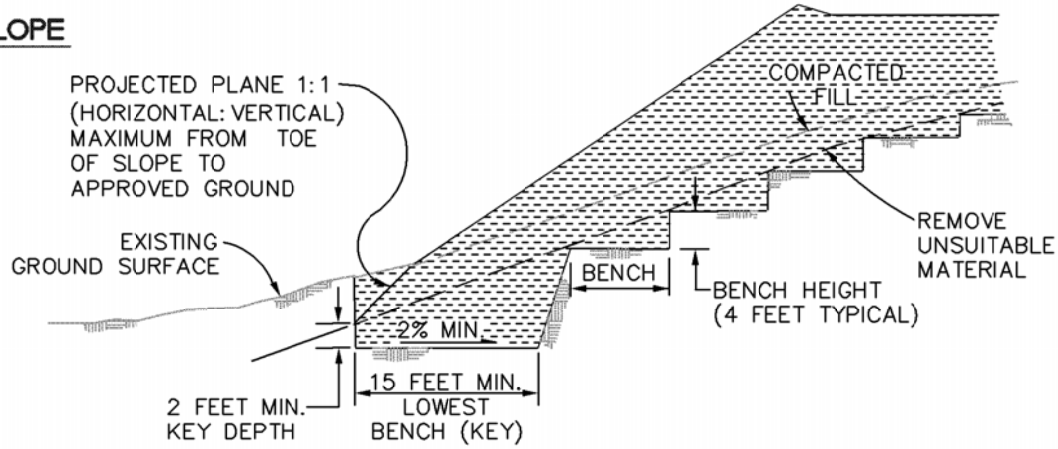
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

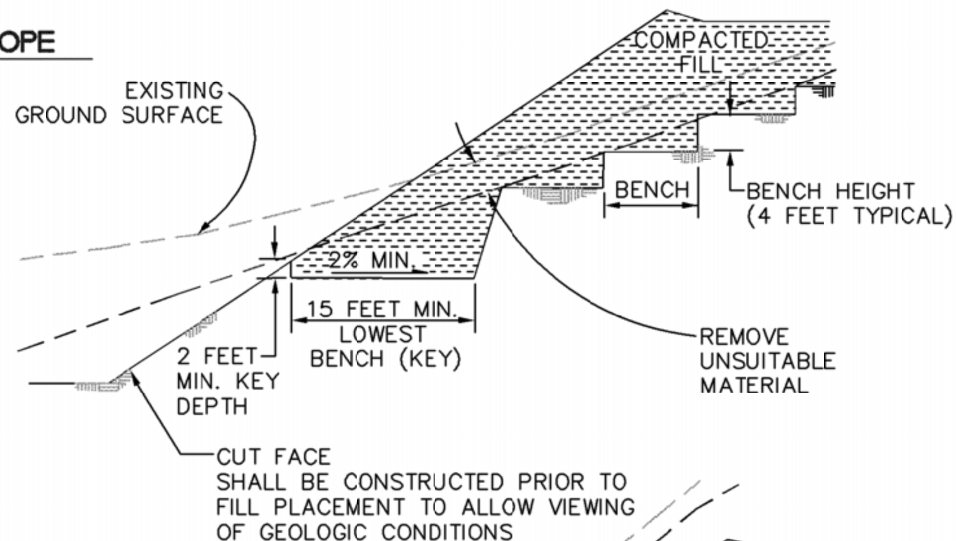
7.4 Observation and Testing

The densification of the bedding around the conduits shall be observed by the Geotechnical Consultant.

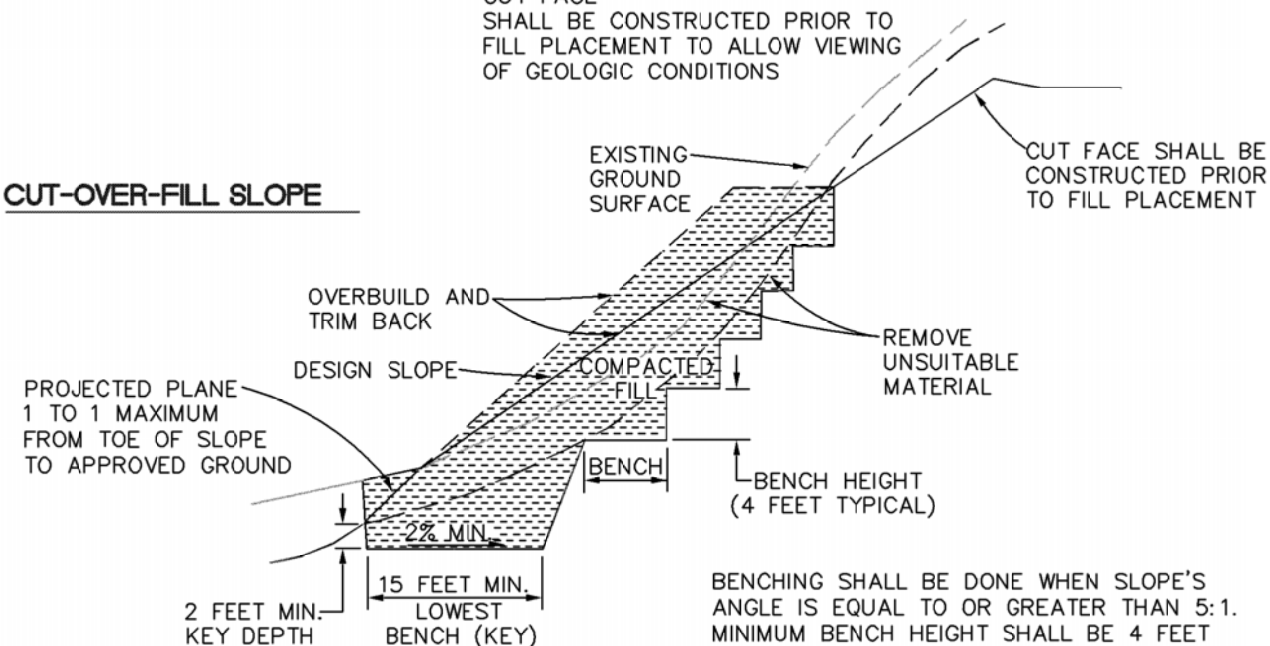
FILL SLOPE



FILL-OVER-CUT SLOPE



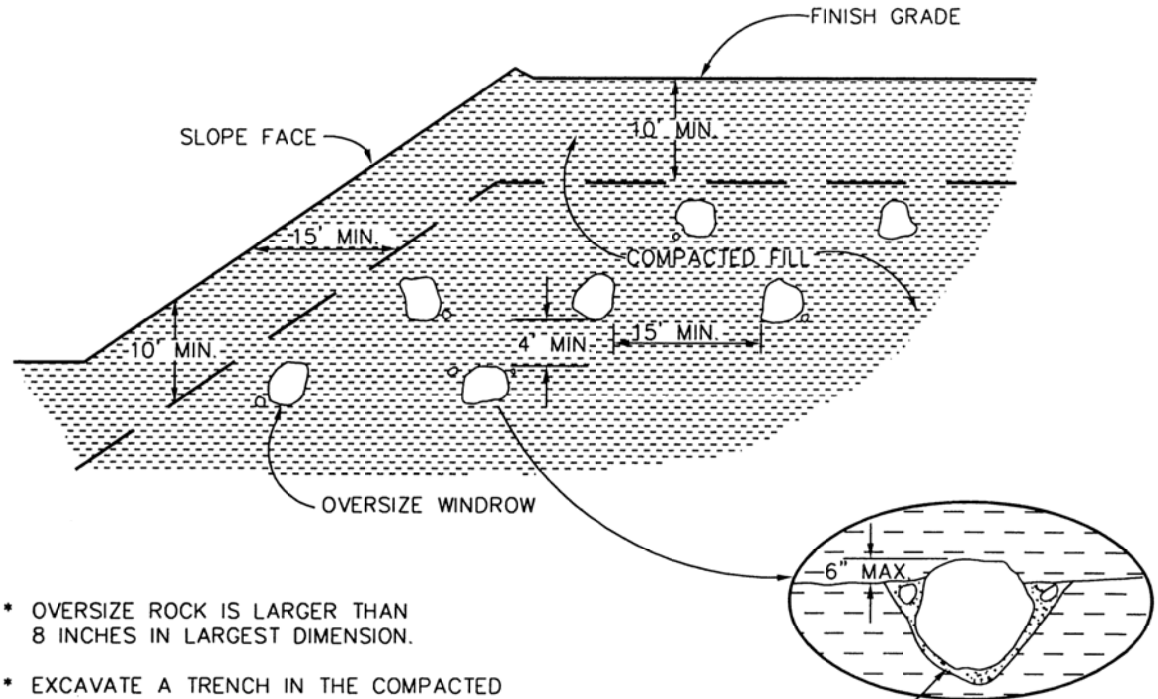
CUT-OVER-FILL SLOPE



BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

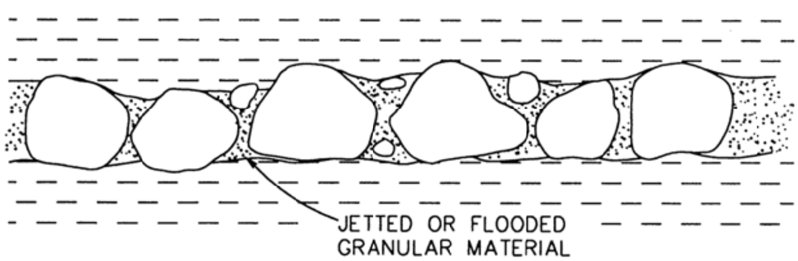
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A



GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

DETAIL

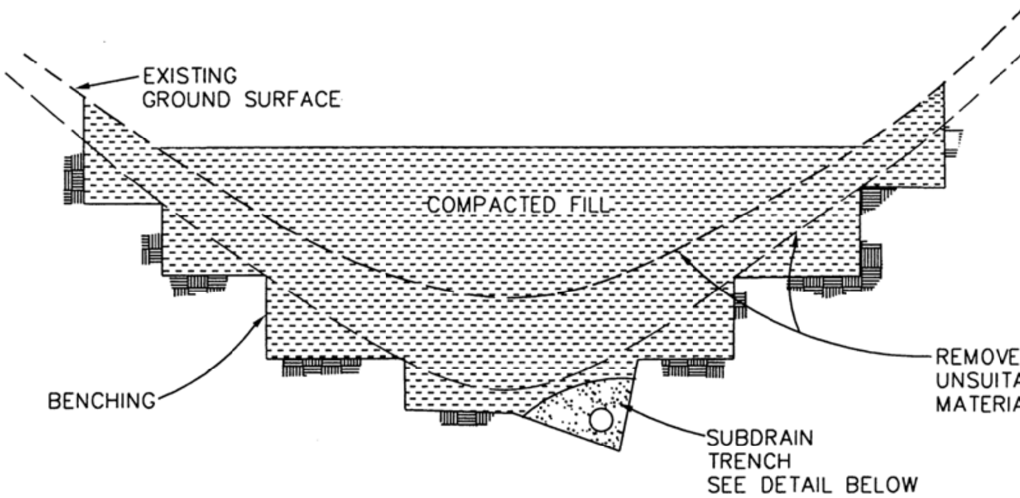
* OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
* EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
* BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
* DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
* WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

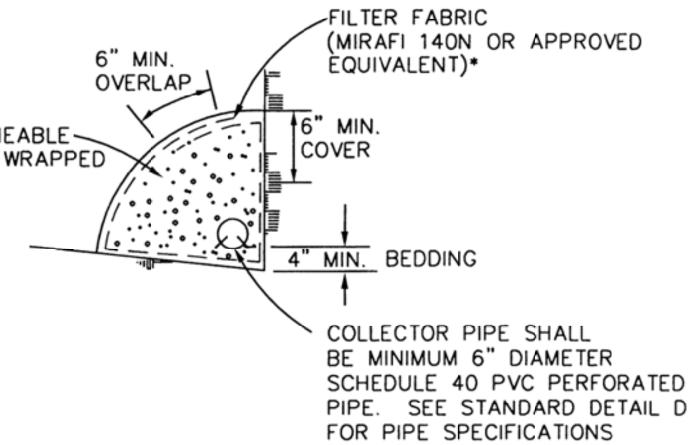


TYPICAL PROFILE ALONG WINDROW

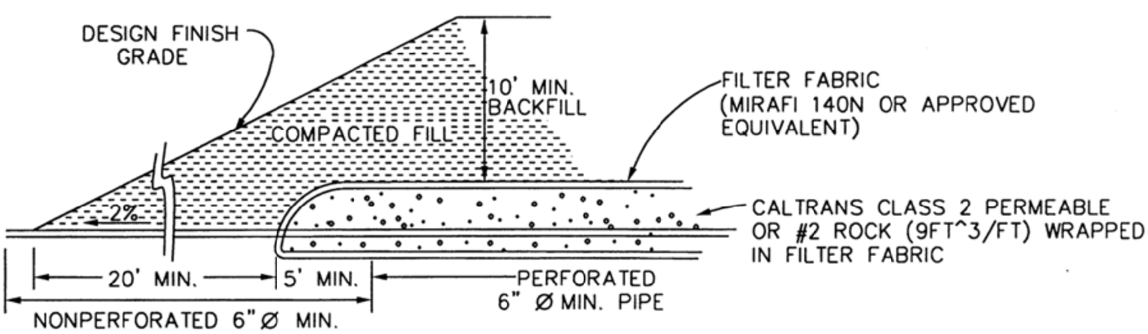
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL B



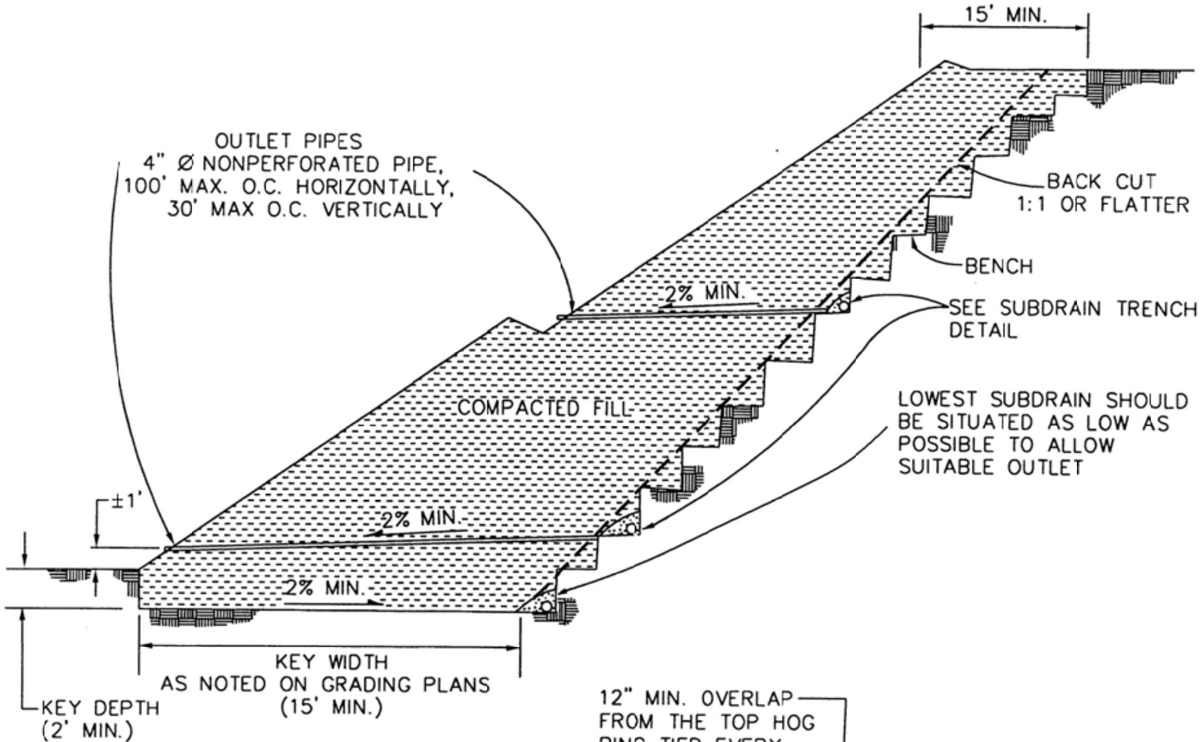


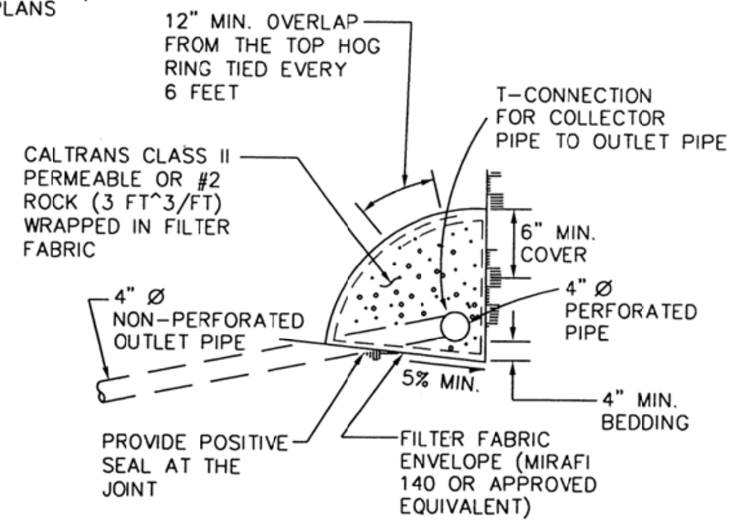
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

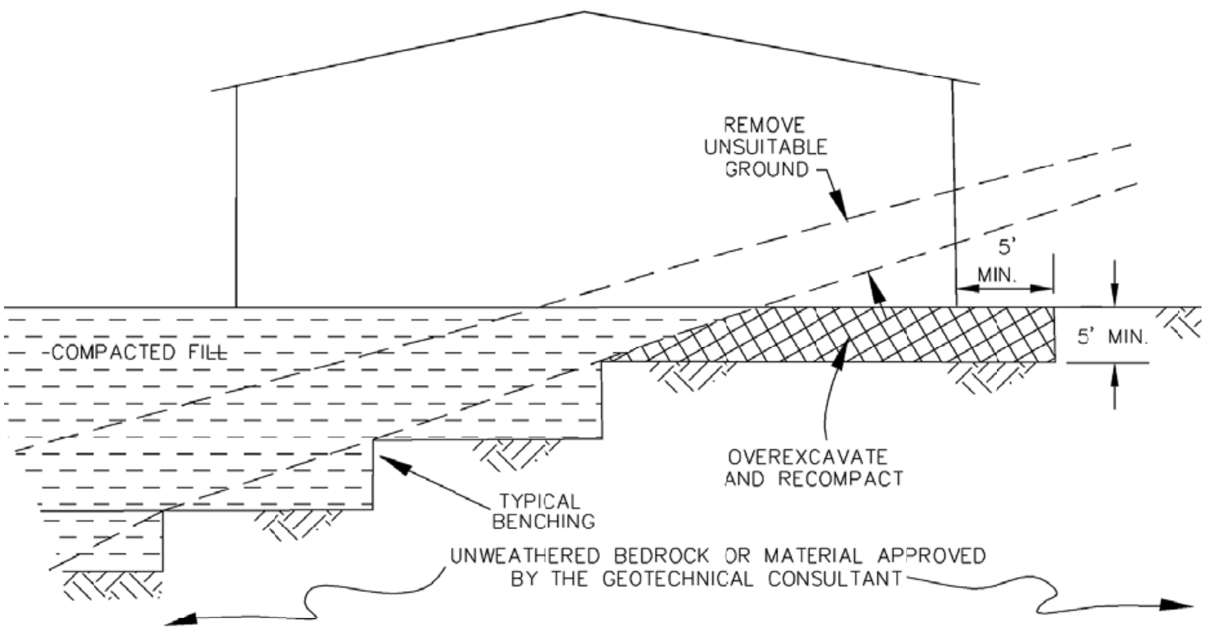
SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

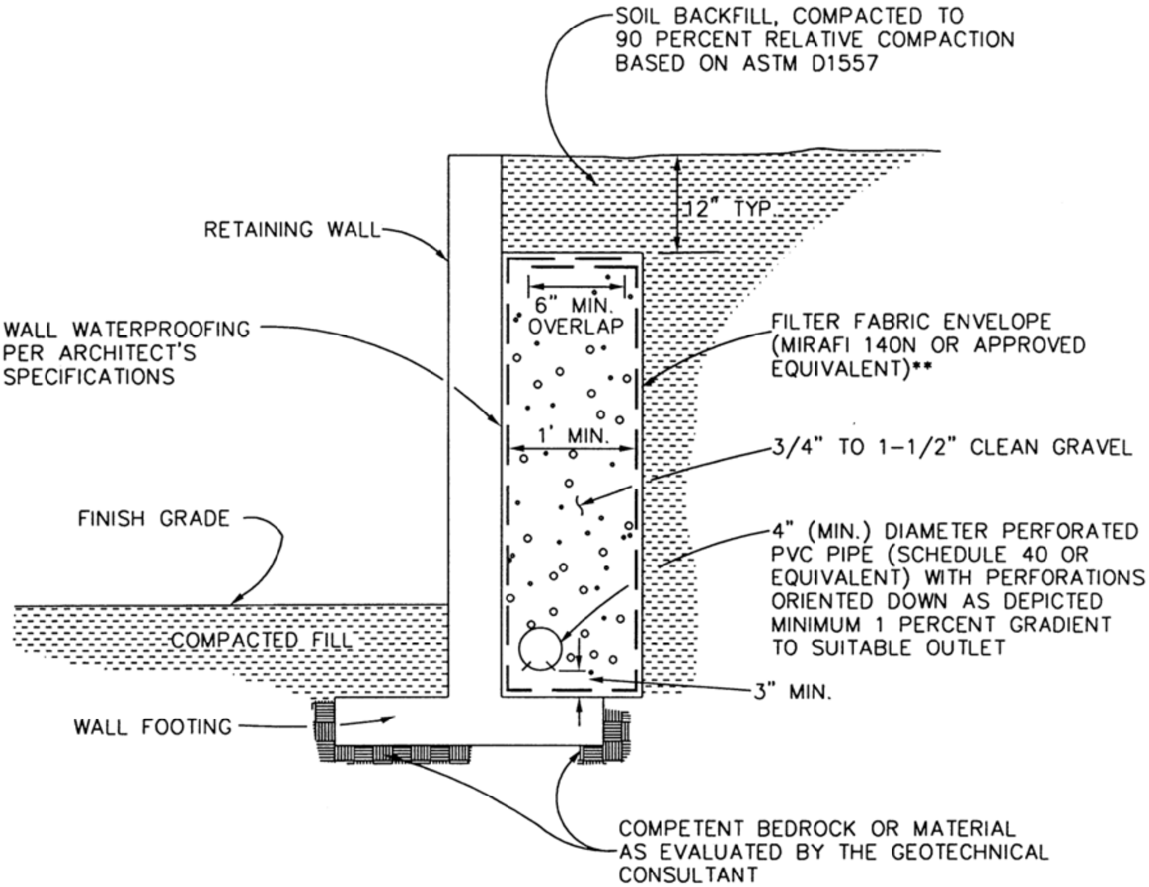
BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL D

CUT–FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

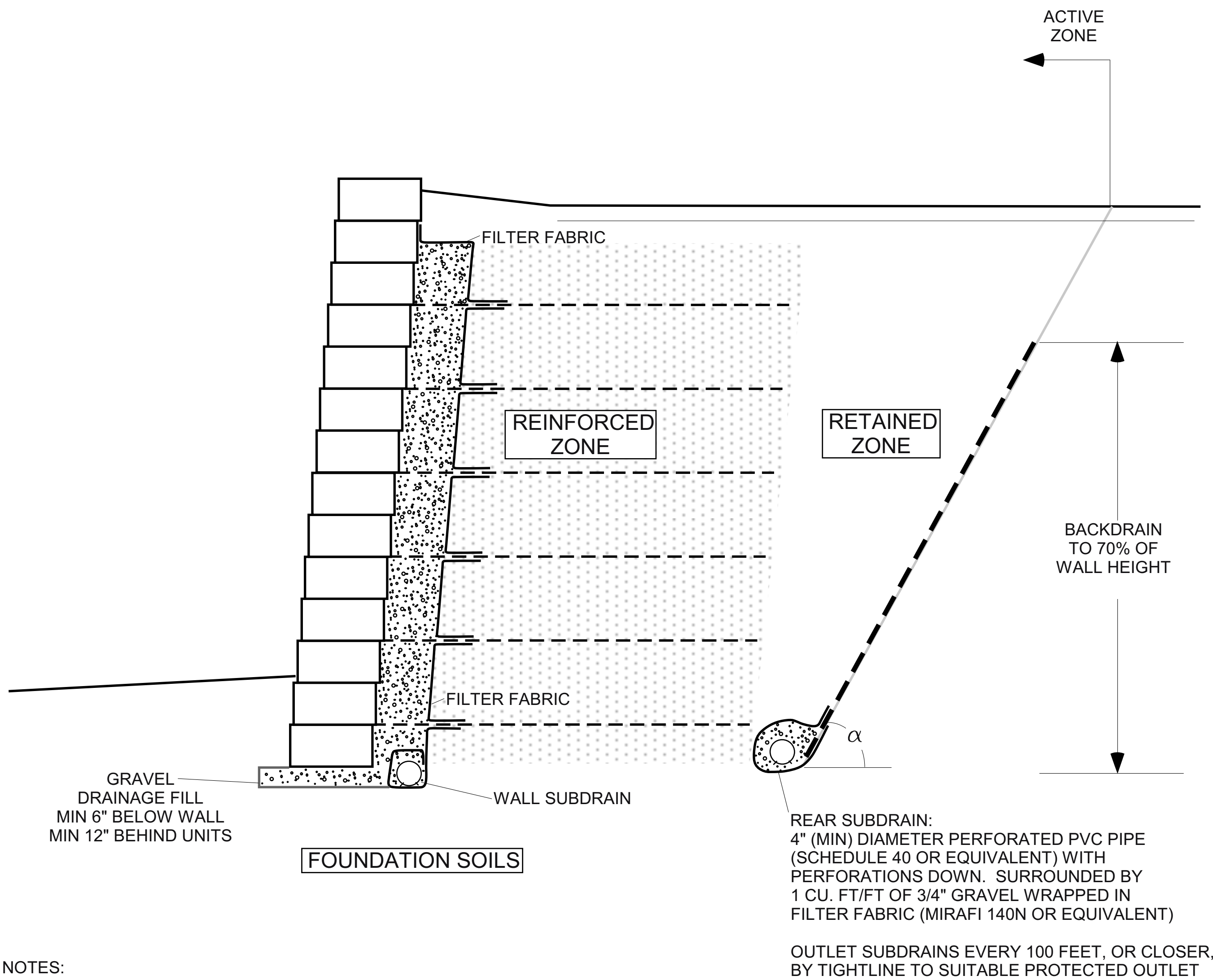
GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL E



NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL F



NOTES:

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20
FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45 + \phi/2$, WHERE ϕ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5) BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL G

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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