GEOTECHNICAL INVESTIGATION
MARY BIRCH HOSPITAL EXPANSION
SHARP METROPOLITAN MEDICAL CAMPUS
MASTER PLAN
7901 FROST STREET
SAN DIEGO, CALIFORNIA

Prepared for:

Sharp Healthcare
7901 Frost Street
San Diego, California 92123

Project No. 12764.001

Revised December 8, 2020
(October 2, 2020)

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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Project No. 12764.001

Sharp Healthcare
7901 Frost Street
San Diego, California 92123

Attention: Mr. Tim Crowe

Subject: Geotechnical Investigation
Mary Birch Hospital Expansion Project
Sharp Metropolitan Medical Campus Master Plan
7901 Frost Street
San Diego, California

In accordance with your request and authorization, Leighton Consulting, Inc. (Leighton) has conducted a geotechnical investigation for the proposed expansion of the Mary Birch Hospital at the Sharp Metropolitan Medical Campus located at 7901 Frost Street in San Diego, California. Our geotechnical study of the site was performed in general accordance with the Office of Statewide Health Planning & Development (OSHPD) requirements within the 2016 California Building Code.

Based on the results of our study, it is our professional opinion that the proposed expansion of the Mary Birch Hospital is feasible provided the recommendations provided herein are incorporated into the design and construction of the proposed improvements. The accompanying geotechnical report presents a summary of our current investigation and provides geotechnical conclusions and recommendations relative to the design and construction of the expansion of Mary Birch Hospital.

If you have any questions regarding our report, please do not hesitate to contact Robert Stroh at 858-300-4090. We appreciate this opportunity to be of service.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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Important Information about This

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

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1.0 INTRODUCTION

We recommend that all individuals utilizing this report read the preceding information sheet prepared by the Geoprofessional Business Association (GBA) and the Limitations, Section 7.0, located at the end of this report.

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for the proposed expansion of the Mary Birch Hospital within the Sharp Metropolitan Medical Campus located at 7901 Frost Street in San Diego, California (Figure 1). 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 structure. Our scope of services for this project included:

- Review of pertinent documents regarding the geotechnical conditions at the site.
- Markout of the exploration locations, notification and coordination of underground utility locators, and coordination with site personnel.
- Excavation of eight exploratory borings in the proximity of the proposed expansion.
- Review of previous geotechnical investigations for the current site area.
- Laboratory testing of selected soil samples. Laboratory testing consisted of unit weight, moisture content, direct shear, expansion index, 200 wash, modified Proctor, and corrosivity tests including - minimum electrical resistivity, pH, and water-soluble sulfate and chloride content tests.
- This study included a review of the subsurface exploration and laboratory testing programs previously conducted by others. The laboratory testing consisted of particle size analysis, Atterberg limits, direct shear, expansion index, and laboratory compaction test data.
- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations with respect to the proposed geotechnical design, site grading and general construction considerations.

1.2 Site Location and Description

The site currently consists of a paved parking lot and utilities. Both underground and above ground utilities are within the footprint of the proposed building addition. The paved parking is located to the north of the covered loading dock and east of the fire access lane. A tree/shrub/grass area is located east of the existing hospital building. Access to the site is provided by driveway entry named Mary Birch Lane along the east of Health Center Drive. In general, the site is bounded by the fire access lane to the east, the existing Mary Birch hospital building to the west, a covered loading dock to the south and Outpatient Pavilion and South Tower to the north. Site topography within the limits of the proposed project is generally flat lying and ranges in elevation from approximately 386 feet at the south-western portion of the site to 389 feet at the northern portion of the site (Figure 2). According to exhibits provided by the project civil engineer, some conduit manholes within the building pad and loading dock extend down to elevation 373 feet.

The latitude and longitude coordinates for the project are:

Latitude:	32.7982º N
Longitude:	117.1544º W

1.3 Proposed Development

The proposed expansion of the Mary Birch Hospital is planned to be constructed within an early phase as part of a much larger Sharp Metropolitan Medical Campus Master Plan redevelopment and retrofit undertaking. The Campus Master Plan is programed to be undertaken in a phased approach over several years.

Generally, the Mary Birch Expansion will be a six-story tall building with an overall footprint of approximately 21,000 SF. The floors and roof will be constructed out of concrete fill over metal deck, supported by steel beams and steel columns. The foundation system will consist of reinforced concrete continuous grade beams under the moment frames and spread footings under the gravity columns. Also proposed is a separate one-story loading dock with overhead canopy. The loading dock platform will be constructed on shallow spread footings. The foundations system of the canopy will consist of reinforced concrete pier footings. The approximate limits of the proposed expansion are depicted on Figure 2.

The finish floor elevation of the proposed addition is to be approximately 384 feet. The loading dock is to be at the same elevation and the loading dock ramp is approximately 4 feet lower and slopes up at 2 percent toward the loading dock driveway. Grades are expected to be lowered up to 10 feet to attain pad grade within the loading dock canopy footprint and 5 feet within the building pad.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Site Investigation

Our subsurface exploration was performed from July 29 to August 6, 2020, which consisted of excavating eight 8-inch diameter geotechnical borings (B-1 through B-7 and B-18) to depths of approximately 15 to 28 feet below the existing ground surface (bgs). Due to very limited site access and the presence of numerous site utilities, we have also utilized other investigations to supplement our data (Section 2.3). Borings B-1 through B-7 were drilled with a truck-mounted CME-95 drill rig and B-18 was drilled with a track-mounted limited-access drill rig. The purpose of our subsurface exploration was to evaluate the underlying stratigraphy, physical characteristics, and specific engineering properties of the soils within the area of the proposed improvements.

During the exploration operations, a geologist from our firm prepared geologic logs and collected bulk and relatively undisturbed samples for laboratory testing and evaluation. Disturbed standard penetration test (SPT) and relatively undisturbed split-barrel soil sampling using a 140-pound automatic-trip hammer free falling 30-inches were performed in accordance with ASTM International standards ASTM D 1586 and ASTM D 3550, respectively. After logging and field testing, the bore holes were backfilled with soil cuttings. Boreholes deeper than 20 feet were backfilled with bentonite in accordance with Department of Environmental Health (DEH) requirements. The boring logs are provided in Appendix B, laboratory test results are included in Appendix C, and the approximate geotechnical boring locations are depicted on Figure 2 (Geotechnical Map).

In addition to the geotechnical borings, a geophysical survey was performed on August 26, 2020 by Atlas Technical Consultants to measure shear wave velocity within the subsurface materials. The approximate location of the survey line is shown on Figure 2 and a copy of the survey report is included in Appendix B.

2.2 Laboratory Testing

Laboratory testing performed on representative soil samples obtained during our subsurface exploration included the following: direct shear, 200 wash, expansion index, laboratory compaction by modified Proctor, geochemical analysis for corrosion, moisture, and density. 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).

2.3 Previous Investigations

As part of our study, we have compiled geotechnical data that has been developed across the campus as part of previous design and construction projects. The studies that provided data within the vicinity of the expansion of Mary Birch Hospital include the following geotechnical reports:

- San Diego Geotechnical Consultants, 1988, Geotechnical Investigation, New Central Utility Plan, Medical Office Building and Women's Center, Sharp Hospital, San Diego, California, dated December 21.

- Shannon & Wilson, Inc., 2011a, Response to Comments by the California Geological Survey, Sharp Memorial Hospital – Central Tower, SPC-2 upgrade, 7901 Frost Street, San Diego, California, OSHPD Permit No. IL-090824-37, Facility No. 12364 dated March 30.

Boring logs from these previous studies that are in the vicinity of the Mary Birch Expansion have been included in Appendix B. Laboratory testing that was performed on samples from the previous borings are included in Appendix C. The locations of the previous explorations are presented on Figure 2.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Geologic Setting

The project area is situated in the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous-aged, Tertiary-aged, and Quaternary-aged sedimentary units. Most of the coastal region of the County of San Diego, including the site, occur within this coastal region and are underlain by sedimentary units. Specifically, the site is located within the coastal plain section of the Peninsular Range Geomorphic Province of California, which generally consists of subdued landforms underlain by sedimentary bedrock.

3.2 Site-Specific Geology

Based on our subsurface exploration, and review of pertinent geologic literature and maps (Appendix A), the geologic units underlying the site consist of undocumented artificial fill materials overlying Quaternary-aged Very Old Paralic Deposits, which in turn are underlain by the Mission Valley Formation and Stadium Conglomerate. A brief description of the geologic units encountered on the site is presented below. The approximate lateral and vertical distribution of these units are shown on the Geologic Cross-Sections A-A’ and B-B’ (Figure 3), and the approximate areal distribution is shown on Figure 2. The general distribution of the geologic formations in the site area is shown on Figure 4, the Geologic Map.

3.2.1 Undocumented Artificial Fill (Afu)

Based on our subsurface exploration, artificial fill soils were encountered in all current and previous geotechnical borings (B-1 through B-7 and B-18 (Current Borings), B-1 and B-5 (San Diego Geotechnical Consultants, 1988) and B-2 (Shannon & Wilson, 2011a)) with thickness varying between 0.4 feet to 13 feet. The thickness of fill soils within the footprint of the proposed building are anticipated to be less than 2 feet, except where existing utilities are present. As encountered during our subsurface exploration, the fill soils generally consisted of loose to very dense, reddish brown to dark reddish brown, dry to very moist, fine-grained, silty to clayey sands with trace gravel. Asphalt concrete over the aggregate base was encountered at the surface within broings B-5 to B-7. Asphalt concrete was also encountered in borings B-1 to B-4, but without underlying aggregate base. It should be noted that the existing pavement and aggregate base section at these borings ranges from approximately 4 inches to 17 inches in thickness.

Undocumented fills are also anticipated to be encountered where buried utilities or below grade structures are present beneath the site.

3.2.2 Quaternary-aged Very Old Paralic Deposits (Qvop$_8$)

Underlying the existing undocumented artificial fill soils, the Quaternary-aged Very Old Paralic Deposits was encountered in all of our geotechnical borings. During our drilling exploration, this material generally consisted of medium dense to very dense, yellowish red to dark reddish-brown, moist, silty or clayey sandstone with variable amounts of gravel and very dense, light yellowish brown to reddish brown, moist, silty gravel with fine sand and trace cobble. A gravel-cobble conglomerate was encountered at depth within the Very Old Paralic Deposits during drilling. The cobble located throughout this unit is 6 to 8 inches in diameter with isolated cobbles up to 1 foot in diameter. Note that this unit was formerly named Lindavista Formation as shown in the previous boring logs by others. Previous investigations classified the material as very dense, light gray brown to reddish brown, damp to moist, silty or clayey sandstone.

3.2.3 Mission Valley Formation (Tmv) and Stadium Conglomerate (Tst)

Although only encountered within one of our boring explorations (B-4), the underlying Mission Valley Formation and Stadium Conglomerate likely occur occur below the cobble-gravel conglomerate that caused drilling refusal on all of the borings. These materials are anticipated to consist of very dense, coarse-grained, light brown to reddish brown, silty cobble-gravel conglomerate with sand. It should be noted that several previous studies (Appendix A) have identified the underlying conglomerate as Stadium Conglomerate. However, based on our interpretation of the geology shown on Figure 4, we believe that the conglomerate is that of the Mission Valley Formation.

3.3 Geologic Structure

The site is located within Zone 52 of the City of San Diego Seismic Safety Study Map (Figure 7) and is classified as “gently sloping to steep terrain, favorable geologic structure, low risk.” Based on previously completed geotechnical report (Appendix A) and our recent subsurface exploration, along with previous work completed at nearby sites, the project site is underlain by generally massive (favorably oriented) geologic structure consisting sandy and clayey gravel-cobble conglomerate of the Mission Valley Formation and the Stadium Conglomerate.

3.4 Landslides

Several formations within the San Diego region are particularly prone to landsliding (Friars Formation). 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 identified at the site during our field exploration or our review of available geologic literature, topographic maps, and stereoscopic aerial photographs. Furthermore, as discussed in Section 3.3 the site is underlain by generally massive, favorable oriented geologic structure. Therefore, the potential for significant landslides or large-scale slope instability at the site is considered low.

3.5 Surface and Groundwater

No indication of surface water or evidence of surface ponding was encountered during our geotechnical investigation performed at the site. However, surface water may drain as sheet flow across the site during rainy periods.

Groundwater was not encountered during our subsurface exploration at the site. It should be noted that groundwater levels may fluctuate with seasonal variations and irrigation and local perched groundwater conditions may exist at the contact between the undocumented artificial fill and the Very Old Paralic Deposits. Beyond nuisance seepage into open holes, we do not anticipate groundwater will be a constraint to the development of the site.

3.6 Engineering Characteristics of On-site Soils

Based on the results of our laboratory testing of representative on-site soils, and our professional experience on similar sites with similar soils conditions, the engineering characteristics of the on-site soils are discussed below.

3.6.1 Compressible Soils

The site is underlain by undocumented artificial fill materials. No records for compaction testing were available at the time of our exploration. Therefore, generally, the upper 1 to 2 feet of undocumented artificial fill is considered compressible in their current state. Recommendations for remedial grading of these soils are provided in the following sections of this report.

3.6.2 Expansion Potential

Expansion index testing on one representative soil sample indicated that the onsite soils generally have a very low potential (EI < 20) for expansion (Appendix C). However, higher expansive soils may be encountered during the grading of the site and during foundation excavation. Expansive soils are not anticipated to significantly impact the proposed site improvements.

3.6.3 Hydrocollapse

Based on the results of our observations during our field investigation, undocumented fill is underlain by dense to moderately indurated Very Old Paralic Deposits and Tertiary-aged Formations. Therefore, the potential for hydro-collapse of the underlying earth materials is considered low at the site.

3.6.4 Soil Corrosivity

A preliminary screening of the on-site soils was performed to evaluate their potential corrosive effect on concrete and ferrous metals. In summary, laboratory testing on representative soil samples obtained during our subsurface exploration evaluated pH, minimum electrical resistivity, and chloride and soluble sulfate content. The samples tested had pH values ranging from 6.9 to 8.1, and a measured minimum electrical resistivity of 1400 ohm-cm, respectively. Test results also indicated that the samples had maximum chloride content of 120 parts per million (ppm), and maximum soluble sulfate content of 165 ppm.

3.6.5 Excavation Characteristics

It is anticipated that the Very Old Paralic Deposits can be excavated with conventional heavy-duty construction equipment. If oversize material (larger than 6 inches in maximum dimensions) is generated, it should be placed in non-structural areas or hauled off site. Also, difficult excavation conditions may be encountered with deeper excavations (elevator pits, utilities, deepened piles, etc.) founded in concretionary and cemented layers below where the Very Old Paralic Deposits transitioned into cobble conglomerate material. It should be noted that drilling refusal was encountered with the Limited Access Drill Rig in Boring B-18 and with a more powerful CME 95 Drill Rig in Borings B-1 through B-7 on the cobble conglomerate. These materials likely will require heavy ripping or breaking with specialized equipment during excavation.

3.7 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 8). In addition, based on our review of topographic maps and aerial photographs, the site is not located downstream of a dam (Figure 9).

3.8 Infiltration

Based on the results of previous geotechnical investigations and our current investigation, the site is anticipated to be a “No Infiltration Site” based on City of San Diego Storm Water Standards (2018).

3.9 Exceptional Geologic Conditions

Exceptional geologic conditions are potential hazards 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.

3.9.1 Hazardous Materials

The site has been developed as a hospital site since the 1950's. We understand emergency fuel is stored within underground storage tanks near the central utility plant. We are not aware of any unauthorized releases into the subsurface within the hospital campus. The presence of methane gas, hydrogen-sulfide gas, tar seeps, and other naturally occurring hazardous materials has not been previously observed or mapped. Therefore, it is our opinion that the probability of such materials existing at the Mary Birch Hospital expansion site is very low.

3.9.2 Regional Subsidence

The site area is not currently utilized for groundwater or oil withdraws. In addition, the dense nature of the Mission Valley Formation and Stadium Conglomerate is not prone to subsidence settlement due to withdraw of fluids. Therefore, regional subsidence potential is considered nil.

3.9.3 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.

3.9.4 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, approximately 70 miles east of the site.

3.9.5 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.

3.9.6 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 has potentially containing Radon-222 gas (Churchill, 2003). 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 SEISMICITY

4.1 Regional Tectonic Setting

The site is located within the Peninsular Ranges Geomorphic Province, which is traversed by several major active faults. The Whittier-Elsinore, San Jacinto, and the San Andreas faults are major active fault systems located east of the site, and the Rose Canyon, Newport-Inglewood (offshore), and Coronado Bank are active faults located west to southwest of the site (Jennings, 2010), see Figure 5. The primary seismic risk to the site area is the Rose Canyon fault zone located approximately 3.0 miles west of the site (USGS, 2008).

The Rose Canyon fault zone consists predominantly of right-lateral strike-slip faults that extend south-southeast bisecting the San Diego metropolitan area (Figure 6). 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 and 2000; 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, 2003) as being Earthquake Fault Zones.

4.2 Local Faulting

The California Geologic Survey (CGS, 2013) defines a Holocene-active fault as a fault which has “had surface displacement within Holocene time (about the last 11,700 years).” Our review of available geologic literature (Appendix A) indicates that there are no known pre-Holocene or Holocene-active faults transecting the site. The subject site is also not located within any State mapped Earthquake Fault Zones or City of San Diego mapped fault zones. The nearest active fault is the Rose Canyon fault located approximately 3 miles west of the site (USGS, 2008).

4.3 Seismicity

The site is considered to lie within a seismically active region, as is all of Southern California. As previously mentioned above, the Rose Canyon fault zone located approximately 3 miles west of the site is considered the ‘active’ fault having the most significant effect at the site from a design standpoint.

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 (>M5.0) is shown on the Historical Seismicity Map in Appendix D. The historic seismicity for the site has been tabulated utilizing the computer software EQSEARCH (Blake, 2018). The results are presented in Appendix D. The results indicate that the maximum historical site acceleration from 1800 to present has been estimated to be 0.137g.

4.4 Seismic Hazards

Severe ground shaking is most likely to occur during an earthquake on one of the regional active faults in Southern California. The effect of seismic shaking may be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California.

4.4.1 Shallow Ground Rupture

No pre-Holocene or Holocene-active faults are mapped transecting or projecting toward the site. Due to the absence of faults at the site, surface rupture from faulting is considered low. In addition, due to the lack of nearby slopes, ground cracking due to shaking from a seismic event is also considered low.

4.4.2 Mapped Fault Zones

The site is not located within a State mapped Earthquake Fault Zone (EFZ), nor is it located within a City of San Diego fault zone. As previously discussed, the subject site is not underlain by known faults.

4.4.3 Site Class

Utilizing 2016 California Building Code (CBC) procedures, we have characterized the site soil profile to be a Site Class C based on our subsurface explorations using SPT blow counts, experience with similar sites in the project area, previously completed geotechnical studies on the Campus (Appendix A), and the completion of a geophysical survey (Appendix B).

4.4.4 Building Code 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 1 are the spectral acceleration parameters for the project determined in accordance with the 2016 CBC (CBSC, 2016) and the SEA/OSHPD Web Application.

Table 1

2016 CBC Mapped Spectral Acceleration Parameters

Site Class	C
Site Coefficients	F_a = 1.000 F_v = 1.387
Mapped MCE Spectral Accelerations	S_S = 1.080g S_1 = 0.413g
Site Modified MCE Spectral Accelerations	S_{MS} = 1.080g S_{M1} = 0.573g
Design Spectral Accelerations	S_{DS} = 0.720g S_{D1} = 0.382g

Utilizing ASCE Standard 7-10, in accordance with Sections 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.461g for the site. For a Site Class C, the F_{PGA} is 1.0 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.461g for the site.

4.5 Secondary Seismic Hazards

In general, secondary seismic hazards can include soil liquefaction, seismically-induced settlement, lateral displacement, surface manifestations of liquefaction, landsliding, seiches, and tsunamis. The potential for secondary seismic hazards at the subject site is discussed below.

4.5.1 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Granular soils tend to densify when subjected to shear strains induced by ground shaking during earthquakes. Research and historical data indicate that loose granular soils underlain by a near surface groundwater table are most susceptible to liquefaction, while the most clayey materials are not susceptible to liquefaction. Liquefaction is characterized by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested at the ground surface by settlement and, possibly, sand boils where insufficient confining overburden is present over liquefied layers. Where sloping ground conditions are present, liquefaction-induced instability can result.

The site is underlain at depth by Quaternary-aged Very Old Paralic Deposits in turn underlain by the Mission Valley Formation and Stadium Conglomerate (Figure 4). Based on the underlying dense character of the Very Old Paralic Deposits, the presence of moderately indurated Tertiary-aged materials below those, and the lack of a shallow groundwater table, it is our opinion that the potential for liquefaction and seismic related settlement across the site is low.

4.5.2 Lateral Spread

Empirical relationships have been derived (Youd et al., 1999) to estimate the magnitude of lateral spread due to liquefaction. These relationships include parameters such as earthquake magnitude, distance of the earthquake from the site, slope height and angle, the thickness of liquefiable soil, and gradation characteristics of the soil.

The susceptibility to earthquake-induced lateral spread is considered to be low for the site because of the lack of susceptibility to liquefaction and a lack of open descending slope faces in the site vicinity.

4.5.3 Tsunamis and Seiches

Based upon the California Emergency Management Agency Tsunami Inundation Map (CalEMA, 2009), the site is not located within a tsunami inundation area. In addition, based on the generally strike-slip character of off-shore faulting and proposed elevation of the site with respect to sea level, the possibility of seiches and/or tsunamis is considered to be nil.

4.6 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 (Figure 7).

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. Furthermore, our field reconnaissance and the local geologic maps indicate the site is generally underlain by generally flat topography and favorable oriented geologic structure, consisting of massively bedded sandstone. Therefore, the potential for significant landslides or large-scale slope instability at the site is considered nil.

4.7 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the site is not located within a floodplain. Based on our review of topographic maps, the site is not located downstream of a dam or within a dam inundation area (Figures 8 and 9). Based on this review and our site reconnaissance, the potential for flooding of the site is considered low.

5.0 CONCLUSIONS

Based on the results of our geotechnical investigation of the site, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications.

- As the site is located in the seismically active southern California area, all structures should be designed to tolerate the dynamic loading resulting from seismic ground motions;
- The site is not transected by pre-Holocene or Holocene-active faults;
- The existing undocumented artificial fill materials are considered potentially compressible and generally unsuitable in their present state to support additional fill or structural loads;
- Based on laboratory testing and site mapping, the site materials possess a very low to low expansion potential. It is possible that higher expansion materials may be encountered in locations not explored;
- The existing onsite soils are generally suitable for use as engineered fill, provided they are free of organic material, debris, and rock fragments larger than 8 inches in maximum dimension;
- If import soils are planned, the soils should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D 4829) and have a low corrosion impact to the proposed improvements;
- Based on the results of our subsurface exploration, we anticipate that the on-site materials should be generally excavatable with conventional heavy-duty earthwork equipment. However, deeper excavations (drilled piles, elevator pits, utilities, etc.) may encounter concretionary and cemented conglomerate layers within the Very Old Paralic Deposits and underlying formation that may require heavy ripping or breaking with specialized equipment during excavation;
- Groundwater was not encountered during our investigation, nor is groundwater anticipated to be encountered during site excavation and construction except as possible seepage during/after episodes of precipitation or in areas of irrigation;

- Based on the results of our geotechnical evaluation, it is our opinion that the proposed expansion of Mary Birch Hospital can be supported with conventional foundations and the loading dock canopy on drilled piles;
- Although Leighton does not practice corrosion engineering, laboratory test results indicate the soils present on the site have a low potential for sulfate attack on normal concrete. However, the onsite soils are considered to have a corrosive potential for corrosion to buried uncoated ferrous metal. A corrosion consultant may be consulted to provide additional recommendations.

6.0 RECOMMENDATIONS

The following recommendations have been developed based on support of the structure by shallow foundations that bear on competent Very Old Paralic Deposits.

6.1 Earthwork

We anticipate that earthwork at the site will consist of minor cuts and fills to cuts extending to 10 feet in depth to attain subgrade elevations within the building pad and loading dock area. 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 E. In case of conflict, the following recommendations supersede those in Appendix E.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill or engineered structures should be cleared of surface and subsurface obstructions, including any existing debris and undocumented or loose fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off-site. Where trees are present, the entire root ball should be removed. It is anticipated that existing utilities will be removed from the building pads. Areas disturbed by demolition activities should be restored to grade with properly compacted fill. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to above-optimum moisture conditions, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D 1557).

6.1.2 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. However, concretionary and cemented layers with oversize rock within the Very Old Paralic Deposits and underlying formation may require heavy ripping or breaking with specialized equipment during excavation if encountered. Excavation for utilities may also be difficult in some areas. Also, artificial fill soils present on site may cave during trenching operations. In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back in accordance with Section 6.7 if workers are to enter such excavations.

6.2 Removal of Compressible Soils

The weathered upper portions of the very old Paralic Deposits and undocumented artificial fill soils at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structural loads supported on conventional foundations. The following recommendations are based on foundations extending to bear on competent Very Old Paralic Deposits.

In the building slab areas, we recommend that the upper 1 foot of soil below proposed subgrade elevations be removed and reprocessed in accordance with Section 6.3 below. Prior to placement of fill soil and in areas of planned improvements, the upper 6 inches of ground surface should be scarified, moisture conditioned as necessary, and properly recompacted.

In non-building areas, such as concrete hardscape, we recommended that the upper 1 feet of soil materials below proposed subgrade elevations should be removed and reprocessed in accordance with Section 6.3 below. Horizontally, the limits of the removal bottoms should extend at least 2 feet laterally beyond the limits of the proposed improvements.

In general, the soil that is removed may be reused and placed as engineered fill provided the material is moisture conditioned to at least 2 percent 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. The actual depth and extent of the required removals should be confirmed during grading operations by the geotechnical consultant.

6.3 Engineered Fill

The onsite soils are generally suitable for use as compacted fill provided they are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension. The onsite soils generally have moisture contents below optimum and may require moisture conditioning prior to use as compacted fill. All fill soils should be brought to at least 2 percent above-optimum moisture conditions and compacted in uniform lifts to at least 90 percent relative compaction based on laboratory standard ASTM Test Method D 1557. The optimum lift thickness required to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in lifts not exceeding 8 inches in thickness.

6.4 Cut/Fill Transition Mitigation

Although grading plans were not available at the time of this report, proposed cuts are expected to expose competent formation within portions of the pad not transected by deeper utilities and all footings are expected to be supported by Very Old Paralic Deposits.

6.5 Expansive Soils and Selective Grading

Based on our laboratory testing and observations, we anticipate the onsite soil materials possess a very low to low expansion potential (Appendix C). Selective grading to provide low expansion materials below slabs is not anticipated.

6.6 Import Soils

If import soils are used, the soil should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D 4829), and have a low corrosion impact to the proposed improvements. Beneath pavements, subgrade materials should possess an R-Value of 20, or greater. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import.

6.7 Temporary Excavations

Sloping excavations may be utilized when adequate space allows. Based on the results of our evaluation, we provide the following recommendations for sloped excavations in fill soils or Very Old Paralic Deposits without seepage conditions.

Table 2
Maximum Slope Ratios

Excavation Depth (feet)	Maximum Slope Ratio Fill Soils	Maximum Slope Ratio In Very Old Paralic Deposit
0 to 5	1:1 (Horizontal to Vertical)	1:1 (Horizontal to Vertical)

The above values are based on the assumption that no surcharge loading or equipment is present within 10 feet of the top of slope. Care should be taken during design of excavations adjacent to the existing structures so that foundation support is preserved. A “competent person” should observe the slope on a daily basis for signs of instability. All excavations should comply with current OSHA requirements.

6.8 Foundation Design

Based on our understanding of the project, we recommend that conventional spread footings founded in granular undisturbed Very Old Paralic Deposits to support the proposed structure. The ancillary canopy structures may be supported by drilled pier foundations. Where both shallow and deep foundation elements support the same structure, the superstructure should be analyzed in accordance with 2016 CBC Section 1808A.2. Where shallow foundations are constructed alongside existing shallow spread footings, any excavation below the depth of the bottom of the existing footing should be performed in a manner to avoid compromising the bearing capacity of the existing footings. The structural engineer should develop a plan showing the anticipated depth of the existing footings that are adjacent to the proposed Mary Birch Hospital Expansion foundations and are to be protected in-place.

6.8.1 Shallow Spread Footings

Shallow spread footings may be used to support the proposed hospital building. Where spread footings need to be deepened to bear on competent Very Old Paralic Deposits, a controlled low strength material (CLSM) can be used to fill the additional excavation prior to construction of the footing. The CLSM should consist of a two-sack, sand-cement slurry and have and have a minimum compressive strength of 125 psi when tested in accordance with ASTM D4832. Water content in the CLSM should be maintained at a proportion to minimize subsidence and bleed water shrinkage. The CLSM should be placed on competent materials. Any standing water and any loose or soft materials should be removed prior to placement of the CLSM. Deepening of spread footings should be anticipated where existing backfilled utility trenches are present where proposed foundations are planned.

Based on exhibits provided by the project structural engineer, we understand grade beams embedded 6 to 10 feet below the finish floor are planned to support the proposed hospital expansion. With the lowering of grades and the anticipated depth of grade beam foundations, we anticipate locally the additional depth needed to bear footings on competent materials will be less than 5 feet. The thickness of CLSM beneath footings should not exceed the width of the footing supported by the CLSM. If greater thickness is needed, the width of the excavation should be increased so that the thickness of the CLSM does not exceed the width of the CLSM. Alternatively, the structural engineer should provide a design for deepening the footing below the design bottom of footing depth.

Footings should extend a minimum of 24 inches beneath the lowest adjacent finish subgrade. At these depths, footings may be designed for a maximum allowable bearing pressure of 8,000 pounds per square foot (psf). This capacity is for dead plus live loads. With an ultimate capacity of at least 32,000 psf, the allowable bearing value may be increased by one-third for short-term wind or seismic loads. The minimum recommended width of footings is 18 inches for continuous footings and 24 inches for square or round footings. The allowable bearing pressures may be increased by 1,000 psf for each additional foot of width or depth of structural concrete, to a maximum value of 12,000 psf. For the allowable pressure of 12,000 psf, footings possess an ultimate value of at least 48,000 psf.

The recommended allowable-bearing capacity is based on a maximum total settlement of 5/8 inch and a differential of 3/8-inch. Since settlement is a function of footing size and contact bearing pressures, some differential settlement can be expected where a large differential loading condition exists. However, for most cases, differential settlements are considered unlikely to exceed 1/4 inch.

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.

6.8.2 Modulus of Subgrade Reaction

We understand the modulus of subgrade reaction will be used to model deflections for grade beams. Grade beams and mat foundations typically experience some deflection due to loads placed and the reaction of the soils underlying the foundations. A design coefficient of subgrade reaction of K_1, of 400 pounds per cubic inch (pci) may be used for evaluating such deflections at the site. This value is based on support by competent Very Old Paralic Deposits and is considered as applied to a unit square foot area. The value should be adjusted for the design foundation size. The coefficient of subgrade reaction K_b for a footing of specific width may be evaluated using the following equation.

$$K_b = K_1\ [(b+1)/2b]^2$$

where b is the least width of the foundation in feet

Detailed analysis to evaluate deflection should be carried out by the structural engineer. In some cases, refinement of the geotechnical recommendations may be needed to improve agreement between geotechnical and structural models.

6.8.3 Drilled Pile Foundations

Cast-in-drilled-hole (CIDH) friction piles at least 18 inches in diameter may be used to support the ancillary canopy structures. For the analysis and development of the vertical capacity of CIDH friction piles, an allowable downward skin friction of 200 psf may be utilized. No increase may be utilized for short term downward loads. For upward loads, a skin friction of 130 psf may be utilized and a one-third increase can be used for wind and seismic loads. Skin friction may be combined with end bearing for downwardly loaded piles where the bottom of the drilled pile excavation has been cleaned of any loose accumulation of cuttings, a value of 4,000 psf may be utilized for allowable end bearing.

Pile settlement is anticipated to be less than 1/4 inch under design loads and normal service conditions. The design skin friction is based on center to center pile spacing of at least 3 pile diameters from other excavations. Where piles or excavation are spaced more closely, a reduction in pile capacity is necessary. Construction of piles should be sequenced such that the concrete of constructed piles is allowed to setup prior to construction of piles within 5 diameters. Where excavations for later phases of buildings are planned near proposed foundations, extending footings deeper with structural concrete should be considered to mitigate impacts. Skin friction and end bearing may be relied upon within the portion of the pile that is at or below the depth of future excavation.

To resist lateral loads, CIDH piles can be designed in accordance with Section 1807A.3 of the 2016 CBC. For level ground conditions, we recommend lateral soil bearing pressures determined from Table 1806A.2 of 200 psf per foot of depth below the finish grade be used for determination of parameters S1 and S3 in the Non-constrained and Constrained designs, respectively. As allowed by Section 1806A.3.4, a two-times increase in lateral bearing pressure may be used for short term loading for buildings that are not adversely affected by ½-inch motion at the ground surface. These pressures assume piles spaced at least eight diameters center-to-center. Where piles are more closely spaced, lateral soil bearing pressures should be reduced using the appropriate reduction factor determined from Figure 10 or 11 (Caltrans, 2019). Where sloping ground is present, revised parameters should be provided. Where retaining structures are present or proposed, lateral surcharge may need to be considered in the retaining wall design to accommodate lateral pile surcharge loading. Similar considerations should be addressed if underground storage tanks are situated within eight pile diameters of laterally loaded piles.

Where the ground surface is level and buried utilities, vaults, tanks, or structures are not present within 8 pile diameters, piles at least 2 feet in diameter may be considered to be laterally supported and Exception 1 of Section 1810A.2.2 withn the 2016 CBC may be applied to piles with a length that does not exceed 12 times the least horizontal dimension .

6.8.4 Pile Installation

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 through the overlying fill soils and into the dense to very dense formational material to the design penetration depths. Once a pile excavation has been started, we recommend the pile be completed within 8 hours, which includes inspection, placement of the reinforcement, and placement of the concrete.

Caving of friable, soft or loose soils may occur where open excavations are made. Additionally, existing footings may surcharge excavations. Therefore, a permanent starter casing may be considered to protect the top of the borehole to mitigate caving or surcharge conditions where fill is present. The manner in which a permanent casing is constructed significantly affects the available skin friction. Where permanent casing is planned, we recommend that skin friction be neglected. Casing should be installed tight to the surrounding soil. Loose materials should be removed from the bottom of the pile excavation prior to concrete placement.

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.

6.8.5 Foundation Setback

We recommend a minimum horizontal setback distance from the face of slopes and retaining walls for all structural foundations, footings, and other settlement-sensitive structures as indicated on the Table 3 below. This distance is measured from the outside bottom edge of the footing, horizontally to the slope face, and is based on the slope 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 3

Minimum Foundation Setback from Slope Faces

Slope Height	Setback
less than 5 feet	5 feet
5 to 15 feet	7 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. Depending on their proximity to the top of slopes, these structures may require retaining walls and/or deepened foundations.

In addition, open or backfilled utility trenches that parallel or nearly parallel structure footings should not encroach within an imaginary 2 to 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.

Where pipes may 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.

6.8.6 Floor Slabs

Slabs-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.

For slab areas where vapor control is appropriate, a minimum 15-mil vapor barrier should be provided between the underslab and gravel capillary break. The vapor barrier should have a permeance of less than 0.01 perms across the entire slab area in the final constructed condition. Measures to protect the barrier should be implemented throughout the installation and slab construction process to prevent damage (ASTM E1643). Vapor barrier materials should conform to ASTM E1745 Class A. The gravel capillary break should consist of a layer of uniform 3/8-inch to 1/2-inch gravel that is at least 4-inches thick. The mix design of the slab concrete should be proportioned to control bleeding, shrinkage and curling.

Moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slabs. Moisture barriers can also prolong the timeframe needed for slabs to fully cure. We recommend that the floor covering/insulation installer test the moisture vapor flux rate prior to flooring installation. “Breathable” floor coverings should be considered if the vapor flux rates are high. Additional guidance is provided in ACI Publications 302.1R-15 Guide for Concrete Floor and Slab Construction and 302.2R-06 Guide for Concrete Slab that Receive Moisture-Sensitive Floor Materials.

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

6.8.7 Loading Dock Slab

The project includes a loading dock with capacity to receive up to 6 trucks at a time. A PCC pavement section for the proposed loading dock slab has been provided based on the design standards presented in the ACI "Guide for the Design and construction of Concrete Parking Lots" (ACI 330R-08) and the assumed Average Daily Truck Traffic Indices (ADTT). The ADTT is to be determined by the design-build designers.

Table 4
PCC Pavement Sections

ADTT*	PCC (Inches)
>700	8.5
≤ 300	7.5
≤ 10	6.5

*Traffic Categories and ADTT per ACI 330, Table 3.3.

The above recommended concrete sections are based on properly compacted fill soils with a very low expansion potential (EI<21) and R-Value greater than 25. They also include a thickness increase of 15% to account for a free edge condition. All utility trenches should be compacted to 90 percent relative compaction and pavement subgrade (upper 12-inches) uniformly compacted (non-yielding) to 95 percent of the laboratory maximum dry density (ASTM D1557) and at/or slightly above optimum moisture content. Compaction should extend a minimum of 12-inches beyond formlines. Slab edges and construction joint details provided by ACI should be followed. Concrete should have a minimum flexural strength of 550 psi. Concrete testing should be performed to confirm quality of aggregates, strength requirements and shrinkage limits during construction. Construction and crack control joints should be designed per structural engineer's requirements ACI guidelines.

6.8.8 Lateral Earth Pressures and Retaining Wall Design

Should retaining walls be added to the project, Table 6 presents the lateral earth pressure values for level or sloping backfill for walls backfilled with and bearing against fully drained soils of very low to low expansion potential (less than 50 per ASTM D 4829).

Table 5
Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	36	55
At-Rest	55	80
Passive	300 (Maximum of 3 ksf)	150 (Sloping Down)

Walls up to 10 feet in height should be designed for the applicable equivalent fluid unit weight values provided above. If conditions other than those covered herein are anticipated, the equivalent fluid unit weight 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 E. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D 1557). If foundations are planned over the wall backfill, the wall 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 Section 6.8.5.

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 0.35 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. The passive resistance and frictional coefficients are allowable values with a factor of safety of 1.5. The passive value for level ground assumes level conditions extend horizontally at least eight times the height of the surface imposing the horizontal loading.

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 2016 CBC Section 1803.5.12 and/or ASCE 7-10 Section 15.6.1 and should also be analyzed for seismic loading. For that analysis, an additional uniform lateral seismic force of 9H 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.

6.8.9 Shoring of Excavations

For deeper excavations and protection of existing foundations, we recommend that excavations be retained either by a cantilever or braced shoring system with cast-in-place soldier piles and sheeting or lagging (i.e. shotcrete and/or wood), as needed. Based on our experience with similar projects, if lateral movement of the shoring system cannot be tolerated, we recommend the utilization of a braced or anchored pile system.

Shoring of excavations is typically performed by specialty contractors with knowledge of the San Diego County area soil conditions. Lateral earth pressures for design of shoring are presented below:

Cantilever Shoring System

Active pressure = 36H(psf), triangular distribution
Passive Pressure = 400h (psf)
H = wall height (active case) or h = embedment (passive case)

Multi-Braced Shoring System

Active Pressure = 24H (psf), rectangular distribution
Passive Pressure = 400h (psf)
H = wall height (active case) or h = embedment (passive case)

Based on subsurface materials encountered during the geotechnical exploration and our experience with nearby projects, it is our opinion that the caving potential of the on-site soils is moderate due to the presence of dense to very dense, but yet friable sands and gravels associated with the underlying Very Old Paralic Deposits. 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 should be anticipated. During downward advancement of the shoring walls care in these cases should be exercised which may include the excavation of shorter open-face segments.

If portions of the planned excavations are proposed with sloped temporary excavations, we recommend a maximum slope of 1 to 1 (horizontal to vertical). Sloped excavations should be observed by the geotechnical consultant during excavation. It should be noted that where temporary slopes excavate proposed foundational soil, then proposed footings will need to be deepened to bear on competent formation.

Settlement monitoring of adjacent building, sidewalks and adjacent settlement sensitive structures should be considered 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.

6.9 Control of Surface Waters

Regarding Best Management Practices (BMP) and Low Impact Development (LID) measures, we are of the opinion that infiltration basins, and other on-site storm water retention and infiltration systems can potentially create adverse perched groundwater conditions, both on-site and off-site, when not installed using proper design recommendations (such as the use of liners) and infiltration design parameters. Due to the dense nature of the Very Old Paralic Deposits and existing site constraints and conditions, we do not recommend infiltration of surface storm water into the existing site soils. However, Low Impact Development (LID) BMPs that contain and filter surface waters (flow-through planters and bioretention areas) are acceptable provided that they are completely lined with an impermeable liner and have subdrain systems that tie into an approved existing or proposed storm drain system.

Surface storm 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. 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.

6.10 Non-Vehicular 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 with No. 4 bars at 24 inches on center or No. 3 bars at 18 inches on center. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 2 percent above optimum moisture content depending on the soil type and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement. Moisture testing should be confirmed 24 hours prior to concrete placement.

6.11 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 test results (Appendix C) indicate an exposure class of S0. We recommend that concrete in contact with earth materials be designed in accordance with Section 4 of ACI 318-14 (ACI, 2014).

Based on the results of preliminary screening laboratory testing, the site soils have a corrosive potential to buried uncoated metal conduits (Caltrans, 2018). We recommend measures to mitigate corrosion be implemented during design and construction. Leighton does not practice corrosion engineering. Therefore, a corrosion engineer may be contacted for additional recommendations.

6.12 Construction Observation and Plan Reviews

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced borings. The interpolated subsurface conditions should be checked 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 so that construction is in accordance with the recommendations of this report. We recommend that where possible, excavation exposures be geologically mapped by the geotechnical consultant during grading for the presence of potentially adverse geologic conditions.

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

7.0 LIMITATIONS

The conclusions and recommendations presented 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. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

Figures

Project: 12764.001	Eng/Geol: SAC/RCS
Scale: 1 " = 500 '	Date: September 2020

Base Map: Bing Maps 2020
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

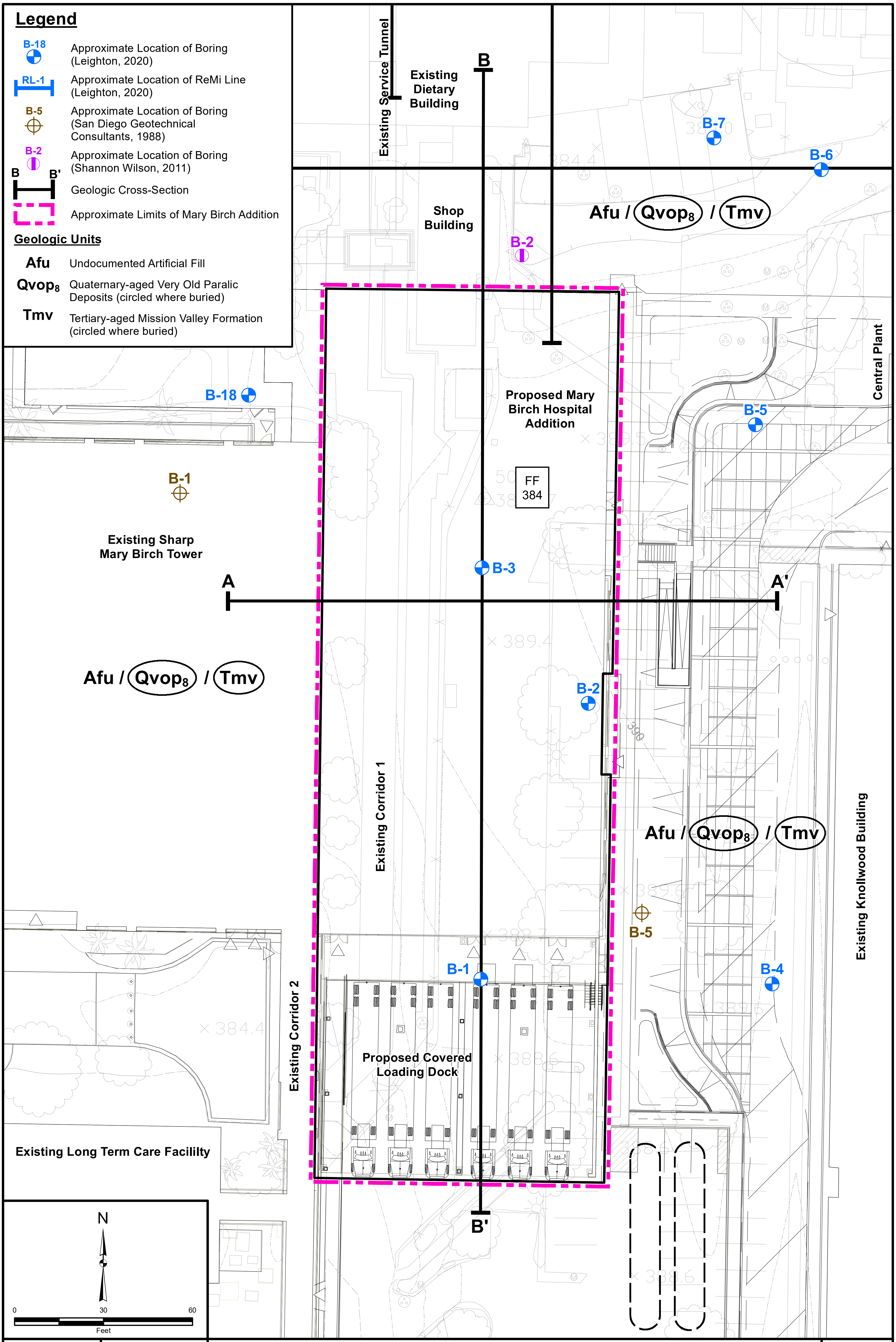
SITE LOCATION MAP

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 1

Leighton

Map Saved as P:\Drafting\12764\Maps\Mary Birch\12764-001_F01_SLM-MB_2020-08-14.mxd on 9/3/2020 9:28:25 AM



Legend

B-18	Approximate Location of Boring (Leighton, 2020)
RL-1	Approximate Location of ReMi Line (Leighton, 2020)
B-5	Approximate Location of Boring (San Diego Geotechnical Consultants, 1988)
B-2	Approximate Location of Boring (Shannon Wilson, 2011)
B B'	Geologic Cross-Section
	Approximate Limits of Mary Birch Addition

Geologic Units

Afu	Undocumented Artificial Fill
Qvop$_8$	Quaternary-aged Very Old Paralic Deposits (circled where buried)
Tmv	Tertiary-aged Mission Valley Formation (circled where buried)

Project: 12764.001

Eng/Geol: SAC/RCS

Scale: 1 " = 30 '

Date: December 2020

Base Maps: HDR/Taylor Design, 2020, Site Development – Package 2, Design Development – 100%, SMMC Campus Redevelopment, plot date September 15. and BWE, 2020, Site Plan – Package 2, Design Development – 50%, SMMC Campus Redevelopment, plot date June 10.

Author: (kmanchikanti)

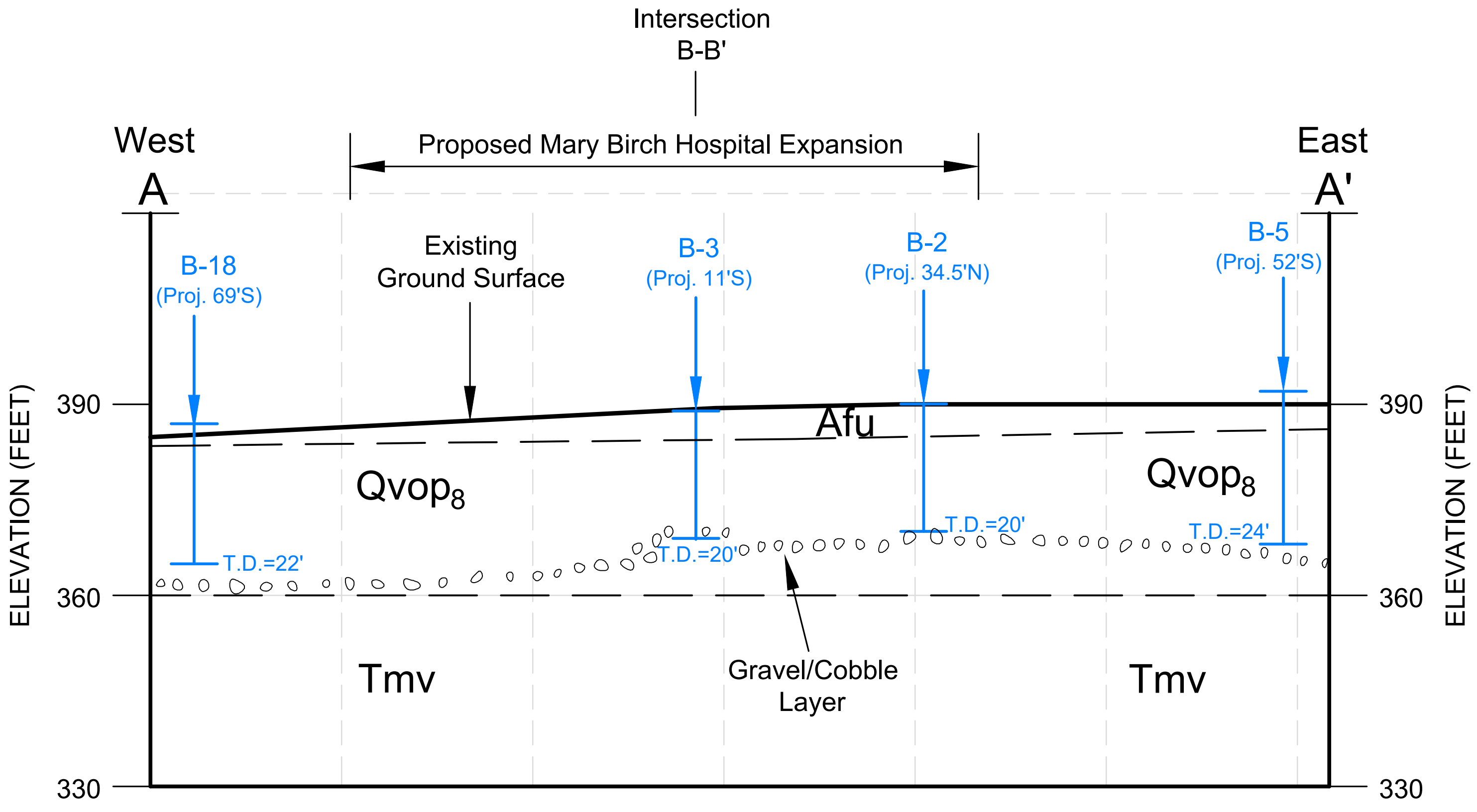
GEOTECHNICAL MAP

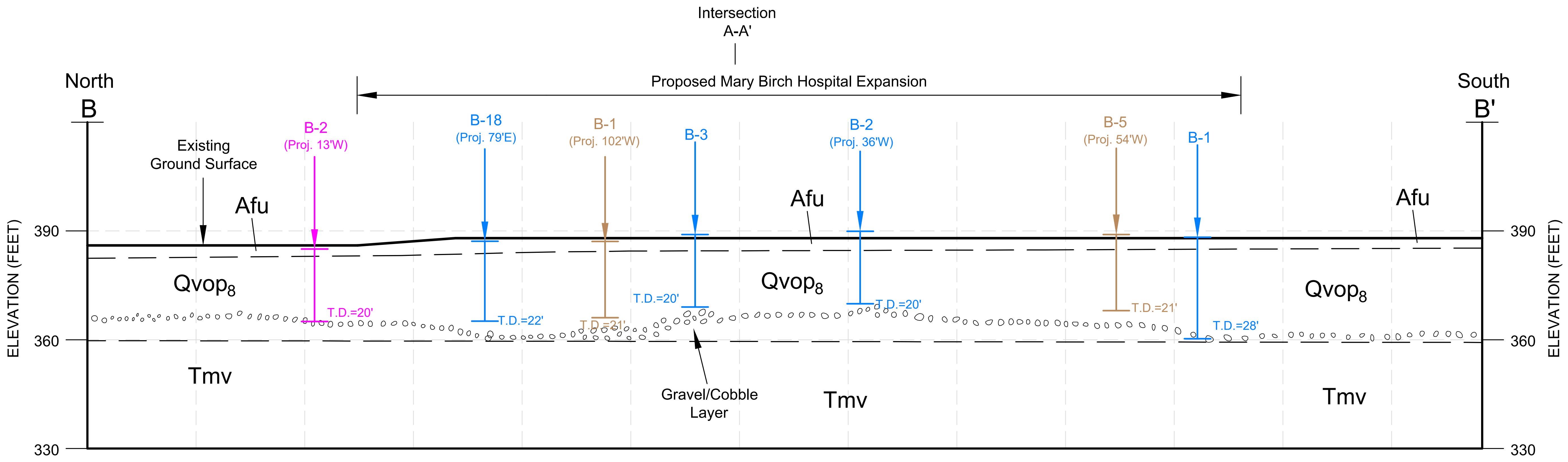
Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 2

Leighton

Map Saved as C:\Users\kmanchikanti\Documents\Maps\12764-001_F02_GM_2020-08-18.mxd on 12/8/2020 1:10:45 PM





Legend

B-18 T.D.=22'	Boring Location showing Total Depth (Leighton, 2020)
B-5 T.D.=21'	Boring Location showing Total Depth (San Diego Geotechnical Consultants, 1988)
B-2 T.D.=18.5'	Boring Location showing Total Depth (Shannon Wilson, 2011)
— — —	Approximate Geologic Contact

Geologic Units

Afu	Undocumented Artificial Fill
$Qvop_8$	Quaternary-aged Very Old Paralic Deposits
Tmv	Tertiary-aged Mission Valley Formation

Project: 12764.001
Eng/Geol: SAC/RCS
Scale: 1"=30'
Date: September 2020
Reference:
Author: MAM

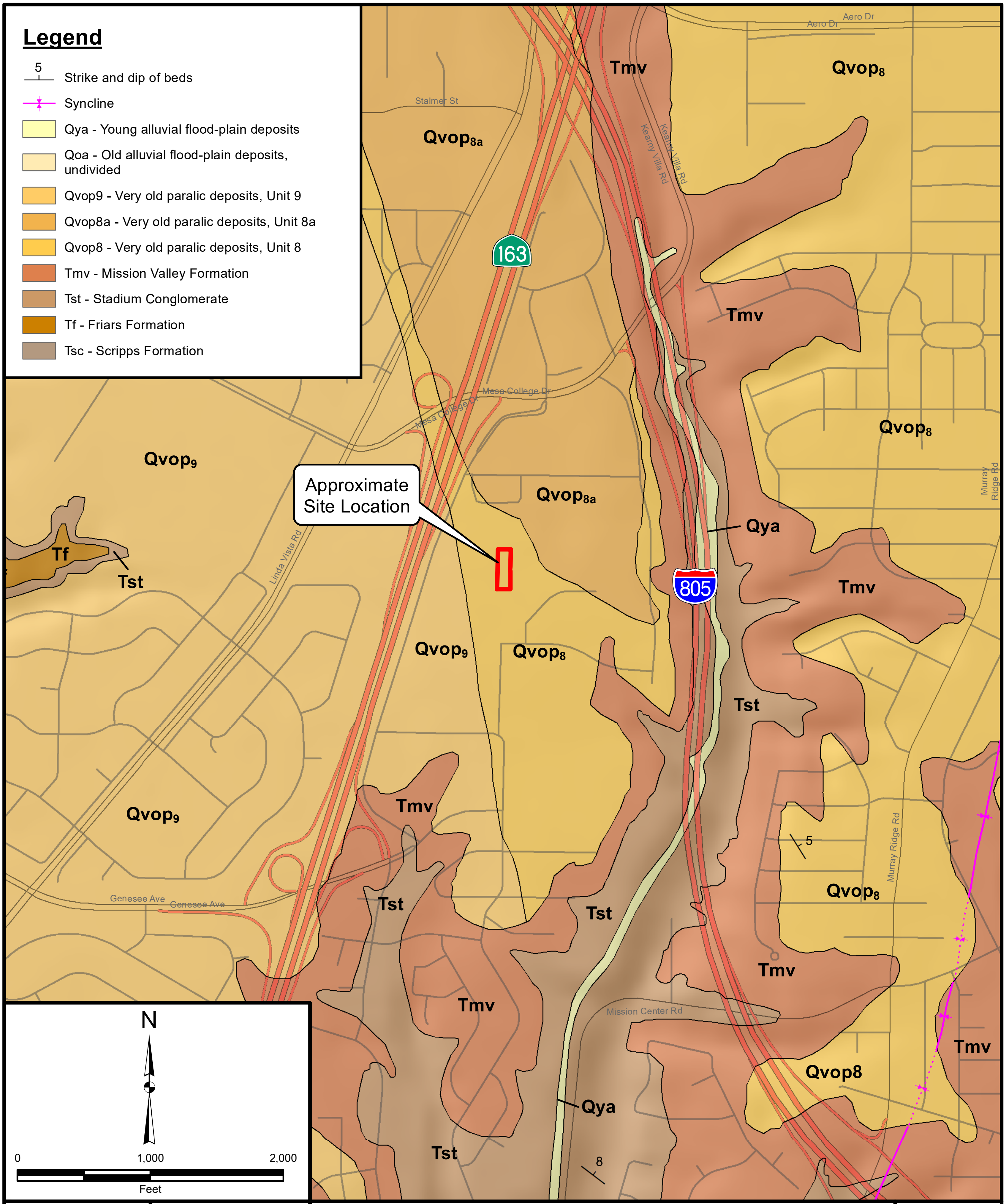
GEOLOGIC CROSS SECTIONS A-A' & B-B'

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 3

Leighton

V:\DRAFTING\12764\CAD\MARY BIRCH ADDITION\12764.001_F03_GCSAA_2020-08-18.DWG (09-24-20 11:51:27AM) Plotted by: mmurphy



Project: 12764.001	Eng/Geol: SAC/RCS
Scale: 1 " = 1,000 '	Date: September 2020

Base map: Kennedy, M.P. and Tan, S.S, 2008, Geologic Map of the San Diego 30' X 60' Quadrangle, California Compiled by Michael P. Digital Preparation by Kelly R. Bovard, Anne G. Garcia and Diane Burns, California Geological Survey.
Author: Leighton Geomatics (mmurphy)

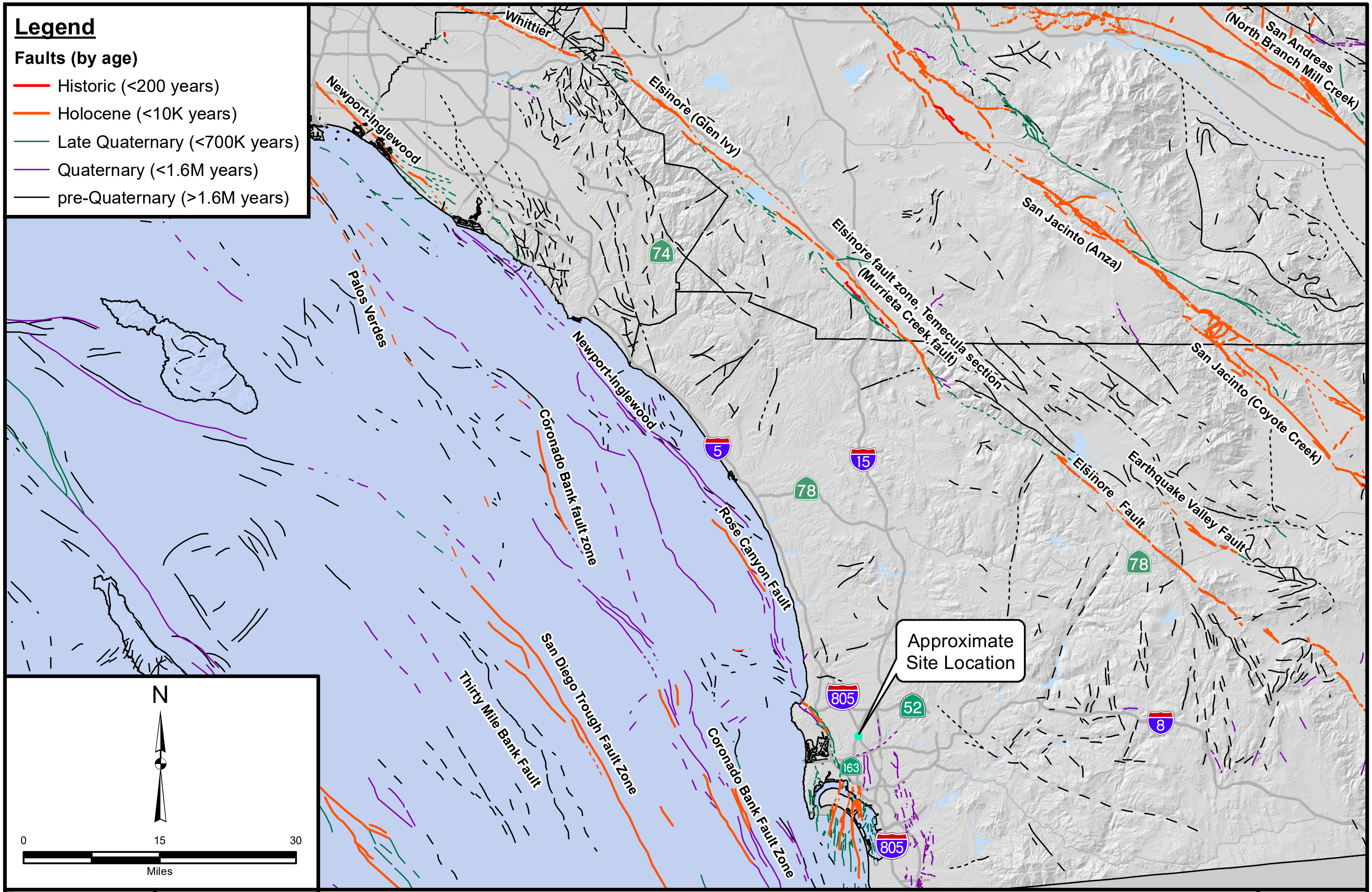
GEOLOGIC MAP

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 4

Leighton

Map Saved as P:\Drafting\12764\Maps\Mary Birch\12764-001_F04_GM_2020-08-14.mxd on 9/3/2020 9:31:52 AM



Project: 12764.001	Eng/Geol: SAC/RCS
Scale: 1 " = 15 miles	Date: September 2020

Faults: Bryant, Bryant CGS 2010
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

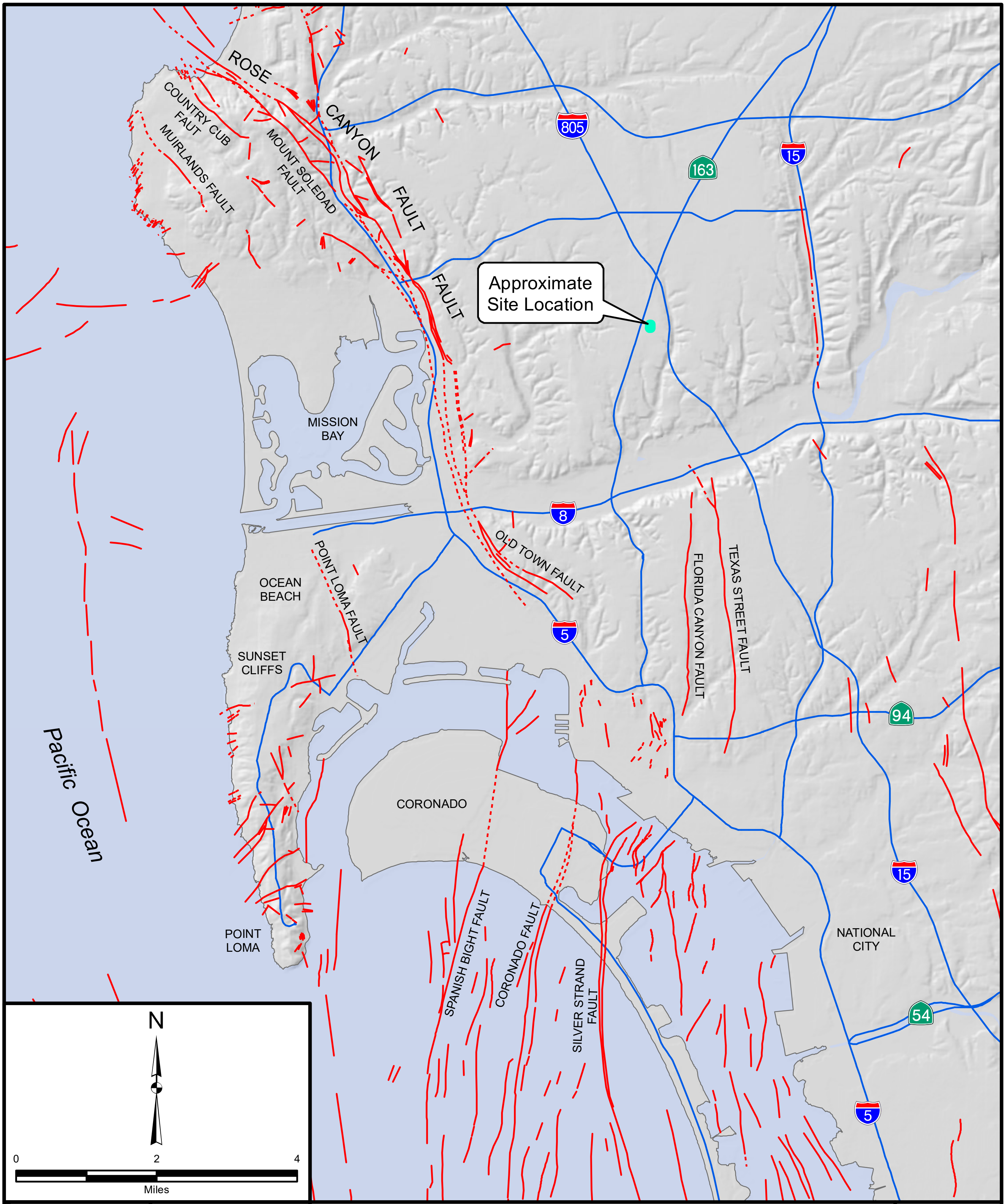
REGIONAL FAULT MAP

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 5

Leighton

Map Saved as P:\Drafting\12764\Maps\Mary Birch\12764-001_F05_RFM_2020-08-14.mxd on 8/14/2020 10:50:12 AM



Project: 12764.001

Eng/Geol: SAC/RCS

Scale: 1 " = 2 miles

Date: September 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

ROSE CANYON FAULT MAP

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 6

Leighton

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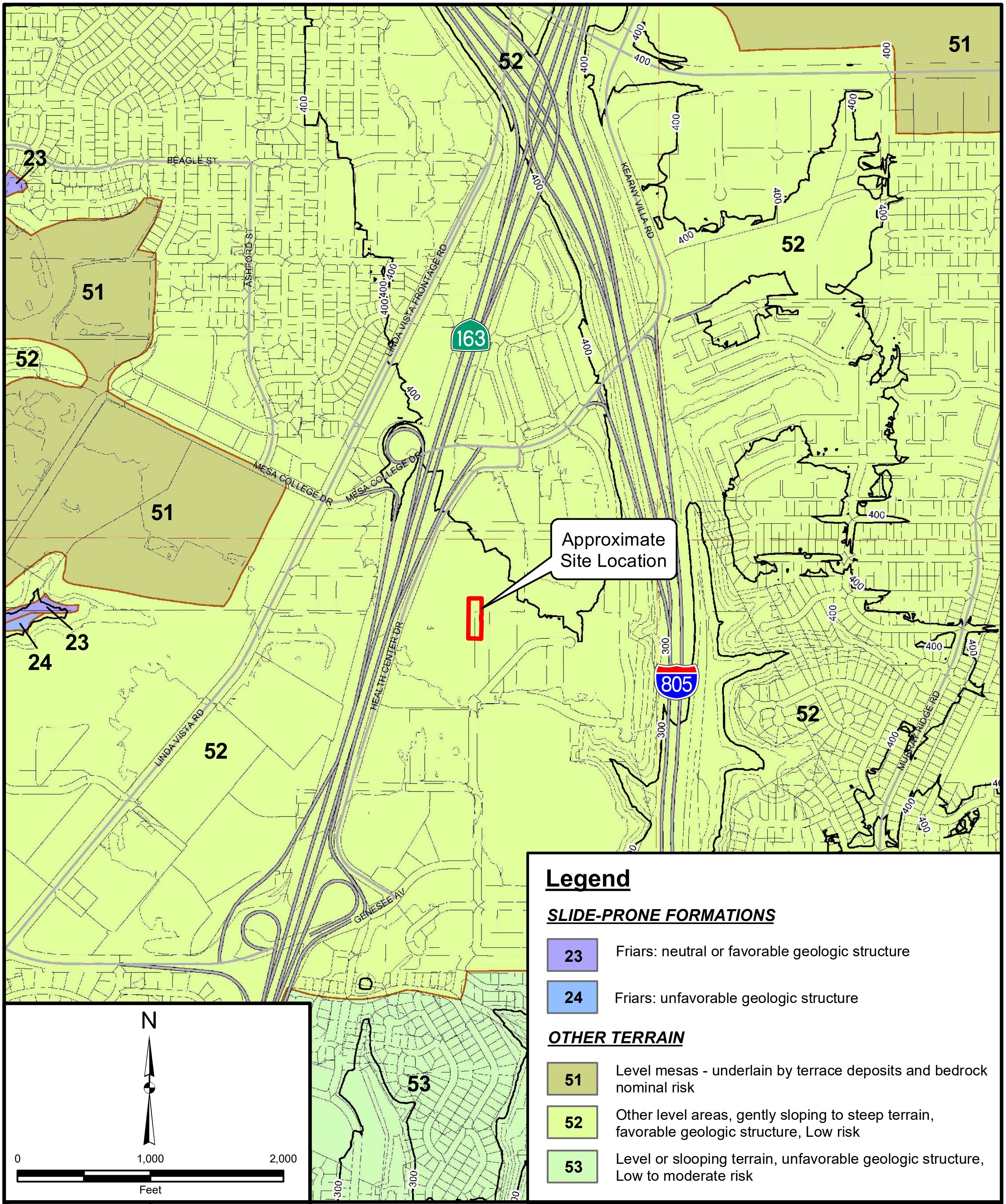


Legend

SLIDE-PRONE FORMATIONS

23	Friars: neutral or favorable geologic structure
24	Friars: unfavorable geologic structure

OTHER TERRAIN

51	Level mesas - underlain by terrace deposits and bedrock nominal risk
52	Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk
53	Level or slooping terrain, unfavorable geologic structure, Low to moderate risk

Project: 12764.001

Eng/Geol: SAC/RCS

Scale: 1 " = 1,000 '

Date: September 2020

Reference: City of San Diego Seismic Safety Study, Geologic Hazards and Faults, dated 4/3/2008.

Author: Leighton Geomatics (mmurphy)

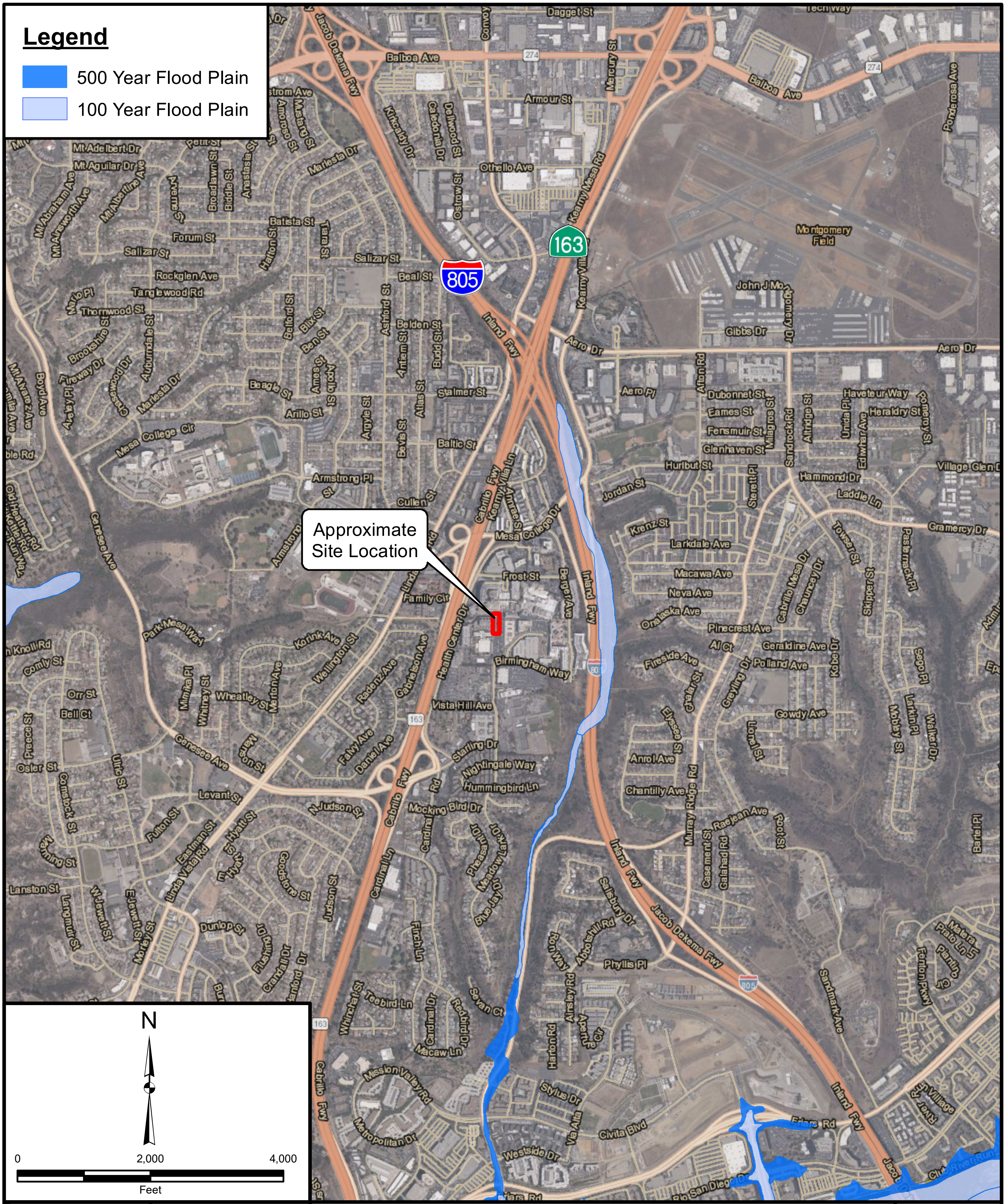
CITY OF SAN DIEGO SEISMIC SAFETY STUDY MAP

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 7

Leighton

Map Saved as P:\Drafting\12764\Maps\Mary Birch\12764-001_F07_CSDSSSM_2020-08-14.mxd on 9/3/2020 9:36:47 AM



Legend

🟦 500 Year Flood Plain

▭ 100 Year Flood Plain

N

0 2,000 4,000

Feet

Project: 12764.001

Eng/Geol: SAC/RCS

Scale: 1 " = 2,000 '

Date: September 2020

Base Map: Bing Maps 2020

Thematic Information: SanGIS, 2020.

Author: Leighton Geomatics (mmurphy)

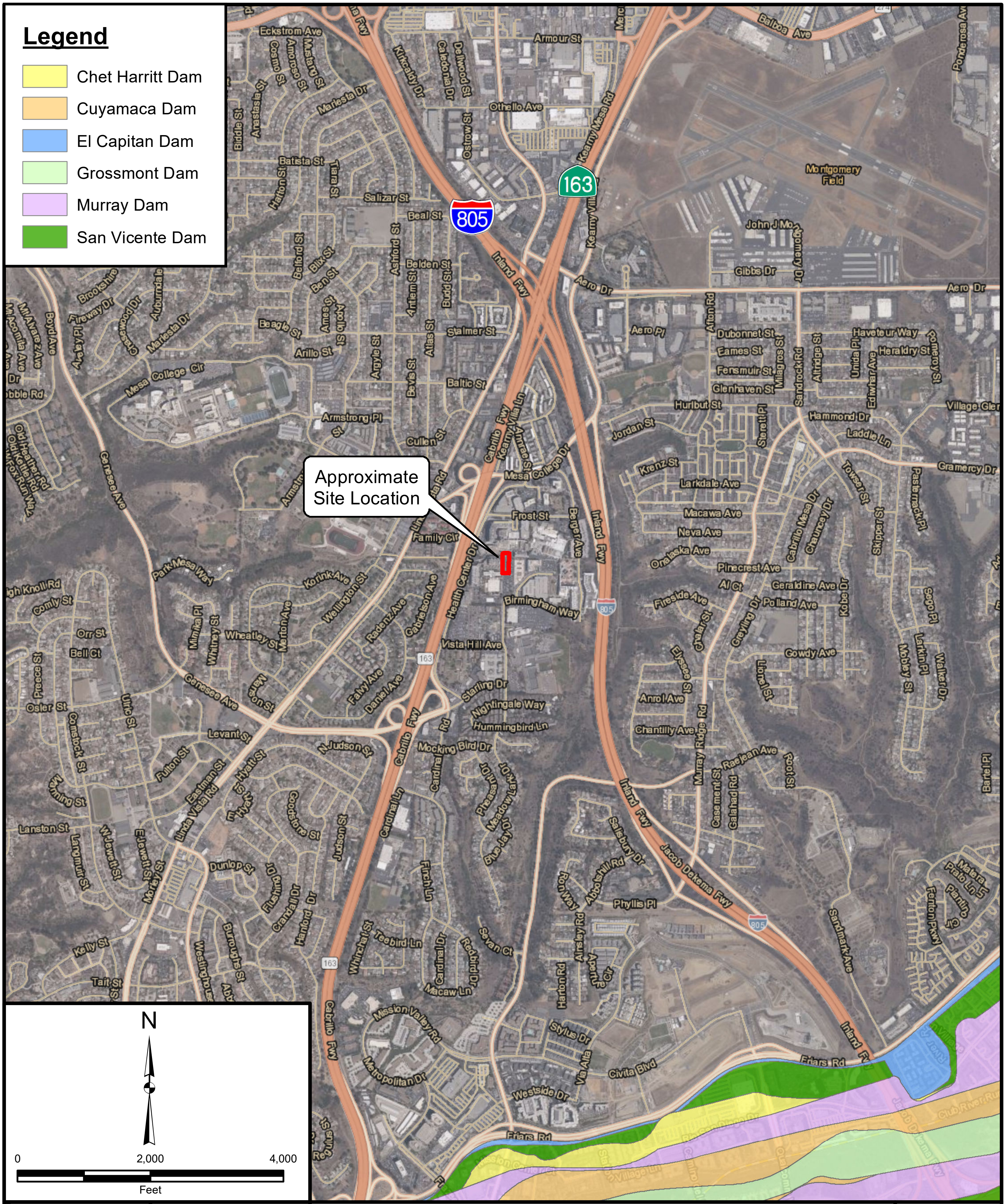
FLOOD HAZARD MAP

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 8

Leighton

Map Saved as P:\Drafting\12764\Maps\Mary Birch\12764-001_F08_FHM_2020-08-14.mxd on 9/3/2020 9:37:56 AM



Base Map: Bing Maps 2020
Thematic Information: Leighton, SanGIS, 2020.
Author: Leighton Geomatics (mmurphy)

DAM INUNDATION MAP

Mary Birch Hospital Expansion
Sharp Metropolitan Medical Campus
San Diego, California

Figure 9

Leighton

Map Saved as P:\Drafting\12764\Maps\Mary Birch\12764-001_F09_DIM_2020-08-17.mxd on 9/3/2020 9:39:15 AM

Appendix A
References

APPENDIX A
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Appendix B
Boring Logs

GEOTECHNICAL BORING LOG KEY

Date __________ **Sheet** 1 **of** 1

Project KEY TO BORING LOG GRAPHICS **Project No.** __________

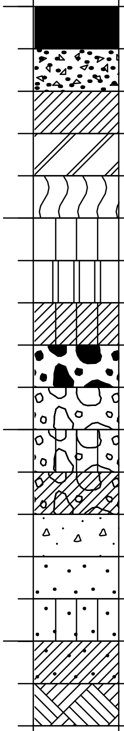
Drilling Co. __________ **Type of Rig** __________

Hole Diameter __________ **Drive Weight** __________ **Drop** ___ "

Elevation Top of Elevation ___ ' **Location** __________

Logged By __________

Sampled By __________



Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	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 1.	
	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:

S	SPLIT SPOON
R	RING SAMPLE
B	BULK SAMPLE
T	TUBE SAMPLE
G	GRAB SAMPLE
SH	SHELBY TUBE

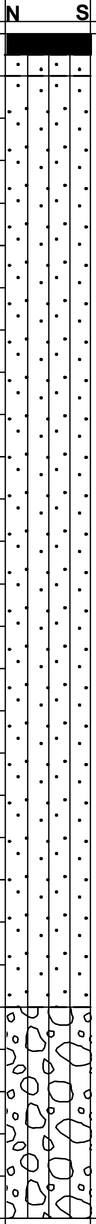
TYPE OF TESTS:

DS	DIRECT SHEAR
MD	MAXIMUM DENSITY
CN	CONSOLIDATION
CR	CORROSION
SA	SIEVE ANALYSIS
AT	ATTERBURG LIMITS
EI	EXPANSION INDEX
RV	R-VALUE

LEIGHTON

GEOTECHNICAL BORING LOG B-1

Project No.	12764.001	**Date Drilled**	7-29-20
Project	Sharp Metro Master Plan (MBH)	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	389' msl
Location	See Figure 2	**Sampled By**	RNB



Elevation Feet	Depth Feet	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							5" ASPHALT CONCRETE	
							SM	UNDOCUMENTED ARTIFICIAL FILL (Afu)	
			B-1 1'-5'			3.6	SM	@ 5"-1': Silty SAND, loose to medium dense, dark reddish brown (5 yr 3/4), moist, fine-grained	EI, CR
								VERY OLD PARALIC DEPOSITS (Qvop8) @ 1': Silty SANDSTONE, medium dense to dense, yellowish red (5 yr 5/6), moist, fine-grained, trace oxidiation	
385									
	5		R-1	50/4"				@ 5': Becomes very dense	DS
380									
	10		R-2	50/5"		5			-200
375									
	15		R-3	33 50/5"	111	8		@ 15': Becomes reddish brown (5 yr 4/4)	
370									
	20		R-4	43 50/4"	108	8			
							GM	@ 23': Cobble CONGLOMERATE, very dense, light reddish brown (5 yr 6/4), moist, cobble/gravel is well-rounded, fine-grained sand matrix	
365									
	25		S-1	50/1"					
			S-2	50/1"					
								Auger Refusal on Cobble at 28 Feet (bgs) **No Groundwater or Seepage Encountered** **Backfilled with Bentonite Grout on 7/29/2020**	
360									
	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.	12764.001	**Date Drilled**	7-29-20
Project	Sharp Metro Master Plan (MBH)	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	390' msl
Location	See Figure 2	**Sampled By**	RNB

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
390	0								4" ASPHALT CONCRETE	
				B-1 1'-5'				SM	UNDOCUMENTED ARTIFICIAL FILL (Afu)) @ 4"-1.5': Silty SAND, loose to medium dense, dark reddish brown (2.5 yr 3/4), moist, fine-grained	
								SM	VERY OLD PARALIC DEPOSITS (Qvop8) @ 1.5': Silty SANDSTONE, medium dense to dense, red (2.5 yr 4/8), moist, fine-grained, trace oxidiation	MD
385	5			R-1	50/4"				@ 5': Becomes very dense	DS
380	10			R-2	50/3"	112	6		@ 10': Becomes red (2.5 yr 4/8) and mottled with light yellowish brown (10 yr 6/4)	
375	15			S-1	21 28 32					-200
								GM	@ 19': Cobble CONGLOMERATE, very dense, light reddish brown (5 yr 6/4), moist, cobble/gravel is well-rounded, fine-grained sand matrix	
370	20			S-2	50/1"					
									Auger Refusal on Cobble at 20 Feet (bgs) **No Groundwater or Seepage Encountered** **Backfilled with Soil Cuttings on 7/29/2020**	
365	25									
360	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:

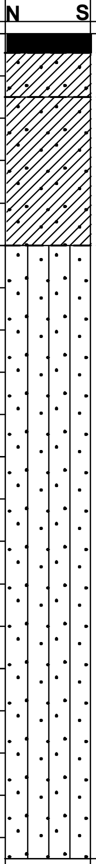
-200	% FINES PASSING
AL	ATTERBERG LIMITS
CN	CONSOLIDATION
CO	COLLAPSE
CR	CORROSION
CU	UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
H	HYDROMETER
MD	MAXIMUM DENSITY
PP	POCKET PENETROMETER
RV	R VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
SG	SPECIFIC GRAVITY
UC	UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-3

Project No. 12764.001
Project Sharp Metro Master Plan (MBH)
Drilling Co. Baja Exploration
Drilling Method CME-95 - 140lb - Autohammer - 30" Drop
Location See Figure 2
Date Drilled 7-29-20
Logged By RNB
Hole Diameter 8"
Ground Elevation 389' msl
Sampled By RNB

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	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SC	4-1/2" ASPHALT CONCRETE UNDOCUMENTED ARTIFICIAL FILL (Afu)) @ 4-1/2": Clayey SAND, loose to medium dense, dark reddish brown (7.5 yr 3/4), moist, fine-grained	
								SC	VERY OLD PARALIC DEPOSITS (Qvop8) @ 1.5': Clayey SANDSTONE, medium dense to dense, red (2.5 yr 4/8), moist, fine-grained, trace oxidation	
385										
	5			R-1 B-1 5'-10'	50/6"	122	7	SM	@ 5': Silty SANDSTONE, very dense, red (2.5 yr 4/8), moist, fine-grained, trace oxidation	
380										
	10			R-2	50/6"	113	10		@ 10': Becomes light yellowish brown (10 yr 6/4)	
375										
	15			R-3	37 50/3"	116	8			
				S-1	19 26 32				@ 18': Becomes light yellowish brown (10 yr 6/4) mottled with reddish brown (2.5 yr 4/4)	
370										
	20								**Bottom of Boring at 19.5 Feet (bgs)** **No Groundwater or Seepage Encountered** **Backfilled with Soil Cuttings on 7/29/2020**	
365										
	25									
360										
	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.	12764.001	**Date Drilled**	7-29-20
Project	Sharp Metro Master Plan (MBH)	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	390' msl
Location	See Figure 2	**Sampled By**	RNB

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
390	0								5" ASPHALT CONCRETE	
				B-1 1'-5'				SC	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ .5": Clayey SAND, loose, dark reddish brown (5 yr 3/4), moist, fine-grained	RV
								SC	VERY OLD PARALIC DEPOSITS (Qvop8) @ 2.5': Clayey SANDSTONE, medium dense to dense, yellowish red (5 yr 5/6), moist, fine-grained, trace oxidation	
385	5			R-1	50/4"	101	6	SM	@ 5': Silty SANDSTONE, very dense, yellowish red (5 yr 5/6), moist, fine-grained, trace oxidation	
380	10			R-2	50/5"	110	6		@ 10': Becomes reddish brown (5 yr 4/4)	
375	15			R-3	27 50/5"	119	11		@ 15': Becomes reddish brown (5 yr 4/4) mottled with light yellowish brown (10 yr 6/4)	
370	20			S-1	27 34 50/4"			SM	MISSION VALLEY FORMATION (Tmv) @ 20': Silty SANDSTONE, very dense, light yellowish brown (10 yr 6/4), moist, fine-grained	
				S-2	50/1"				@ 24': Gravel/Cobble layer encountered	
365	25								**Auger Refusal on Cobble at 24 Feet (bgs)** **No Groundwater or Seepage Encountered** **Backfilled with Bentonite Grout on 7/29/2020**	
360	30									

SAMPLE TYPES:

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

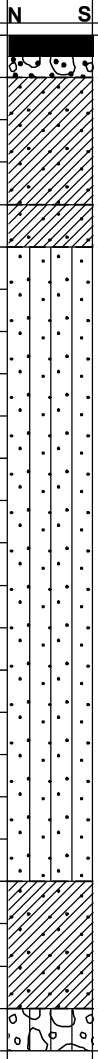
TYPE OF TESTS:

-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-5

Project No.	12764.001	**Date Drilled**	7-30-20
Project	Sharp Metro Master Plan (MBH)	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	390' msl
Location	See Figure 2	**Sampled By**	RNB



Elevation Feet	Depth Feet	Graphic Log	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
390	0								5" ASPHALT CONCRETE over 4" AGGREGATE BASE	
				B-1 1'-5'				SC	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 9"-4': Clayey SAND, loose to medium dense, dark reddish brown (5 yr 3/4), moist, fine-grained, trace gravel	-200, EI, CR
								SC	VERY OLD PARALIC DEPOSITS (Qvop8) @ 4': Clayey SANDSTONE, medium dense to dense, red (2.5 yr 4/6), moist, fine-grained, trace oxidation	
385	5			R-1	50/4"			SM	@ 5': Silty SANDSTONE, very dense, red (2.5 yr 4/6), moist, fine-grained, trace oxidation	DS
380	10			R-2	39 50/5"	111	14		@ 10': Becomes dark reddish brown (2.5 yr 2.5/4)	
375	15			R-3	33 50/5"	120	10			
370	20			R-4	37 50/5"	117	7	SC	@ 20': Clayey SANDSTONE, very dense, red (2.5 yr 4/6), moist, fine- to medium-grained, trace gravel, trace oxidation	-200
				S-1	50/2"			GM GM	@ 23': Cobble CONGLOMERATE, very dense, light reddish brown (5 yr 6/4), to reddish brown (2.5 yr 4/4), moist, cobble/gravel is well-rounded, fine-grained sand matrix	
365	25								**Auger Refusal on Cobble at 24 Feet (bgs)** **No Groundwater or Seepage Encountered** **Backfilled with Bentonite Grout on 7/30/2020**	
360	30									

SAMPLE TYPES:

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

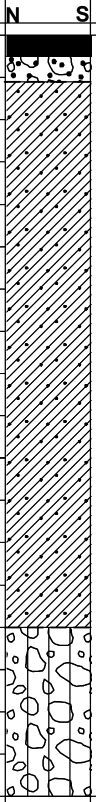
TYPE OF TESTS:

-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-6

Project No.	12764.001	**Date Drilled**	7-30-20
Project	Sharp Metro Master Plan (MBH)	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	386' msl
Location	See Figure 2	**Sampled By**	RNB



Elevation Feet	Depth Feet	Graphic Log	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								5" ASPHALT CONCRETE over 6" AGGREGATE BASE	
385				B-1 1'-5'				SC	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 11": Clayey SAND, loose, dark reddish brown (5 yr 3/4), moist, fine-grained, trace gravel	MD, CR
	5			R-1	7 15 16	115	9		@ 5': Becomes medium dense	
380										
	10			R-2	4 5 6	115	9		@ 10': Becomes loose	
375										
								GM	VERY OLD PARALIC DEPOSITS (Qvop) @ 14': Cobble CONGLOMERATE, very dense, light reddish brown (5 yr 6/4) to reddish brown (2.5 yr 4/4), moist, cobble/gravel is well-rounded, fine-grained sand matrix, trace oxidation staining	
	15			S-1	50/1"					
370										
				S-2	50/3"					
									Auger Refusal on Cobble at 18 Feet (bgs) **No Groundwater or Seepage Encountered** **Backfilled with Soil Cuttings on 7/30/2020**	
	20									
365										
	25									
360										
	30									

SAMPLE TYPES:

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

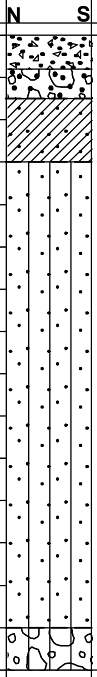
TYPE OF TESTS:

-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-7

Project No.	12764.001	**Date Drilled**	7-30-20
Project	Sharp Metro Master Plan (MBH)	**Logged By**	RNB
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	384' msl
Location	See Figure 2	**Sampled By**	RNB



Elevation Feet	Depth Feet	Graphic Log	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								8" Reinforced CONCRETE over 9" AGGREGATE BASE	
				B-1 1.5'-5'				SC	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 17": Clayey SAND, loose to medium dense, dark reddish brown (5 yr 3/4), moist to very moist, fine-grained, trace gravel	EI
								SM	VERY OLD PARALIC DEPOSITS (Qvop8) @ 3': Silty SANDSTONE, medium dense to dense, red (2.5 yr 4/6), moist, fine-grained, trace oxidation	
380										
	5			R-1	33 50/4"				@ 5': Becomes very dense	DS
375										
	10			R-2	25 36 50/6"	106	8			
370								GM	@ 14': Cobble CONGLOMERATE, very dense, reddish brown (2.5 yr 4/4), moist, gravel/cobble is well-rounded, fine- to medium-grained sand matrix, trace oxidation staining	
	15			S-1	50/1"					
									Auger Refusal on Cobble at 15 Feet (bgs) **No Groundwater or Seepage Encountered** **Backfilled with Soil Cuttings on 7/30/2020**	
365										
	20									
360										
	25									
355										
	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-18

Project No. 12764.001
Project Sharp Metro Master Plan (MBH)
Drilling Co. Baja Exploration
Drilling Method LAR - 140lb - Autohammer - 30" Drop
Location See Figure 2

Date Drilled 8-6-20
Logged By RNB
Hole Diameter 8"
Ground Elevation 387' msl
Sampled By RCS

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							SM	UNDOCUMENTED FILL	
				B-1 0.5'-2'				SM	@ 0-0.5': Silty SAND, dark reddish brown, damp to moist, very dense, moderate cementation	-200
									VERY OLD PARALIC DEPOSITS (Qvop8) @ 0.5': Silty SANDSTONE, dark reddish brown, damp to moist, very dense, moderate cementation	
385				R-1	50/6"	106	8			
				B-2 3'-6'						EI, CR
	5			R-2	50/4"		8			
380										
									@ 8': Becomes more clayey, brown to reddish brown	
	10			R-3 B-3 10'-13'	21 47 50/2					
								SC	@ 11': Clayey SANDSTONE, reddish brown, moist, very dense, weak to moderately cemented	DS
375										
	15			R-4	25 50/3"		13		Disturbed	
370										
	20			R-5	36 50/3"	120	9			
365									@ 22': Refusal on GRAVEL-COBBLE layer	
									Bottom of Boring at 22 Feet **No Groundwater or Seepage Encountered** **Backfilled with Soil Cuttings 8/6/2020**	
	25									
360										
	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. * * *

Logs from San Diego Geotechnical Consultants, 1988

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

GRAIN SIZES

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

200 40 10 4 3/4" 3" 12"

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

▽ GROUNDWATER LEVEL AT TIME OF DRILLING.

▼ GROUNDWATER LEVEL MEASURED LATER IN STANDPIPE.

◨ LOCATION OF SAMPLE TAKEN USING A STANDARD SPLIT TUBE SAMPLER, 2-INCH O.D., 1-3/8-INCH I.D. DRIVEN WITH A 140 POUND HAMMER FALLING 30-INCHES.

■ LOCATION OF SAMPLE TAKEN USING A MODIFIED CALIFORNIA SAMPLER, 3-1/8-INCH O.D., WITH 2-1/2-INCH I.D. LINER RINGS, DRIVEN USING THE WEIGHT OF KELLY BAR (LARGE DIAMETER BORINGS) OR USING A 140 POUND HAMMER FALLING 30-INCHES (SMALL DIAMETER BORING).

▯ LOCATION OF SAMPLE TAKEN USING A 3-INCH O.D. THIN-WALLED TUBE SAMPLER (SHELBY TUBE) HYDRAULICALLY PUSHED.

▩ LOCATION OF BULK SAMPLE TAKEN FROM AUGER CUTTINGS.

KEY TO LOGS - UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

JOB NO.: 05-6713-003-00-00

DATE: DECEMBER 1988

FIGURE: B-1

DATE OBSERVED: 11-21-88 **METHOD OF DRILLING:** 8" Hollow Stem Auger

LOGGED BY: MD **GROUND ELEVATION:** 387.0 **LOCATION:** See Site Plan

LOG OF BORING NO. 1

Sheet 1 of 1

DEPTH (FEET)	CLASSIF-ICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0				▨			A.C. - 3" with no base	Sieve, Atterberg Limits
		50/ 6"	■	▨			FILL: Dark red-brown slightly clayey, silty fine to medium SAND, moist, loose to medium dense	
5		50/ 3"	■				LINDAVISTA FORMATION: Light gray-brown silty fine to medium SANDSTONE, trace cobbles, damp, hard	
				▨			Becoming light orange to yellow-gray, mosit	
10		72/ 6"	■					
15								
				▨			Orange-brown medium to coarse SANDSTONE, moist, hard	
20		50/ 6"	■		11.2	111.3	Light yellow-gray, silty, fine to medium SANDSTONE, red-orange staining in-part, damp to moist, hard	
							Total Depth: 21' No Groundwater Backfilled 11-21-88	
25								
30								
35								

JOB NO.: 05-6713-003-00-00

San Diego Geotechnical Consultants, Inc.

FIGURE: B-2

DATE OBSERVED: 11-21-88 **METHOD OF DRILLING:** 8" Hollow Stem Auger

LOGGED BY: MD **GROUND ELEVATION:** 389.0 **LOCATION:** See Site Plan

LOG OF BORING NO. 5

Sheet 1 of 1

DEPTH (FEET)	CLASSIF-ICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0							AC to 4"	
							FILL: Dark brown silty fine to medium SAND, damp to moist, loose	
5							LINDAVISTA FORMATION: Light orange brown silty fine to medium SANDSTONE, damp, hard @ 5' becoming red-brown, moist	
10		50/ 6"	■		10.5	109.0	@ 10' light gray mottling in-part	
15							Brick red fine to medium SANDSTONE, poorly graded, moist, hard, trace cobbles	
20		50/ 3"	■		10.2	105.2	@ 20' gray mottling, trace cobbles	
							Total Depth: 21' No Groundwater Backfilled 11/21/88	
25								
30								
35								

JOB NO.:
05-6713-003-00-00

San Diego Geotechnical Consultants, Inc.

FIGURE: B-6

Logs from Shanon & Wilson, Inc., 2011

BORING 2

Date Drilled: March 5, 2011
Equipment Used: Hollow Stem Auger (7-inch diameter)
Depth to Water: Not Encountered
Driving Weight & Drop: 140 lbs/30 inches
Location : South East of loading dock

SURFACE ELEVATION: 385 (ft. MSL)

ELEVATION (feet)	DEPTH (feet)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	N VALUE (SPT)	BLOW COUNT (blows / 12-inches)	SAMPLE LOCATION
	5			57		
		12.8	109		45	
	10			34		
	15	15.6	112		69/9"	
				20/0"		
	20					
	25					
	30					
	35					
	40					

4" Asphalt Paving; 2" Base Course

FILL
SM - SILTY SAND - fine, some Clay, few Gravel, very moist, reddish brown

LINDAVISTA FORMATION (Qln)
SC - CLAYEY SAND - fine, moist, reddish brown

SM - SILTY SAND - fine, trace Clay, moist, red brown

no Clay

[Hammer bouncing 3" into sample zone / no additional penetration]
[Rock at sampler tip]

[Hard drilling]
[Rock at sampler tip]

END OF BORING AT 18.5 FEET

Notes:

1. Hand auger used to depth of 2.5 feet.
2. Fill encountered to a depth of 2.5 feet.
3. Groundwater not encountered.
4. Boring backfilled with soil cuttings and pavement patched with asphalt.

Job No: 51-1-09001-003 By: SCG Date: 3-7-2011 Checked: Printed: 3-29-11 [LOG FOR FIELD; 51-1-09001-003.GPJ]

The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated.
It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIGURE A-1.2

Geophysical Evaluation, Atlas Technical Consultants, 2020

6280 Riverdale Street
San Diego, CA 92120
(877) 215-4321 | oneatlas.com

August 28, 2020

Atlas No. 120378SWG
Report No. 1

MR. BOB STROH, P.G., CEG
LEIGHTON
3934 MURPHY CANYON ROAD
SAN DIEGO, CALIFORNIA 92123

Subject: **Geophysical Evaluation**
Sharp Healthcare SMH
San Diego, California

Dear Mr. Stroh:

In accordance with your authorization, Atlas Technical Consultants has performed a geophysical evaluation pertaining to the Sharp Healthcare SMH project located at 7901 Frost Street in San Diego, California (Figure 1). The purpose of our study was to develop a Shear-wave velocity profile to be used for design and construction at the study site. This letter report presents our methodology, equipment used, analysis, and findings. Our services were conducted August 26, 2020.

Our scope of services for the project included performance of one refraction microtremor (ReMi) profile (RL-1) at a preselected area of the project site (Figure 2). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a Shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one-dimensional sounding which represents the average condition across the length of the line. The ReMi method does not require an increase of material velocity with depth; therefore, low velocity zones (velocity inversions) are detectable with ReMi.

Our ReMi evaluation included the use of a 24-channel Geometrics Geode seismograph and 24, 4.5-Hz vertical component geophones. For RL-1, geophones were spaced 9 feet apart for a total line length of 207 feet. Fifteen records, each 32 seconds long, were recorded and then downloaded to a computer. The data was later processed using Surface Plus 9.1 – Advanced Surface Wave Processing Software (Geogiga Technology Corp., 2020), which uses the refraction microtremor method (Louie, 2001) and other surface wave analysis methods. The program generates phase velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determine the best fitting model. The result is a one-dimensional shear-wave velocity model of the site with roughly 85 to 95 percent accuracy.

Figure 3 presents the result for RL-1 from our evaluation. Based on our analysis of the collected data for RL-1, the average characteristic site Shear-wave velocity down to a depth of 100 feet is 2,055 feet per second (ft/s) (CBC, 2019). These values correspond to site classifications of **C**. It should be noted the ReMi results represent the average condition across the length of the line.

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration Additional subsurface evaluating will be performed upon request.

This document is intended to be used only in its entirety. No portions of the document, by itself, is designed to completely represent any aspect of the project described herein. Atlas should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use of or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have questions related to this report, please call us at (858) 527-0849.

Respectfully submitted,

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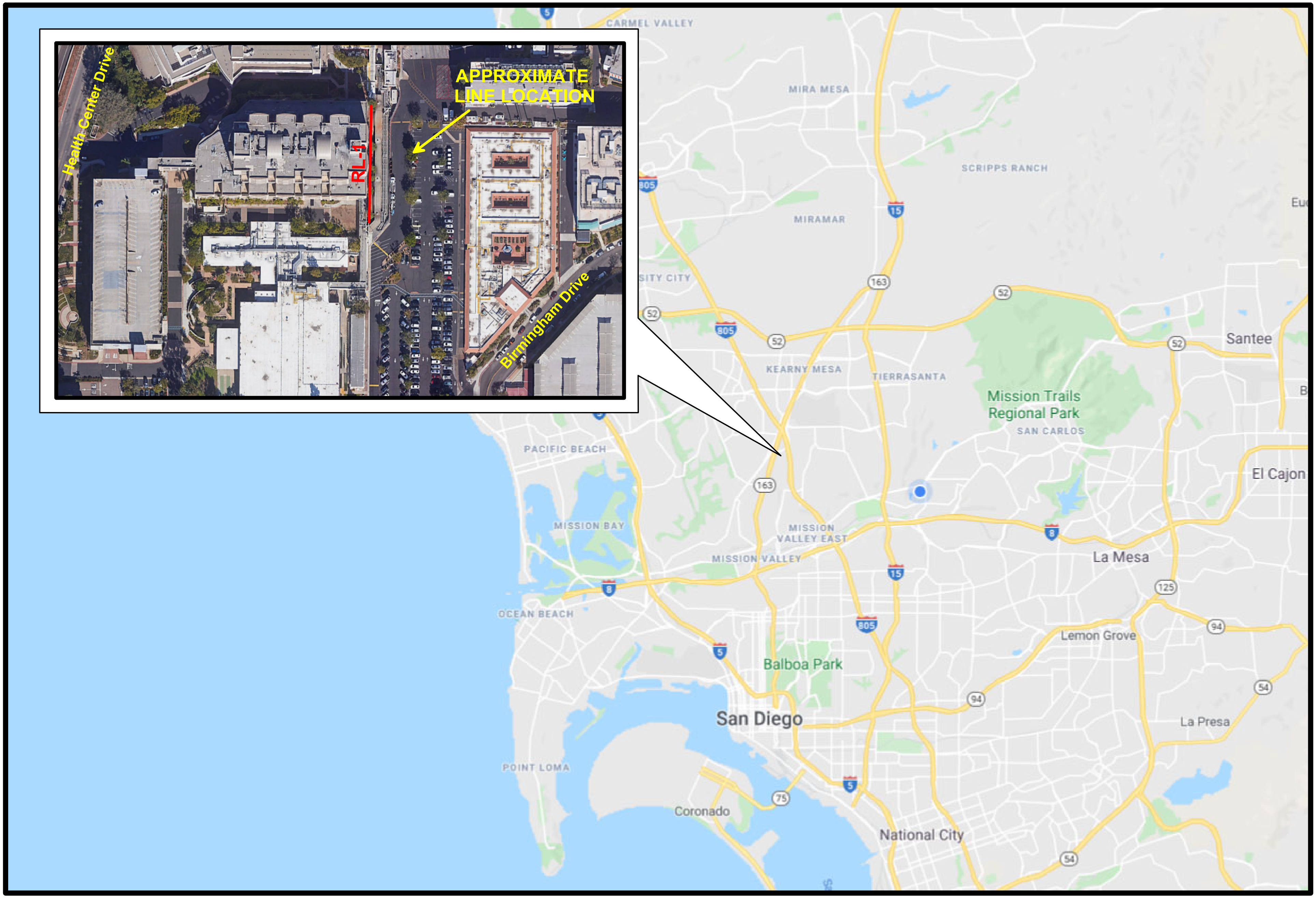
ECA:PFL:pfl:ds

Attachments: Figure 1 – Site Location Map
Figure 2 – Seismic Line Location Map
Figure 3 – ReMi Results (RL-1)

Distribution: Bob Stroh at BStroh@leightongroup.com



SITE LOCATION MAP

N

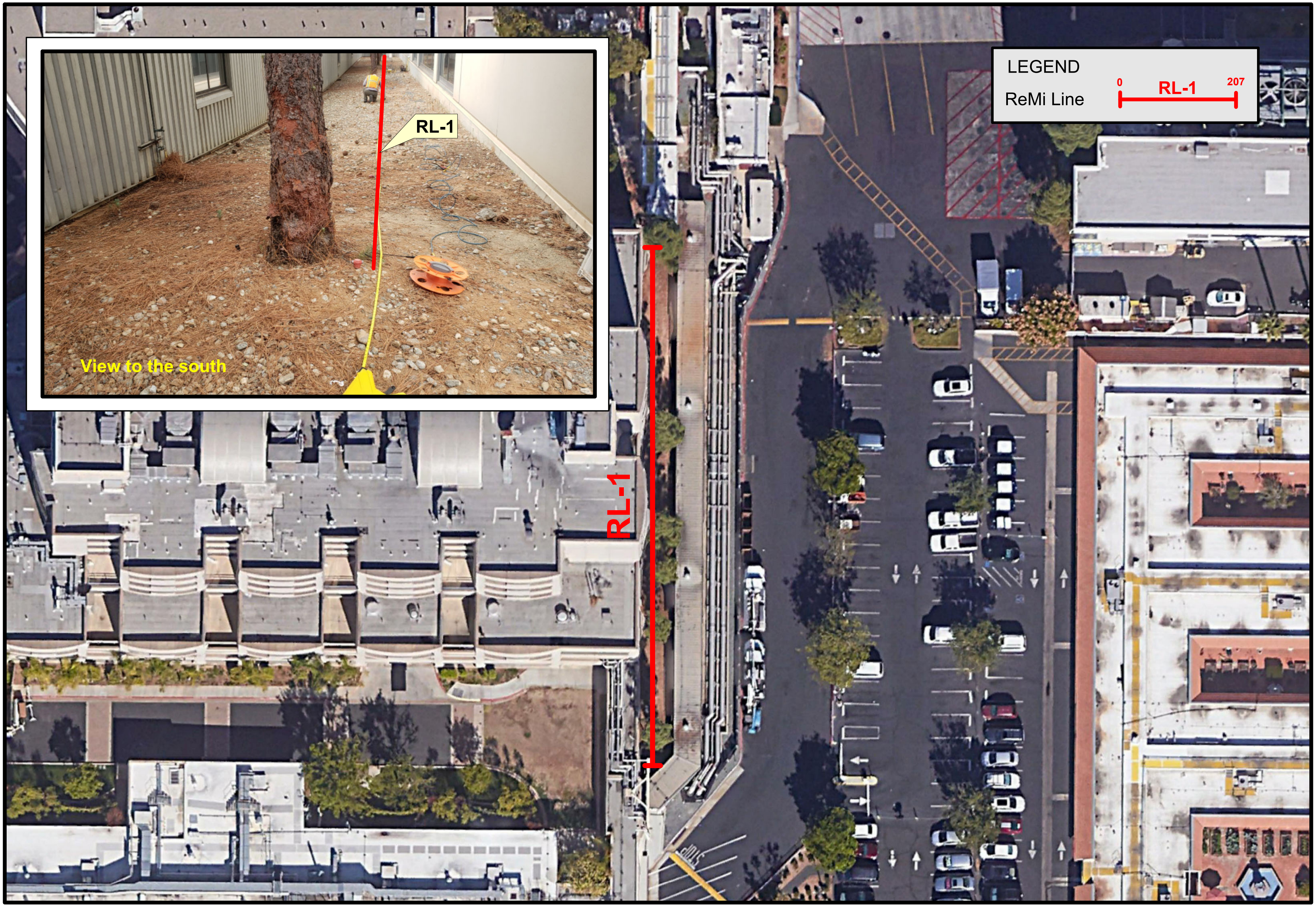
Sharp Healthcare SMH
San Diego, California

Project No.: 120378SWG

Date: 08/20

ATLAS

Figure 1



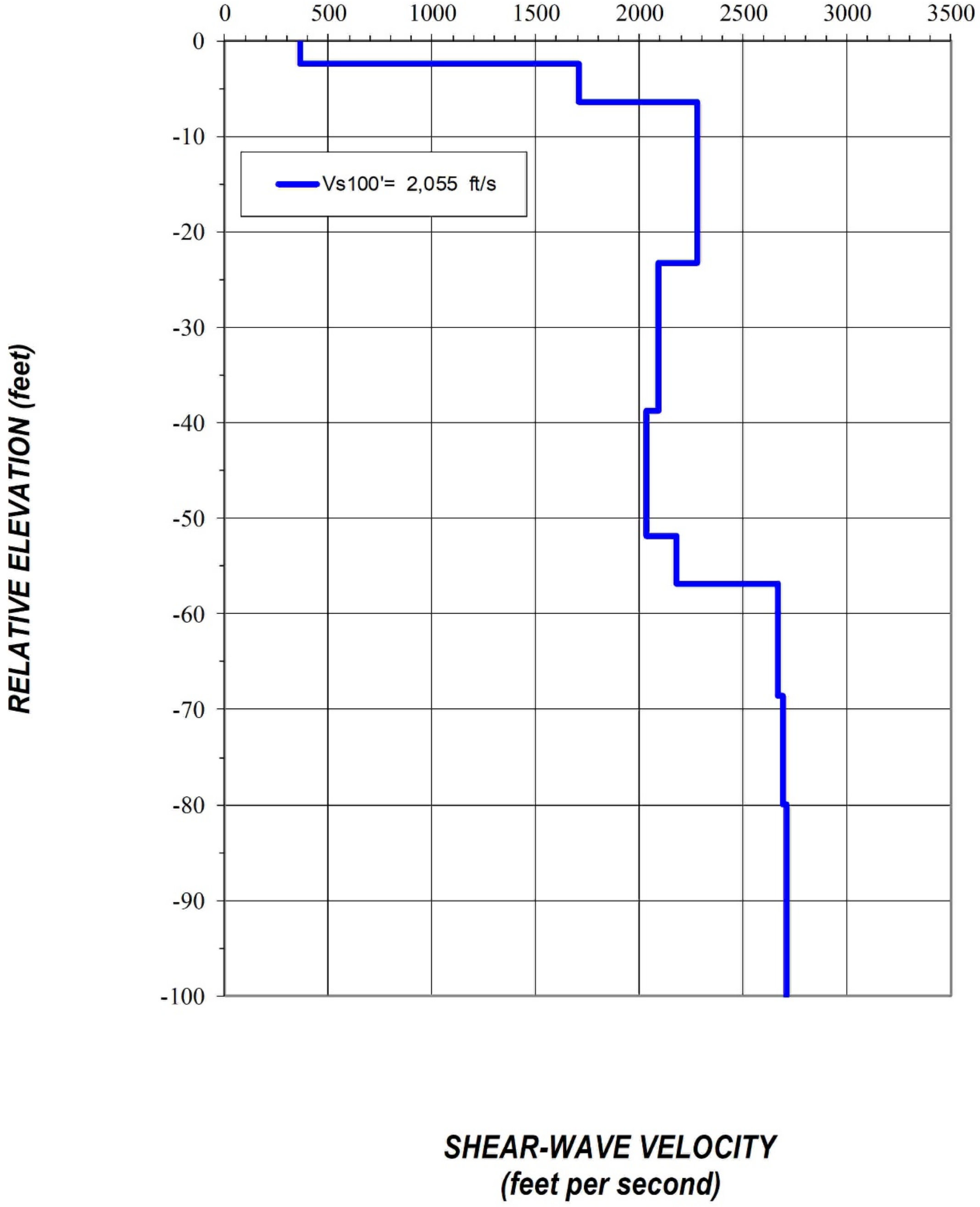
SEISMIC LINE LOCATION MAP

Sharp Healthcare SMH
San Diego, California

Project No.: 120378SWG
Date: 08/20

ATLAS
Figure 2

Vs Model



ReMi RESULTS **RL-1**	Sharp Healthcare SMH San Diego, California	ATLAS Figure 3
	Project No.: 120378SWG Date: 08/20	

SASW Measurements, Geovision, 2001

SURFACE WAVE (SASW)
MEASUREMENTS

Conducted at the

Sharp Memorial Hospital
7901 Frost Street
San Diego, California

Prepared for

Shannon & Wilson, Inc.
400 N. 34th Street, Suite 100
Seattle, Washington 98103

Prepared by

GEOVision Geophysical Services
a Division of Blackhawk GeoServices
1151 Pomona Road, Unit P
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Report 1351-01
June 29, 2001

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INTRODUCTION

In-situ seismic measurements using the Spectral Analysis of Surface Waves (SASW) method were performed in a parking lot immediately north of the Sharp Memorial Hospital, 7901 Frost Street, San Diego, California on June 13, 2001. The purpose of this investigation was to provide a shear wave velocity profile at the site to a depth of 30 meters, to be used for UBC site classification. Subsurface geologic conditions of the site were expected to consist of several feet of fill soils overlying the Lindavista Formation.

This report contains the results of the SASW measurements conducted by Antony Martin and Chuck Carter of **GEO***Vision*. Analysis of the surface wave dispersion data to determine the corresponding shear wave velocity profiles was performed by Antony Martin. An overview of the SASW method is given, followed by the procedures used in this investigation. The shear wave velocity profiles obtained from the SASW data are presented in graphic and tabular form. A brief discussion of the results follows. The SASW method is described in detail in Appendix A.

OVERVIEW OF THE SASW METHOD

Spectral analysis of surface waves (SASW) testing is an in-situ seismic method for determining shear wave velocity (V_S) profiles [Stokoe et al., 1994; Stokoe et al., 1989]. It is non-invasive and non-destructive, with all testing performed on the ground surface at strain levels in the soil in the elastic range (< 0.001%).

The basis of the SASW method is the dispersive characteristic of Rayleigh waves when propagating in a layered medium. The phase velocity, V_R, depends primarily on the material properties (V_S, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. Waves of different wavelengths, λ, (or frequencies, f) sample different depths. As a result of the variance in the shear stiffness of the layers, waves with different wavelengths travel at different phase velocities; hence, dispersion. A surface wave dispersion curve, or dispersion curve for short, is the variation of V_R with λ or f. SASW testing consists of collecting surface wave phase data in the field, generating the dispersion curve, and then using iterative modeling to back-calculate the shear stiffness profile.

A detailed description of the SASW field procedure is given in Joh [1997]. A vertical dynamic load is used to generate horizontally-propagating Rayleigh waves (Figure 1). The ground motions are monitored by two vertical receivers and recorded by the data acquisition system capable of performing both time and frequency-domain calculations. Theoretical as well as practical considerations, such as attenuation, necessitates the use of several receiver spacings to generate the dispersion curve over the wavelength range required to evaluate the stiffness profile. To minimize phase shifts due to differences in receiver coupling and subsurface variability, the source location is reversed.

After the time-domain motions from the two receivers are converted to frequency-domain records using the Fast Fourier Transform, the cross power spectrum and coherence are calculated. The phase of the cross power spectrum, ϕ_w (f), represents the phase differences between the two receivers as the wavetrain propagates past them. It ranges from $-\pi$ to π in a wrapped form and must be unwrapped through an interactive process called masking. Phase jumps are specified, near-field data (wavelengths longer than three times the distance from the source to first receiver), and low-coherence data are removed. The experimental dispersion curve is calculated from the unwrapped phase angle and the distance between receivers by:

$$V_R = f * d_2/(\Delta\phi/360°),$$

where V_R is Rayleigh wave phase velocity, f is frequency, d_2 is the distance between receivers, and $\Delta\phi$ is the phase difference in degrees.

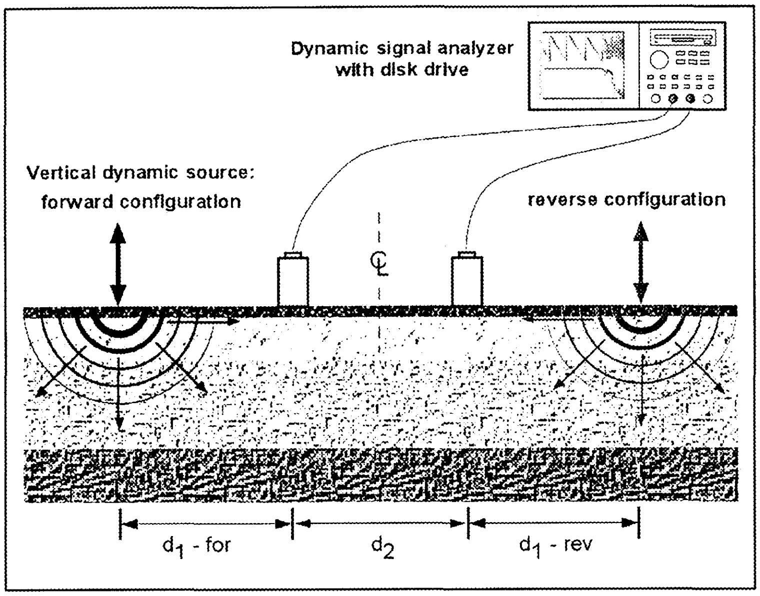


Figure 1 Basic Configuration of SASW Measurements [Modified from Joh, 1997].

WinSASW, a program developed at the University of Texas at Austin, is used to reduce and interpret the dispersion curve. Through iterative forward modeling, a V_S profile is found whose theoretical dispersion curve is a close fit to the field data.

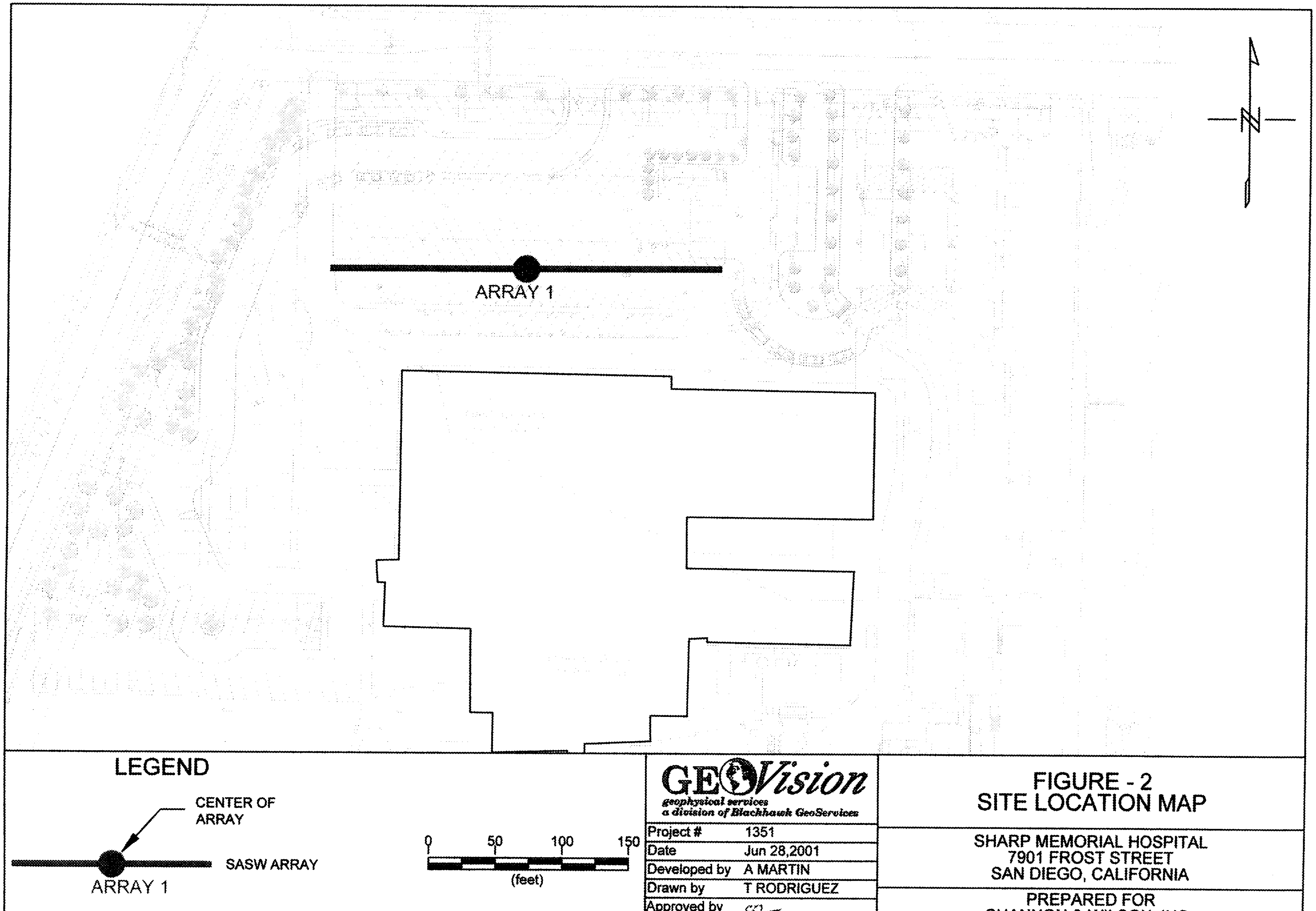
The final model profile is assumed to represent actual site conditions. Several options exist for forward modeling: a formulation that takes into account only fundamental-mode Rayleigh wave motion (called the 2-D solution), and those that include all stress waves and incorporate receiver geometry (3-D solution) [Roesset et al., 1991].

PROCEDURES

SASW data were collected along one array (Array 1) as shown in Figure 2. The general location of the array was selected by Shannon & wilson, Inc. Although SASW data were collected in the evening, the parking lot was in continual use and some noise from vehicular traffic and nearby utility lines was observed.

The data were collected with receiver spacings of 0.2, 0.4, 2, 4, 8, 16, and 30 m, with a common centerline. This provided overlap of data from different receiver spacings. Generally, the high frequency (short wavelength) surface waves were measured across the short spacings and the low frequency (long wavelength) surface waves were measured with the large receiver spacings.

The 0.2 and 0.4 m receiver spacings were used in an attempt to image the thin asphalt layer at the site. For receiver spacings up to 16 m, small hammers, rock hammers, 10-lb, and 20-lb sledgehammers were used as seismic sources (Figure 3). Data from the transient impacts were averaged 10 to 20 times to improve the signal-to-noise ratio. An electromechanical shaker was used for the 16 and 30 m spacing. Surface waves were monitored by two 1-Hz Kinemetrics Ranger Model SS-1 geophones (2 to 30 m receiver spacings) or two Oyo Geospace 100-Hz geophones (0.2 and 0.4 m spacing), and recorded by an HP 35670A dynamic signal analyzer. WinSASW was used to average forward- and reverse-direction data, to mask phase data and to generate the dispersion curve.



NOTE: SITE MAP PROVIDED BY SHANNON & WILSON, INC.

GEOVision
geophysical services
a division of Blackhawk GeoServices

Project #	1351
Date	Jun 28,2001
Developed by	A MARTIN
Drawn by	T RODRIGUEZ
Approved by	
File	C:\AcadMap2K\1351\1351-2.dwg

FIGURE - 2
SITE LOCATION MAP

SHARP MEMORIAL HOSPITAL
7901 FROST STREET
SAN DIEGO, CALIFORNIA

PREPARED FOR
SHANNON & WILSON, INC.

The 2-D model was used for the SASW modeling. This model calculates the fundamental-mode Rayleigh wave dispersion and provides satisfactory results at sites with gradual increases in V_S with depth.

Constant mass density values of 1.9 and 2.1 g/cc were used in the profiles for fill soils and Lindavista Formation, respectively. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible effect on dispersion. Compression wave velocity, V_P, was calculated from the assumed value of Poisson's ratio, ν, of 0.33, from the relationship:

$$V_P = V_S\,[(2(1-\nu))/(1-2\nu)]^{0.5}$$

Figure 3 SASW Testing with Various Hammers as the Seismic Source.

RESULTS

The fit of the theoretical dispersion curve to the experimental data collected along Array 1 is shown in Figure 4. The V_S profile for Array 1 is shown graphically in Figure 5. The resolution decreases gradually with depth, because of loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The V_S and V_P profile used to match the field data is provided in tabular form as Table 1. The depth to which these profiles are valid is about 30 m. The V_S depth profile shows 1.4 m (4.5 ft) of fill soil with a velocity of 155 to 170 m/s (509 to 558 ft/s) overlying Lindavista Formation with velocity increasing with depth from 425 m/s (1394 ft/s) to 850 m/s (2789 ft/s).

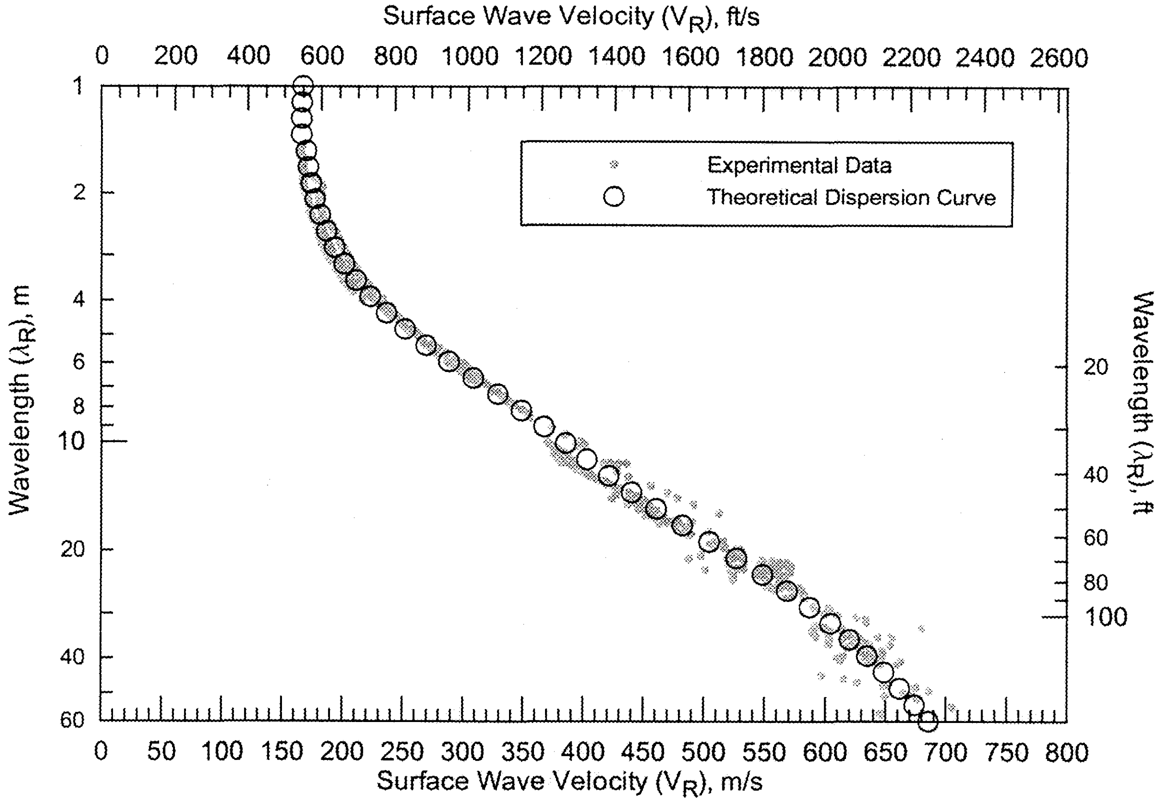


Figure 4 **Comparison of Field Experimental Data and Theoretical Dispersion Curve from SASW Testing along Array 1**

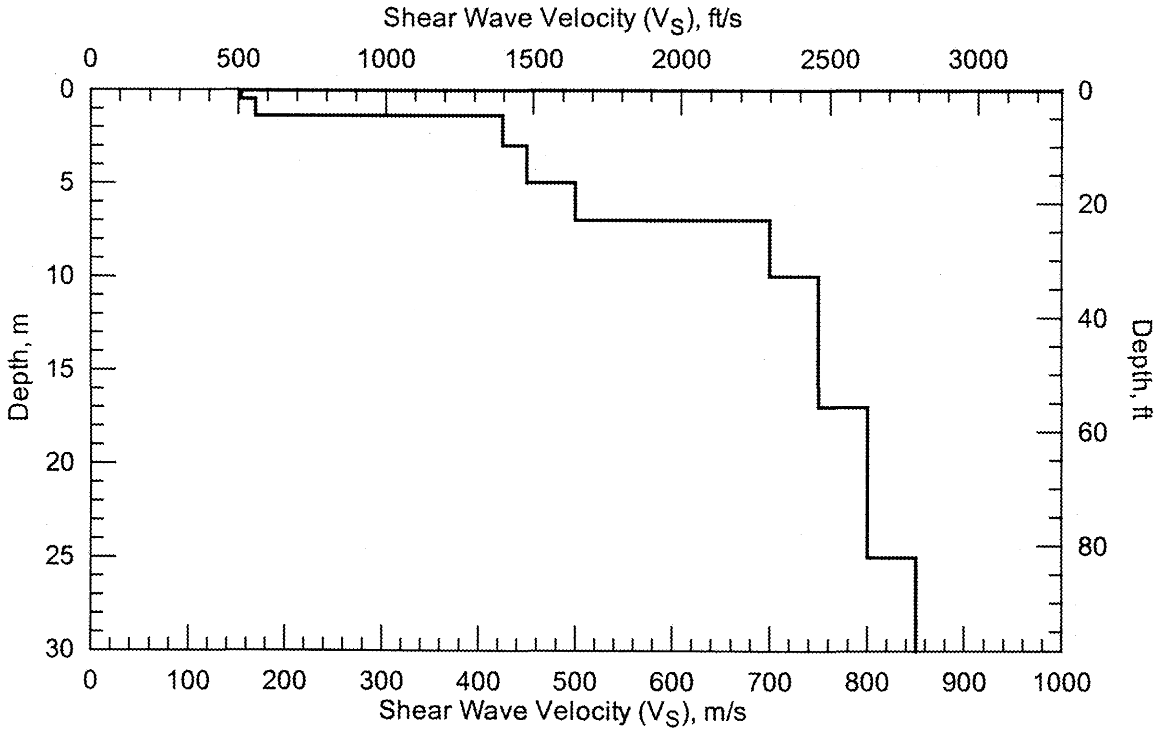


Figure 5 **V_S Profile from SASW Testing along Array 1**

Table 1 V_S Profile Used in the SASW Model for Array 1

Depth to Top of Layer		Layer Thickness		S-Wave Velocity		P-Wave Velocity	
m	ft	m	ft	m/s	ft/s	m/s	ft/s
0	0.0	0.05	0.2	1000	3281	2000	6561
0.05	0.2	0.45	1.5	155	509	310	1017
0.5	1.6	0.9	3.0	170	558	340	1115
1.4	4.6	1.6	5.2	425	1394	850	2788
3	9.8	2	6.6	450	1476	900	2952
5	16.4	2	6.6	500	1640	1000	3281
7	23.0	3	9.8	700	2297	1400	4593
10	32.8	7	23.0	750	2461	1500	4921
17	55.8	8	26.2	800	2625	1600	5249
25	82.0	10	32.8	850	2789	1700	5577

Note: P-wave velocity calculated assuming Poisson's ratio = 0.33.

DISCUSSION

The surface wave dispersion data from the site have some variability at small wavelengths (Figure 4). This is primarily caused by lateral heterogeneity in shallow soils at the site. The velocities of the small-wavelength surface waves are measured across short distances, whereas the velocities of the longer wavelength surface waves are measured over greater distances. The dispersion data averaged across longer distances are smoother as the affects of localized heterogeneities are averaged. Some of the variability in the surface wave dispersion data may be caused by noise resulting from vehicular traffic, utilities and various other sources.

The theoretical model used to interpret the dispersion assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of SASW testing provide a good "global" estimate of the material properties along the array. The results may be more representative of the site than a borehole "point" estimate.

Based on the our experience at other sites, the shear wave velocity models determined by SASW testing are within 20% of the velocities that would be determined by other seismic methods [Brown, 1998]. The average velocities, however, are much more accurate than this, often to better than 10%, because they are much less sensitive to the layering in the model.

Average shear wave velocities to a depth of 30 m, V_s30, is 597 m/s (1959 ft/s) for Array 1. The high velocity asphalt layer was not used in the V_s30 calculation. According to the 1997 Uniform Building Code, the site is classified as C, very dense soil and soft rock (BSSC, 1994).

CONCLUSIONS

Spectral Analysis of Surface Waves (SASW) testing was performed at the Sharp Memorial Hospital, San Diego, California. The shear wave velocity profile for Array 1 a determined by this method is presented in this report as Figure 5 and Table 1. V_s30 is approximately 597 m/s (1959 ft/s) for the array. Therefore, according to the 1997 Uniform Building Code, the site is classified as C, very dense soil and soft rock (BSSC, 1994).

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Brown, L.T., 1998, "Comparison of V_S profiles from SASW and borehole measurements at strong motion sites in Southern California", Master's thesis, University of Texas at Austin.

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Stokoe, K.H.,II, Rix, G.L. and S. Nazarian, 1989, "In situ seismic testing with surface waves" *Proceedings, Twelfth International Conference on Soil Mechanics and Foundation Engineering, Vol. 1,* Rio de Janeiro, Brazil, pp. 330-334.

APPENDIX A

Excerpt from:

Brown, L.T., 1998, "Comparison of V_S profiles from SASW and borehole measurements at strong motion sites in Southern California", Master's thesis, University of Texas at Austin.

Modified from "Brown, L.T., 1998, Comparison of V_S Profiles from SASW and Borehole Measurements at Strong Motion Sites in Southern California, M.S. Thesis, University of Texas at Austin."

OVERVIEW OF SASW METHOD

2.1 INTRODUCTION

Spectral-analysis-of -surface-waves testing, known as SASW testing, is an in-situ seismic method for determining shear wave velocity profiles. It is non-invasive and non-destructive; the test is performed on the ground surface and strain levels in the soil are in the elastic range (< 0.001%). From the modeled shear wave velocity (V_S) profile, a small-strain shear modulus, G_{max}, profile can be determined using an estimated material density, ρ, as:

$$G_{max} = \rho * V_S^2. \tag{2.1}$$

SASW has been used for a variety of engineering applications requiring shear stiffness data, including earthquake site response, liquefaction susceptibility analysis, soil compaction control, and pavement testing (Brown et al., 1999; Andrus, 1994; Stokoe and Rix, 1987; Rix and Stokoe, 1989).

The basis of the SASW method is the dispersive characteristic of Rayleigh waves when propagating in a layered system. The phase velocity, V_R, depends primarily on the material properties (shear wave velocity, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. Waves of different wavelengths, λ, (or frequencies, f) sample different depths as illustrated in Fig. 2.1. As a result of the shear stiffnesses of the layers varying, different wavelength waves travel at different phase velocities. A surface wave dispersion curve, or dispersion curve for short, is the variation of V_R with λ or f.

SASW testing consists of collecting surface wave phase data in the field, generating the dispersion curve, and then using iterative modeling to back-calculate the material properties with depth. In this chapter, the development of the SASW method is reviewed. The SASW field procedure is then outlined, including the equipment and experimental setup used in this research. Data reduction and interpretation methods are discussed, with an emphasis on the techniques used to evaluate a shear wave velocity profile from an experimental dispersion curve.

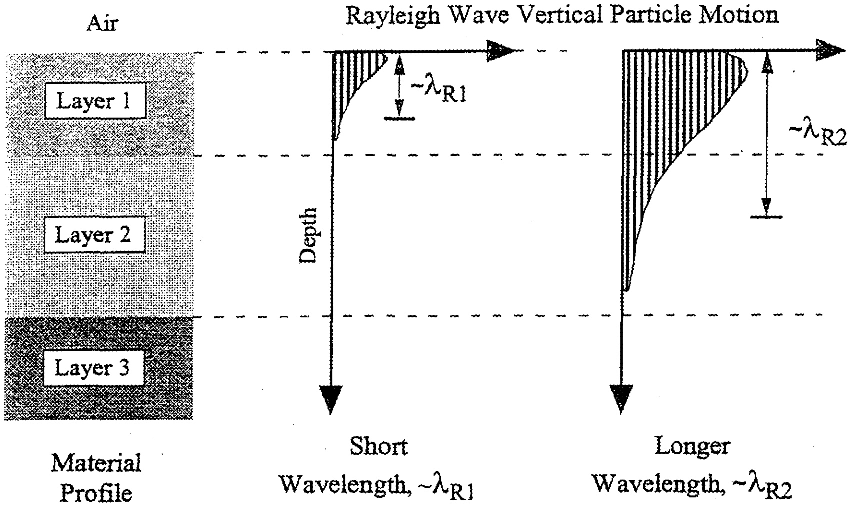


Figure 2.1 The Theoretical Basis of SASW Testing is that Rayleigh Waves of Different Wavelengths Penetrate to Different Depths and Sample Different Material.

2.2.1 RAYLEIGH WAVES

Theoretically, a vertical impact on a half-space generates both body waves and surface waves, with 67% of the impact energy imparted to the Rayleigh waves, 26% to shear waves, and 7% to compression waves (Miller and Pursey, 1955). Rayleigh waves propagate radially outward from the source in a cylindrical wavefront. In contrast, body waves propagate along a hemispherical wavefront (Fig. 2.2). Rayleigh waves produce both vertical and horizontal motion, with the overall motion being a retrograde ellipse at the surface. The variation of particle motion with depth is shown in Fig 2.3.

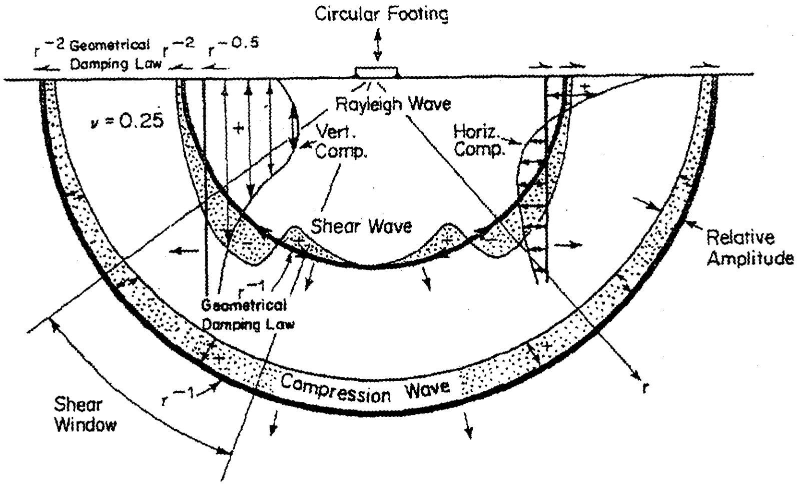


Figure 2.2 Distribution of Displacement Waves from a Circular Footing on a Homogeneous, Isotropic, Elastic Half-space (from Woods, 1968).

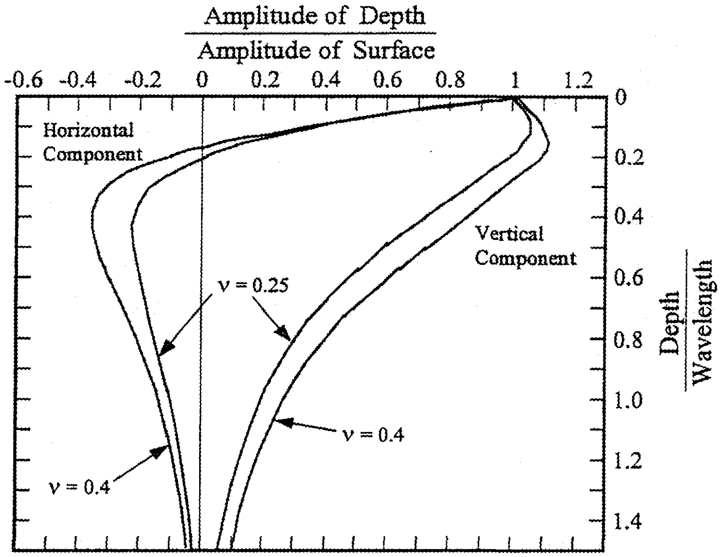


Figure 2.3 Variation of Rayleigh Wave Particle Motion with Depth, for Different Values of Poisson's Ratio, ν (Modified from Woods, 1968).

2.2.2 DEVELOPMENT OF SASW METHOD

The SASW method originated from the steady-state Rayleigh wave method of the 1950's - '60's (Richart et al., 1970). In this early method, the Rayleigh-wave phase velocities are measured using receivers pairs at in-phase points of a steady-state wavefield. The receiver pairs must be moved for each wavelength measured. Dispersion data are interpreted by an empirical method.

The introduction of digital signal analyzers, simplified test procedures, more accurate theoretical models, and more efficient computing has led to the development of the modern SASW method.

2.3 SASW FIELD MEASUREMENTS

A considerable amount of research has been conducted to develop the theoretical basis and practical applications of the modern SASW method (Nazarian 1984, Sanchez-Salinero 1987, Sheu 1987, Rix 1988, Roesset et al. 1990, and Joh 1997). This work includes the development of a practical and theoretically sound field procedure.

2.3.1 Purpose

The purpose of SASW field work is to measure the data needed to generate the surface wave dispersion curve for the range of wavelengths (or frequencies) needed to back out the material properties to the desired depth. The necessary data consist of surface wave phase differences between pairs of geophones. The general test setup is shown in Figure 2.4.

The source is used to generate surface waves which propagate towards the first and second receiver. The receivers transform the ground motion into electrical signals. As the surface waves pass by the monitoring receivers, the motion between the two receivers will generally be out of phase. The phase difference between the receivers is calculated from the receiver motions recorded by the data acquisition system, as discussed below.

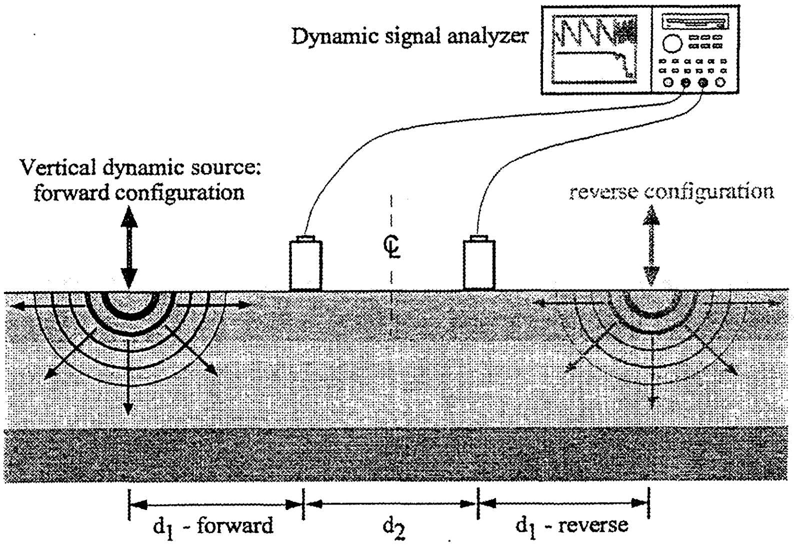


Figure 2.4 Basic configuration of SASW measurements.

2.3.2 Equipment

2.3.2.1 Source

A variety of mechanical systems can be used as surface wave sources. They must be capable of generating vertical dynamic loads on the ground surface. There are two general types of sources: transient impact and continuos sources. The source used depends on the desired profiling depth and site restrictions. Heavier sources are used to generate lower frequency surface waves that penetrate deeper into the ground.

Dropped weights, sledge hammers, and small hand-held hammers are common transient sources. The frequencies generated depend on the material and weight of the source and the stiffness of the site. Hammers are rugged, portable, and have few restrictions in their use on site. Frequencies generated by the 16-lb. (7.3 kg) sledge hammer are approximately 15 to 150 Hz. The geology hammer (pick) generates surface waves in the frequency range of 30 to 300 Hz. Ten to twenty strikes are averaged together in the frequency domain to obtain a higher signal-to-noise ratio.

In the continuous source category, a portable electromagnetic shaker, eccentric mass oscillator, bulldozer, or a vibroseis truck is commonly used. A sweep of frequencies (swept-sine) or random noise may be used as the source function for an electromagnetic shaker or vibroseis truck. The advantage of the swept-sine function is that the energy is concentrated at individual frequencies in succession, resulting in a higher signal-to-noise ratio. Shakers are available with significant output to frequencies as low as 5 Hz. A 50,000-lb vibroseis truck typically generates frequencies down to around 2 to 3 Hz.

Bulldozers or heavy equipment are used to generate continuous random vibrational energy. Because the signal is relatively weak, data must be averaged over a long time (15 to 60 minutes). Depending on the mass, heavy equipment is capable of generating surface waves with frequencies of 1 to 2 Hz.

2.3.2.2 Receivers

Receivers convert particle ground motion into a voltage signal that is recorded by the data acquisition system. Although surface waves produce both vertical and horizontal particle motions, only vertical particle motions are recorded in these SASW measurements. Time-motion records of vertical particle motions are converted to frequency-domain records for later use in calculating the dispersion curve.

The receivers are required to have significant output over the relevant frequency range (1-400 Hz). The receivers are calibrated in the laboratory and are combined in two-receiver sets which possess negligible differences in phase shift between the two receivers. Typically, 70% critically damped 1-Hz and/or 4.5 Hz vertical geophones are used for SASW testing of soils.

2.3.2.3 Data Acquisition System

Several electronic devices can be used to record and process the receiver signals, including dedicated waveform analyzers and microcomputer based systems (Gucunski and Woods, 1991). It is recommended that the recording device have a dynamic range of at least 100 dB with a full-scale sensitivity of 3 mV, have anti-aliasing filters, have two or more recording channels, and be capable of performing spectral calculations in real time in the field (Stokoe et al., 1994).

A dynamic signal analyzer can be used both as a source function generator and a recording device. A dynamic signal analyzer is a digital oscilloscope with a built in microprocessor that allows it to make calculations in both the frequency and time domains. Several sets of spectral calculations are made in the field to monitor the progress of the SASW experiment. The cross power spectrum and coherence are the most important for analyzing the SASW data.

With the dynamic signal analyzer, the time-domain records from the two geophones, x(t) and y(t), are transformed into frequency-domain records, X (f) and Y (f), respectively, using the Fast Fourier Transform algorithm , F:

$$X(f) = F(x(t)) \tag{2.2}$$

$$Y(f) = F(y(t)). \tag{2.3}$$

The auto power spectra, G_{XX} (f) and G_{YY} (f), are calculated by:

$$G_{XX}(f) = X^{*}(f)\, X(f) \tag{2.4}$$

$$G_{YY}(f) = Y^{*}(f)\, Y(f), \tag{2.5}$$

where * represents the complex conjugate. To reduce the random noise level and incoherent signals, a technique called coherent signal averaging (Model 3562A Operating Manual, 1985) is used in data acquisition. This involves collecting several wavetrains, usually 3 to 5, and averaging the spectra in the frequency domain. The auto power spectra are representative of the source characteristics, site behavior, and receiver response.

The cross power spectrum represents the difference in the wave trains at the two geophones. From the phase of the cross power spectrum, the propagational velocity of different frequency components of the wave train can be calculated. The cross power spectrum, G_{XY} (f), is derived from the averaged frequency-domain records:

$$G_{YX}(f) = Y(f)\, X^{*}(f). \tag{2.6}$$

The wrapped phase angle, ϕ_w (f), ranges from $-\pi$ to π and represents the phase differences at the two receivers as the wavetrain propagates past them. The wrapped phase angle must be unwrapped, as described in Section 2.4.1 on masking, to obtain the true phase angle. The wrapped phase angle at a certain frequency is calculated from that frequency component of the cross power spectrum:

$$\phi_w(f) = \tan^{-1}(\text{im}\, G_{YX}(f) / \text{re}\, G_{YX}(f)), \tag{2.7}$$

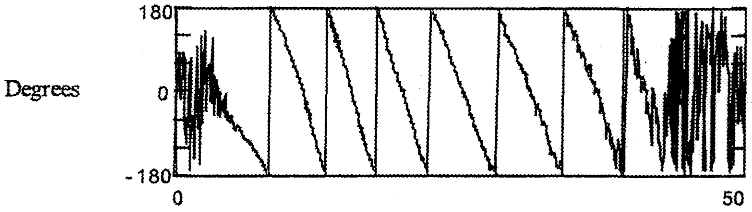
Where "im" represents the imaginary part and "re" the real part of a complex number. Coherent signal averaging also allows the calculation of the coherence function, γ^2 (f), an indicator of signal quality. The coherence function is calculated by:

$$\gamma^2(f) = (\text{abs}(G_{YX}(f))^2 / G_{XX}(f)\, G_{YY}(f), \tag{2.8}$$

where "abs" represents the absolute value of the quantity.

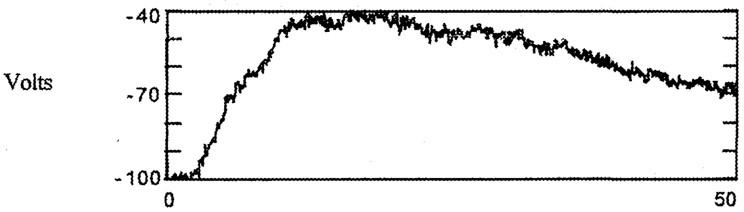
Both time and frequency domain records are calculated in real time in the field so that the experiment can be modified as needed based upon the operator observing the wrapped phase angles and coherence. The data are saved on the attached disk drive. A complete set of frequency domain records is shown in Fig. 2.7.

a. Phase of the Cross Power Spectrum



b. Coherence Function

c. Receiver 1 Auto Power Spectrum



d. Receiver 2 Auto Power Spectrum

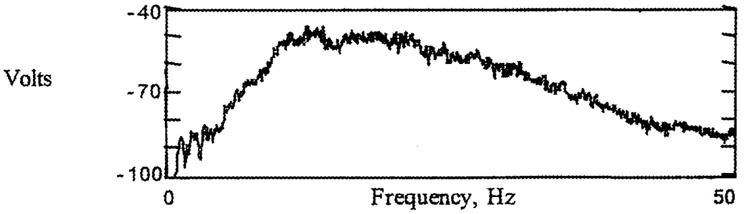


Figure 2.7 Complete Set of Frequency-Domain Records Generated from the SASW Data Acquisition System (from Andrus, 1994).

2.3.3 Experimental Setup

The source-receiver geometry in the SASW testing setup is shown in Figure 2.4. The source and receivers are located along a linear array, with the distance from the source to the first receiver equal to d_1, and the distance between receivers equal to d_2. Theoretical studies (after practical field testing) have shown that the most favorable dispersion curve is generally obtained when the distance to the first receiver is around one to two wavelengths and the distance between receivers is equal to the distance from the source to the first receiver (Sanchez-Salinero 1987, Roesset et al. 1990). To minimize phase shifts due to differences in receiver coupling, the location of the source is reversed. This also helps average out the effects of lateral variability and dipping soil layers.

A wide range of Rayleigh wave wavelengths are needed to evaluate the stiffness profile from SASW testing, typically 1 to 800 ft (0.3 to 250 m) for a shear wave velocity profile depth of 300 ft (90 m). The theoretical considerations previously mentioned as well as attenuation and near-field effects necessitate the use of several receiver spacings to obtain the dispersion curve. By using many receiver spacings, considerable overlap in the frequency range from the individual data sets is produced and a smoother, more representative dispersion curve is obtained. A complete set of receiver spacings is called an array.

Commonly, the source and receiver spacings are increased, keeping a common midpoint. Or, if source mobility is limited by site restrictions or time constraints, the source location may be constant and the receivers moved increasingly further away. The two setups are shown in Fig. 2.8.
The " common receivers midpoint" geometry usually produces the best data, because each receiver spacing setup samples some of the same near-surface material. With a " constant source location" geometry, lateral variability may make the resulting dispersion curve difficult to interpret, but a lot of time is saved by not repositioning the source. The receiver geometry of both setups is such that the measured dispersion curve and resulting stiffness profile is representative of a spatial average of the material properties at the site.

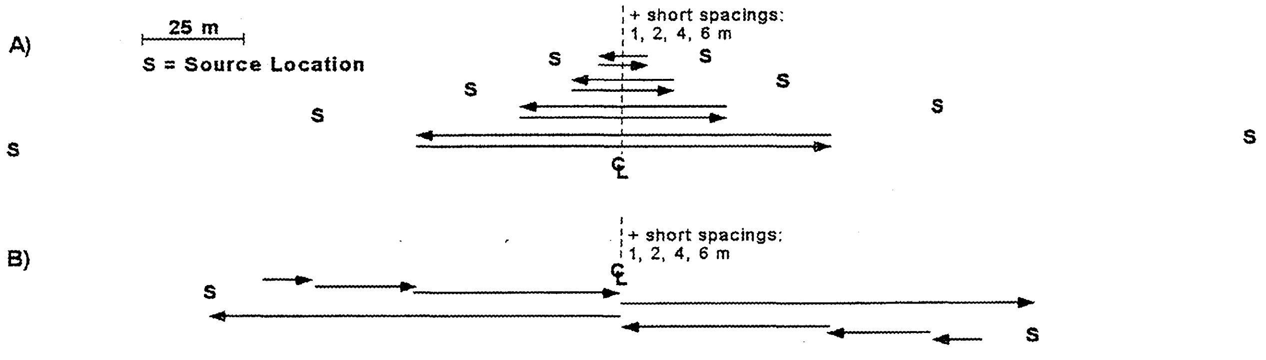


Figure 2.8 Typical Source-Receiver Geometry for 100-m V_S profile. A) Common Receivers Midpoint Array B) Common Source Location Array. Arrow Represents Direction of Surface Wave Propagation as Recorded by Receivers at Both Ends of Arrow, Lines are Offset for Illustrative Purposes Only, Actual Receiver Pairs in Arrays Would Be In-line.

To avoid near-field effects associated with surface waves and body waves, the distance from the source to the first receiver, d_1, is at least half of the maximum desired wavelength (0.5 cycles). The interreceiver distance, d_2, is typically 4-6 times the minimum wavelength. Once the desired wavelength range is determined, the appropriate receiver spacing and source frequency range must be determined, based on initial estimates of the site stiffness. For example, if the Rayleigh wave velocity of a half-space were 500 ft/s (152 m/s), to generate wavelengths between 2 and 20 ft (0.6 to 6.1 m), a frequency range of 25 to 250 Hz would be required. This relationship is calculated by:

$$V_R = f * \lambda. \tag{2.9}$$

An appropriate receiver spacing would be $d_1 = d_2 = 10$ ft. (3.3 m).

2.4 SASW DATA REDUCTION

The raw SASW data collected in the field includes the cross power spectrum, coherence function, and auto spectra for each set of receiver spacings in the experiment. Data reduction and analysis requires a significant amount of computational time and is performed back in the laboratory. The data is first converted to ASCII format and then a windows-based program, WinSASW, developed at the University of Texas (Joh, 1992) is used to reduce and interpret the data. The wrapped phase angle from a cross power spectrum is first unwrapped using an interactive masking process. An individual experimental dispersion curve is calculated for that receiver spacing. The individual dispersion curves from an SASW array make up a composite experimental dispersion curve. To facilitate the interpretation of the dispersion curve, the individual dispersion curves are averaged to form a compact experimental dispersion curve.

2.4.1 Masking

The phase of the cross power spectrum, or phase spectrum, and the coherence function for one receiver spacing at a typical soil site are shown in Fig. 2.9. For a site where shear wave velocity generally increases with depth, the wrapped phase spectrum is a sawtooth pattern with the phase gradually increasing from -180° to 180°, with regular jumps from 180° back to -180°. Poor quality data must be discarded and then the proper number of cycles (jumps) specified to extract the phase spectrum. This process is called masking.

The near-field region is masked out using a filter criteria based on wavelength which is defined by:

$$\lambda \text{ is included if } \lambda \leq k * d_1, \tag{2.10}$$

where k is usually 2, as explained in Section 2.3.3. Frequency ranges with low quality phase data, characterized by significantly undulating phase angles, a backwards sawtooth pattern, or scatter caused by random noise, should also be masked out. The coherence function can be a guide in masking, with low coherence generally indicating high random noise and poor data quality. Wavelengths shorter than four times the diameter of the receiver are also masked out:

$$\lambda \text{ is included if } \lambda \geq 4 * D_R \qquad (2.11)$$

where D_R is the diameter of a receiver.

The phase data from the forward and reverse profiles can be averaged together in the frequency domain before masking. An example of the masking process is shown in Fig. 2.9. Once the unwanted data are masked out and the number of cycles specified, the phase spectrum can be unwrapped. The unwrapped phase spectrum is shown in Fig. 2.10.

2.4.2 Experimental Dispersion Curve

The experimental dispersion curve is calculated from the unwrapped phase spectrum and the receiver spacing:

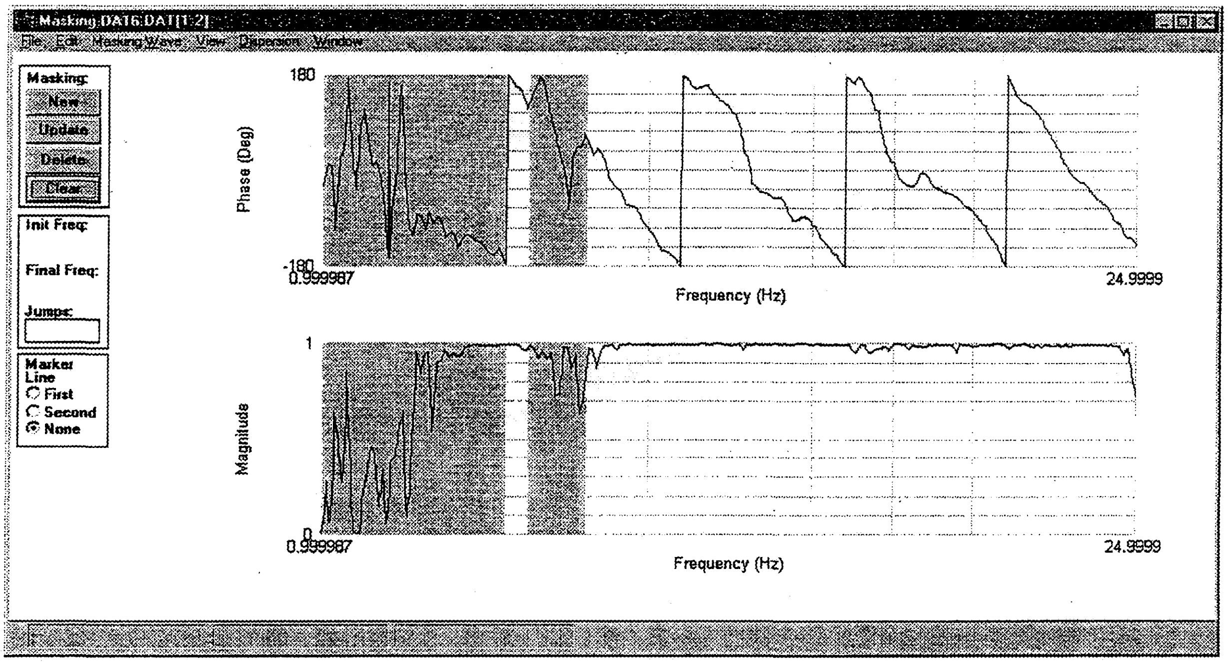
$$V_R = f * \lambda \qquad (2.9)$$

$$V_R = f * d_2 / (\Delta\phi/360°), \qquad (2.12)$$

where d_2 is the distance between receivers and $\Delta\phi$ is the phase difference in degrees. Experimental dispersion curves are generated for each receiver spacing.

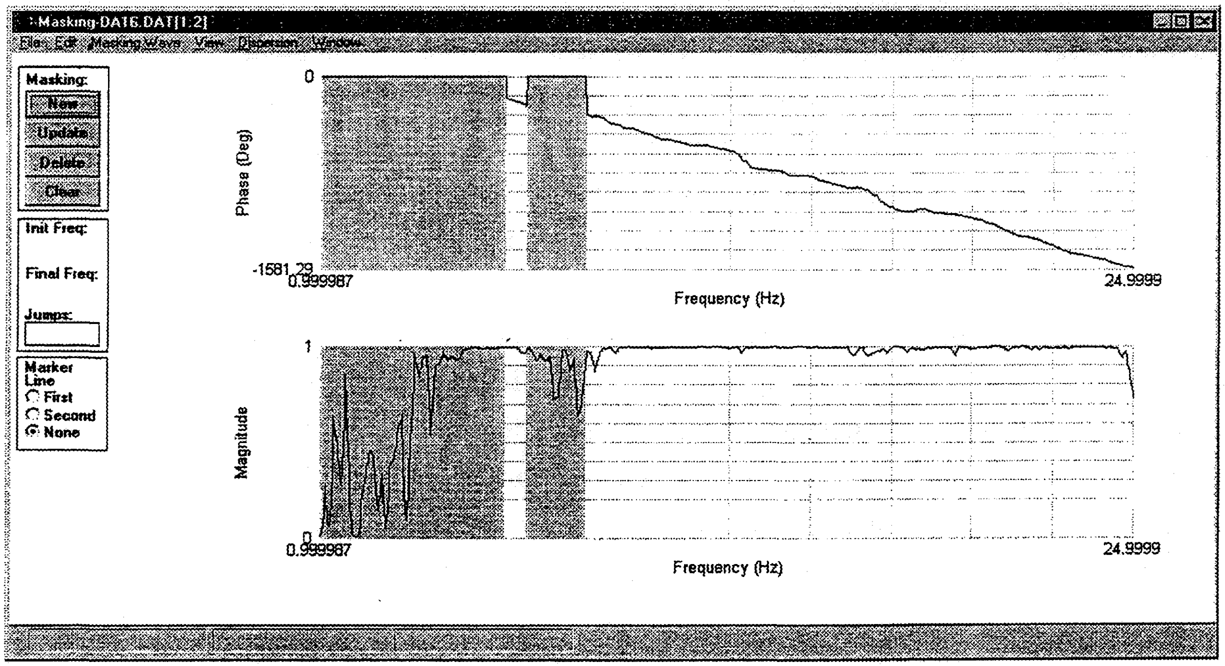
Often the masking process is ambiguous--it may not be clear how many cycles to specify in the unwrapped phase spectrum. Masking is also an iterative process. The unwrapped phase spectrum is modeled as a dispersion curve, checked for consistency, remasked, and modeled again.

All the individual dispersion curves together form the composite experimental dispersion curve. It can be considered the surface wave "signature" of the site.. The composite experimental dispersion curve is sometimes called the field dispersion curve. It may be shown as either phase velocity versus frequency, f, or phase velocity, V_R, versus wavelength, λ. A composite experimental dispersion curve, in terms of log λ - V_R, is shown in Fig. 2.11.



Marker Line
○ First
○ Second
● None

Figure 2.9 Masked Phase of Cross Power Spectrum and Coherence for the 100 ft (30.5 m) Forward Direction Receiver Spacing at Rinaldi Receiving Station.



Marker Line
○ First
○ Second
● None

Figure 2.10 Unwrapped Phase Spectrum and Masked Coherence for the 100 ft (30.5 m) Forward Direction Receiver Spacing at Rinaldi Receiving Station.

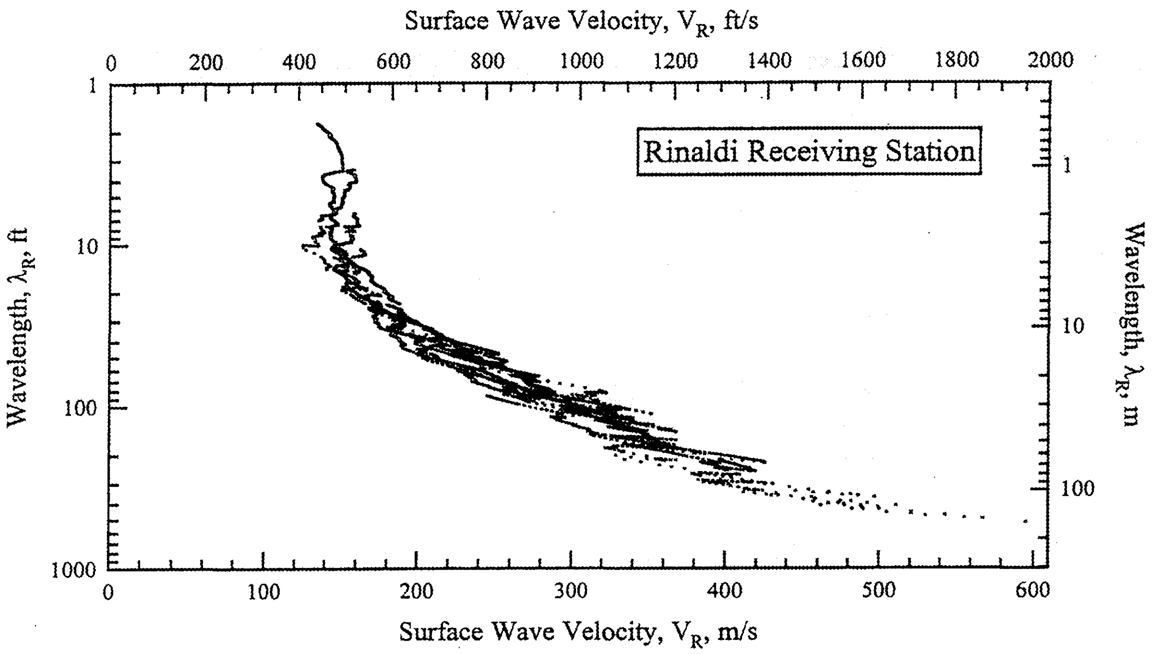


Figure 2.11 Experimental Dispersion Curve for the Rinaldi Receiving Station Site, in Terms of log λ - V_R.

2.4.3 Compact Experimental Dispersion Curve

The composite experimental dispersion curve is difficult to work with in computations. It may contain several thousand data points with considerable scatter. A smoother "compact" dispersion curve containing many fewer points can be calculated. There are several averaging algorithms available for determining the compact dispersion curve. The phase velocities can be averaged in non-overlapping wavelength segments (Rix, 1987). Polynomial best-fit lines to overlapping data segments may produce a smoother curve and more stable inversion process (Joh, 1997).

The compact dispersion curve may be calculated with a linear or logarithmic distribution of data in the wavelength domain. A logarithmically distributed compact dispersion curve gives more weight to the shorter wavelengths (Fig. 2.12), and a linearly distributed compact dispersion curve emphasizes the longer wavelengths. Both distributions are useful in interpreting the dispersion curve; the logarithmically distributed compact curve is used first in modeling the shallow layers and the linear distribution is used for the deeper layers.

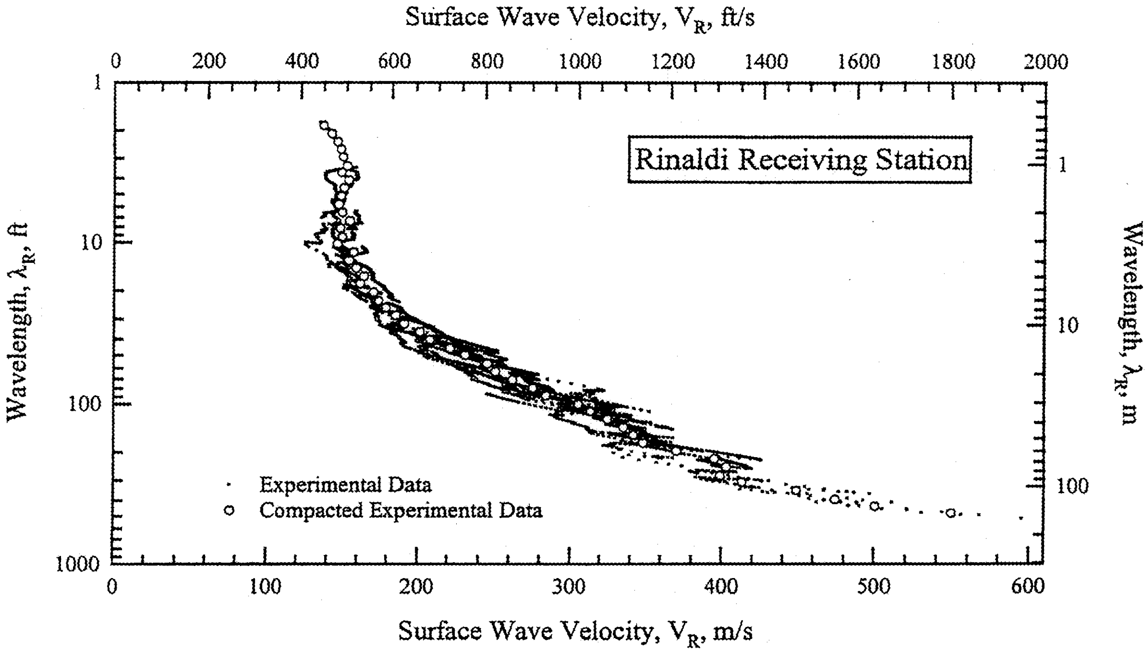


Figure 2.12 Log-Distributed Compact Dispersion Curve for the Rinaldi Receiving Station site.

The averaging can be done in either the frequency or wavelength domain, although frequency domain averaging is theoretically sounder. It separates the variable with uncertainty (V_R) from the one without uncertainty (f). Wavelength domain modeling is usually easier because there is a more apparent physical relationship between wavelength and depth of influence, as shown in Fig. 2.1 and Fig. 2.3.

2.5 SASW DATA INTERPRETATION

The end product of SASW testing is usually a shear wave velocity profile of the subsurface. There are several methods for obtaining this stiffness profile: empirical relationships, iterative forward modeling, and inversion analysis. This section will focus on iterative forward modeling.

The simplified relationship used to interpret data from the steady-state Rayleigh wave method gives the highly smoothed variation of shear wave velocity with depth. Like Heisey et al. (1982) and Roesset et al. (1991), this study found that the shear wave velocity was most closely related to the Rayleigh wave velocity at a depth of 1/3 of the wavelength:

$$z = \lambda/2 \text{ or } \lambda/3. \qquad (2.13)$$

Shear wave velocity is determined by:

$$V_S \cong 1.1 * V_R. \qquad (2.14)$$

2.5.1 Iterative Forward Modeling

In forward modeling, a theoretical dispersion curve is calculated for a given set of material properties. Layer thickness, and layer properties such as shear wave velocity, Poisson's ratio (or compression wave velocity), and mass density are the specified model parameters. The initial assumed profile is based on background information on the site or estimated from past experience. The entire stiffness profile is usually not modeled initially. First, the near-surface properties are modeled, since the short wavelength portion of the dispersion curve (theoretical or experimental) is independent of the properties of the deeper layers. Longer wavelength portions of the dispersion curve are still affected by the near surface properties, so modeling is done with progressively deeper layers.

There are different ways to calculate the theoretical dispersion curve, as discussed in Section 2.5.2. The theoretical dispersion curve is compared to the composite experimental dispersion curve or compact experimental dispersion curve. If they do not match well enough, the material properties are adjusted and the theoretical dispersion curve is recalculated. This process continues until a satisfactory match between theoretical and experimental dispersion curves is obtained. The interpreter must balance the closeness of the fit to data with the reasonableness of the model parameters. An example of the final match between the theoretical dispersion curve and the compact curve is shown in Fig. 2.13.

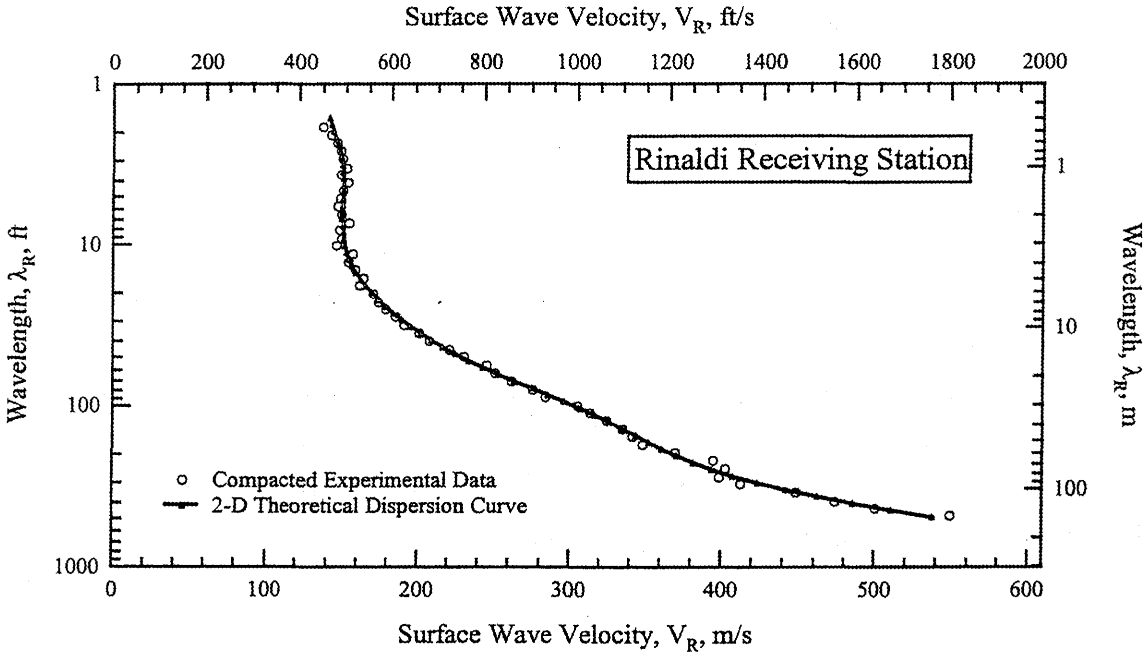


Figure 2.13 Comparison of the Theoretical Dispersion Curve from the SASW Solution with the Compacted Experimental Dispersion Curve, in Terms of log λ - V_R.

The final model stiffness profile is assumed to represent the actual site conditions. A comparison between SASW and borehole seismic results from a "blind" study is shown in Fig. 2.14.

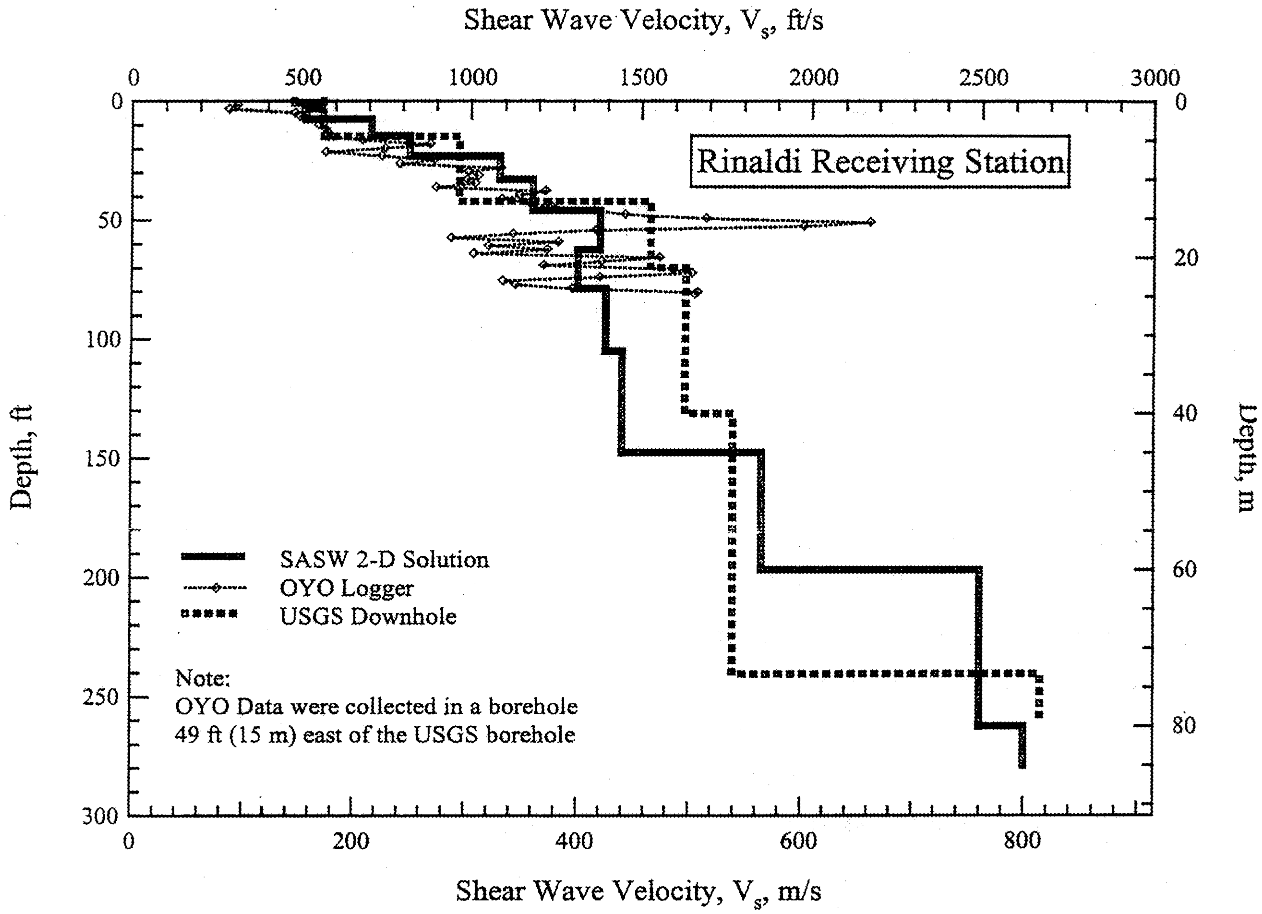


Figure 2.14 Comparison of V_S profiles from SASW 2-D solution, USGS downhole testing (Gibbs et al., 1996), and OYO suspension logging.

For practical reasons, only layer thickness and shear wave velocity are varied in iterative forward modeling. Reasonable estimates of mass density and Poisson's ratio are used throughout the analysis. If the depth of the water table is known, it is be better to specify a compression wave velocity of 5000 ft/s (1500 m/s) for the saturated soil zone and then evaluate Poisson's ratio. The absolute value of mass density is not important since it is only relative differences between layers that affect the theoretical dispersion curve.

Since forward modeling is a trial-and-error procedure and the initial estimate is based on the interpreter's judgment, questions arise concerning the uniqueness and accuracy of the resulting profile. One advantage of a full inversion analysis is that the resolution of the shear wave velocity profile and the sensitivity of the theoretical dispersion curve to the final profile can be quantified. Otherwise, a manual sensitivity analysis can be performed, or the final profile can be reported to a depth with which the interpreter feels confident. For normally dispersive sites V_S can often be resolved to a depth of one half to one third of the longest wavelength in the dispersion curve.

2.5.2 Theoretical Dispersion Curve

The stiffness profile from the forward modeling analysis depends on the method used to calculate the theoretical dispersion curve. The most prominent approaches are the transfer matrix method (Thompson, 1950; and Haskell, 1953) and the dynamic stiffness matrix method.

WinSASW uses the dynamic stiffness matrix method, as described in Kausel and Roesset (1981) and Roesset et al. (1991) to compute a theoretical dispersion curve for a given stiffness profile. It does this by determining the theoretical response of a layered system to a dynamic load. From the vertical response of the system, the theoretical phase spectrum is calculated. The resulting dispersion curve is calculated from the phase spectrum.

It is important to remember that the formulation of the forward problem is a model, based on assumptions which may or may not represent the actual field conditions well. The subsurface is assumed to be a horizontally layered, laterally invariant, homogeneous, isotropic system. It is assumed that if the subsurface model produces a theoretical dispersion curve that is consistent with the field data, the subsurface model is representative of the site. Different formulations of the forward problem exist, from a fundamental-mode Rayleigh wave model to those that incorporate body waves and the experimental geometry.

2.5.2.1 Fundamental-Model Rayleigh Wave Model

In the "2-D" formulation in WinSASW, the response of the layered system due to a vertical line load is calculated. The solution for a plane Rayleigh wave is determined; that is, the wavefronts are planar. The assumption is valid for a very remote source. Body waves are not taken into account. Although it is possible to compute higher modes of propagation, the 2-D analysis in WinSASW uses the first mode (smallest eigenvalue of the dynamic stiffness matrix).

Using the first, or fundamental, mode Rayleigh wave dispersion curve gives good results for sites where the shear wave velocity gradually increases with depth (Foinquinos, 1991). The shear wave velocity profile resulting from a 2-D forward modeling analysis is called a "2-D solution".

2.5.2.2 Full Stress-Wave Model

The "3-D" model simulates body wave effects and higher modes of propagation. These additional waves are important because the surface wave phase data collected in the field are contaminated with body waves and possibly higher modes, and it is not practical to separate modes in SASW analysis. The 3-D formulation models the response of receivers at various distances from a vertical unit circular dynamic load (Kausel and Peek, 1982). The wavefronts are assumed to be cylindrical for the surface waves and hemispherical for the body waves. All stress waves are modeled, so the resulting dispersion curve includes the effects of higher modes of surface waves and body wave reflection and refraction. At sites where the stiffness decreases with depth or there are large contrasts in shear wave velocity, this formulation is a more accurate simulation of the recorded data.

There are several options for 3-D forward modeling: to assume a generalized wavelength-dependent receiver spacing (3-D global) or to incorporate the actual receiver spacings (3-D array) into the model. The computational time is greatly increased, so this theoretical simulation of SASW measurements is only warranted for sites with large stiffness contrasts that must be well resolved. Differences between the 2-D and 3-D solutions are described in Roesset et al. (1991).

2.6 SUMMARY

In this chapter, the development, theoretical basis, field procedures, and methods of data analysis used in the SASW method are reviewed. The goal of SASW testing is to determine a shear wave velocity profile representative of a site. The method takes advantage of the dispersive property of Rayleigh waves, when propagating through a layered system. Testing consists of three parts: field measurements of surface wave phase data, data reduction and generation of an experimental dispersion curve, and evaluation of the corresponding shear wave velocity profile.

The field procedure, including testing equipment and experimental design, is reviewed. Proper sources, receivers, data acquisition system, and experimental setup to collect the surface wave phase spectrum are discussed. Data reduction, consisting of masking out unwanted phase data and generating the experimental dispersion curve, is then explained. Finally, the forward modeling process used to interpret the experimental data is discussed. The current stress wave modeling theories used to generate theoretical surface wave dispersion curves are also briefly presented.

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Appendix C
Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Direct Shear Strength Tests: Direct shear testing, in accordance with ASTM D3080, was performed on select samples which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the samples to the shear box, and reloading the samples, pore pressures set up in the samples due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing 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 accompanying plots.

Maximum Density Tests: The maximum dry density and optimum moisture content of representative bulk soil samples were determined in accordance with ASTM Test Method D1557. Test results are presented on the *Modified Proctor Compaction Test* figures in this appendix.

Moisture and Density Determination Tests: Moisture content (ASTM Test Method D2937) and dry density determinations were performed on relatively undisturbed ring samples obtained from the test borings. The results of these tests are presented in the geotechnical boring logs (Appendix B).

APPENDIX C (Continued)

Expansion Index Tests: The expansion potential of selected material samples were evaluated by the Expansion Index Text, ASTM Test Method D4829. The specimens were molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded to an equivalent 144 psf surcharge and inundated with water until volumetric equilibrium was reached. The results of the tests are presented in the table below:

Sample Location	Sample Description	Expansion Index	Expansion Potential
B-1 at 1-5 Feet	Silty Sand (SM), Reddish Brown	7	Very low
B-5 at 1-5 Feet	Clayey Sand (SC), Reddish Brown	12	Very low
B-7 at 1-5 Feet	Silty Sand (SM), Reddish Brown	11	Very low
B-18 at 3-6 Feet	Silty Sand (SM), Reddish Brown	9	Very low

Particle Size Analysis (ASTM D1140): Particle size analyses were performed by mechanical sieving methods according to ASTM D1140. These tests were performed to assist in the classification of the soil and to determine grain size distributions of the tested soil. The percent fine particles from the analyses are summarized below:

Sample Location	Percent Passing No. 200 Sieve
B-1 at 10 Feet	26
B-2 at 15 Feet	24
B-5 at 1-5 Feet	35
B-5 at 20 Feet	13
B-18 at 0.5-2.0 Feet	15

APPENDIX C (Continued)

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with Caltrans Test Method CT643 and standard geochemical methods. The results are presented in the table below:

Sample Location	Sample Description	pH	Minimum Resistivity (ohms-cm)
B-1 at 1-5 Feet	Silty Sand (SM), Reddish Brown	8.02	1400
B-5 at 1-5 Feet	Clayey Sand (SC), Reddish Brown	8.09	1590
B-6 at 1-5 Feet	Clayey Sand (SC), Reddish Brown	8.07	1500
B-18 at 3-6 Feet	Silty Sand (SM), Reddish Brown	6.85	2300

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method CT422. The results are presented below:

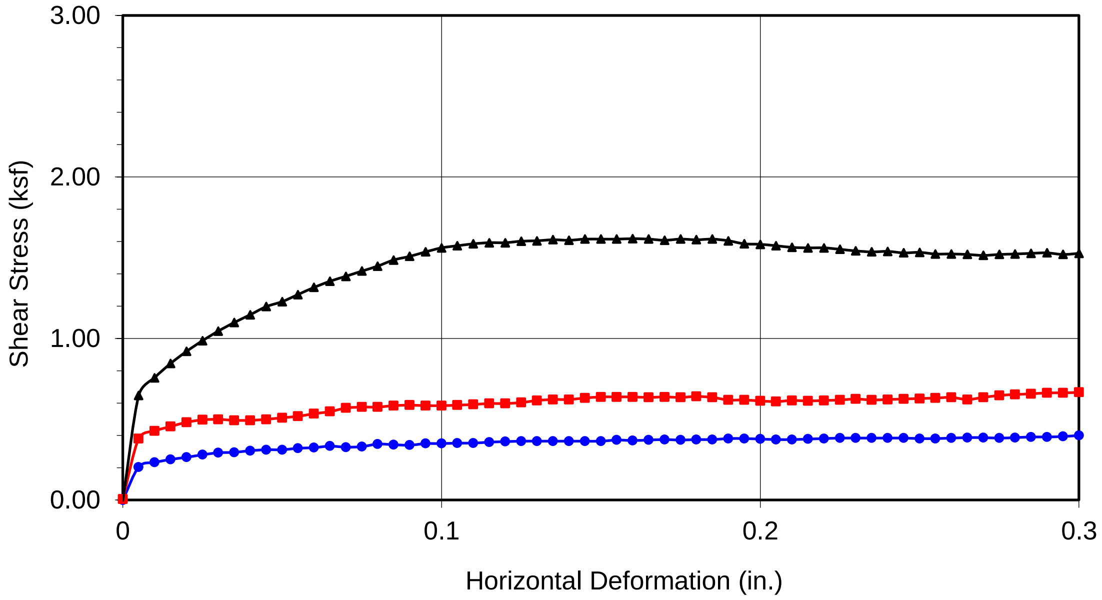
Sample Location	Sample Description	Chloride Content, ppm
B-1 at 1-5 Feet	Silty Sand (SM), Reddish Brown	120
B-5 at 1-5 Feet	Clayey Sand (SC), Reddish Brown	60
B-6 at 1-5 Feet	Clayey Sand (SC), Reddish Brown	60
B-18 at 3-6 Feet	Silty Sand (SM), Reddish Brown	60

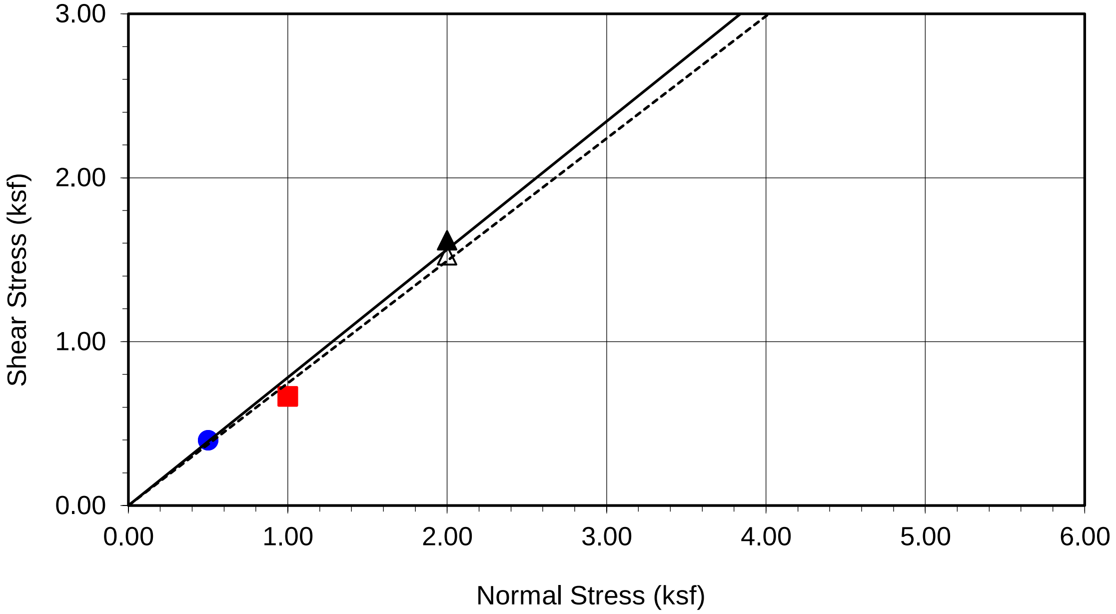
APPENDIX C (Continued)

<u>Soluble Sulfates</u>: The soluble sulfate contents of selected samples were determined by standard geochemical methods (Caltrans Test Method CT417). The test results are presented in the table below:

Sample Location	Sample Description	Sulfate Content, ppm	Exposure Class*
B-1 at 1-5 Feet	Silty Sand (SM), Reddish Brown	165	S0
B-5 at 1-5 Feet	Clayey Sand (SC), Reddish Brown	180	S0
B-6 at 1-5 Feet	Clayey Sand (SC), Reddish Brown	270	S0
B-18 at 3-6 Feet	Silty Sand (SM), Reddish Brown	165	S0

*Based on the 2014 edition of American Concrete Institute (ACI) Committee 318R, Table No. 19.3.1.1





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

Soil Identification:
Silty Sand (SM), Reddish Brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	0	38
Ultimate	0	37

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.399	■ 0.666	▲ 1.618
Shear Stress @ End of Test (ksf)	○ 0.399	□ 0.666	△ 1.527
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 (%)	4.50	4.50	4.50
Dry Density (pcf)	85.0	97.2	104.0
Saturation (%)	12.3	16.5	19.6
Soil Height Before Shearing (in.)	0.9047	0.9537	0.9498
Final Moisture Content (%)	22.4	18.6	16.7

Leighton

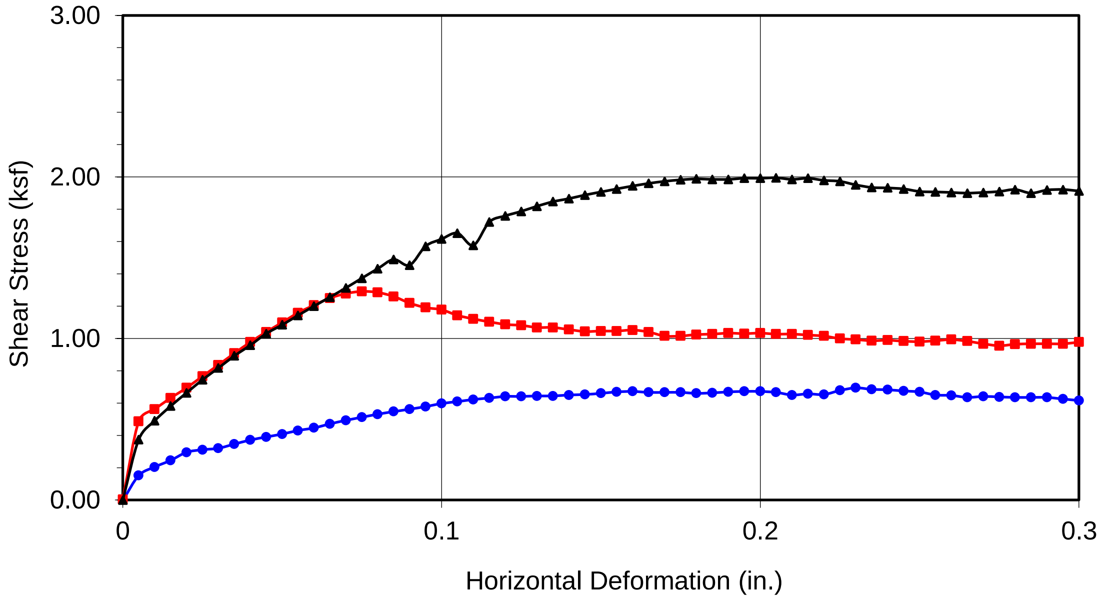
DIRECT SHEAR TEST RESULTS

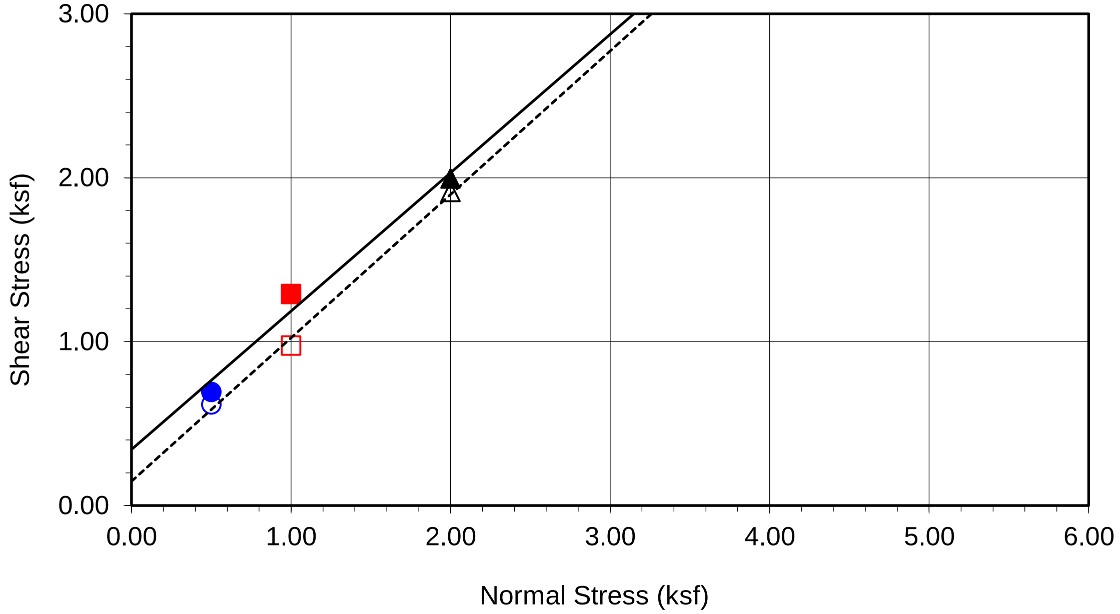
Consolidated Drained - ASTM D 3080

Project No.: 12764.001

Sharp Metro Master Plan Geo

08-20





Boring No.	**B-5**
Sample No.	**R-1**
Depth (ft)	**5**

Sample Type: Ring

Soil Identification:
Silty Sand (SM), Reddish Brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	342	40
Ultimate	148	41

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.694	■ 1.291	▲ 1.995
Shear Stress @ End of Test (ksf)	○ 0.616	□ 0.977	△ 1.913
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 (%)	7.71	7.71	7.71
Dry Density (pcf)	95.0	109.7	99.2
Saturation (%)	26.9	38.8	29.8
Soil Height Before Shearing (in.)	0.9947	0.9946	0.9632
Final Moisture Content (%)	19.6	16.5	17.0

Leighton

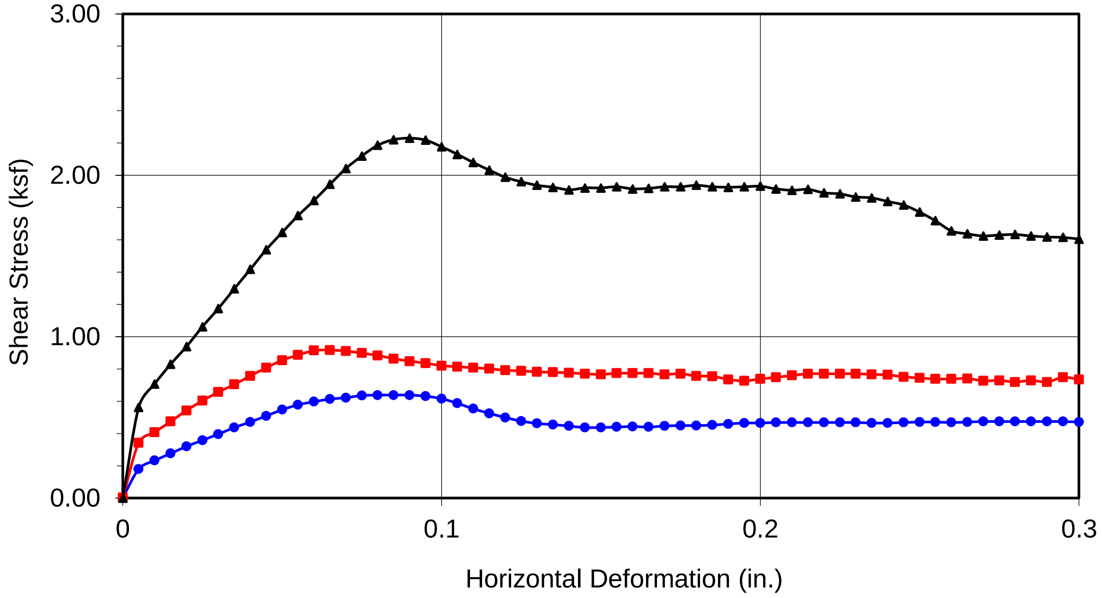
DIRECT SHEAR TEST RESULTS

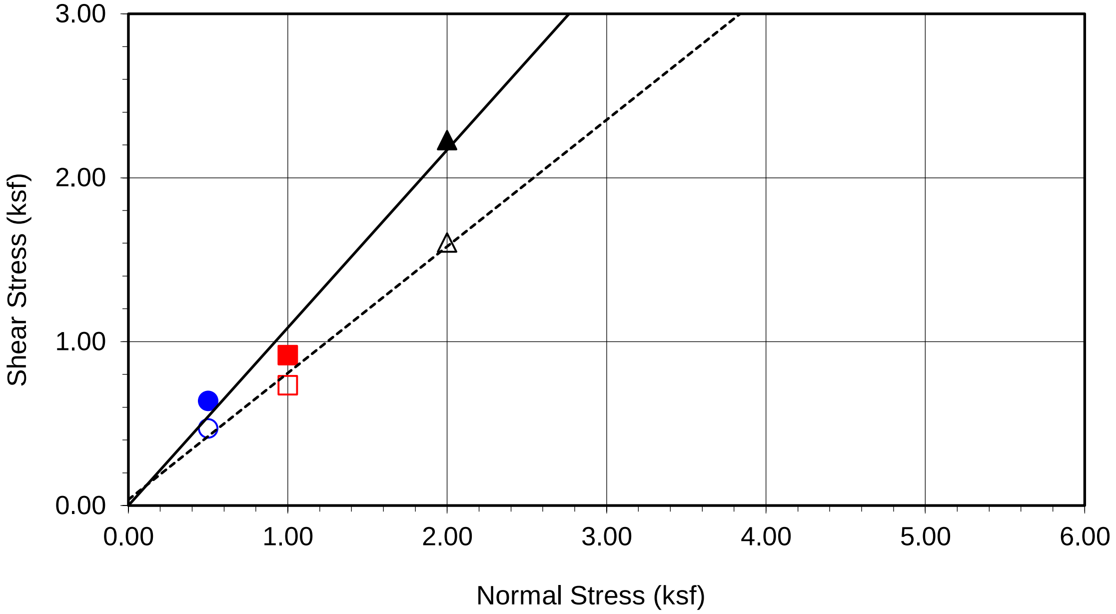
Consolidated Drained - ASTM D 3080

Project No.: 12764.001

Sharp Metro Master Plan Geo

08-20





Boring No.	**B-7**
Sample No.	**R-1**
Depth (ft)	**5**

Sample Type: Ring

Soil Identification:
Silty Sand (SM), Reddish Brown.

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	0	47
Ultimate	36	38

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.638	■ 0.917	▲ 2.230
Shear Stress @ End of Test (ksf)	○ 0.471	□ 0.735	△ 1.605
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 (%)	7.55	7.55	7.55
Dry Density (pcf)	103.4	106.0	107.1
Saturation (%)	32.4	34.5	35.6
Soil Height Before Shearing (in.)	0.9867	0.9861	0.9798
Final Moisture Content (%)	19.3	17.7	17.0

Leighton

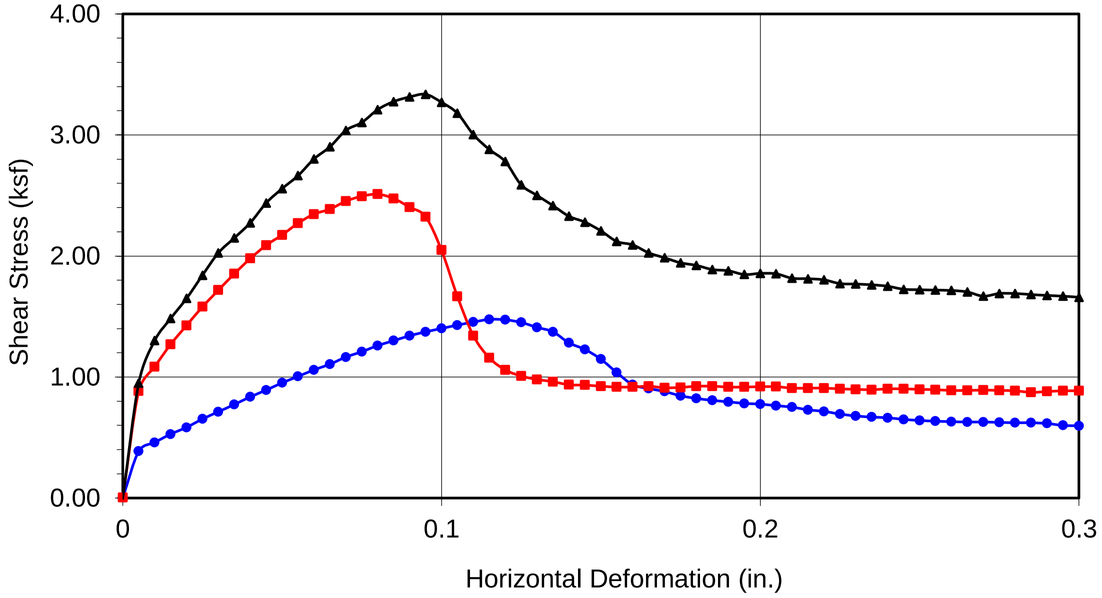
DIRECT SHEAR TEST RESULTS

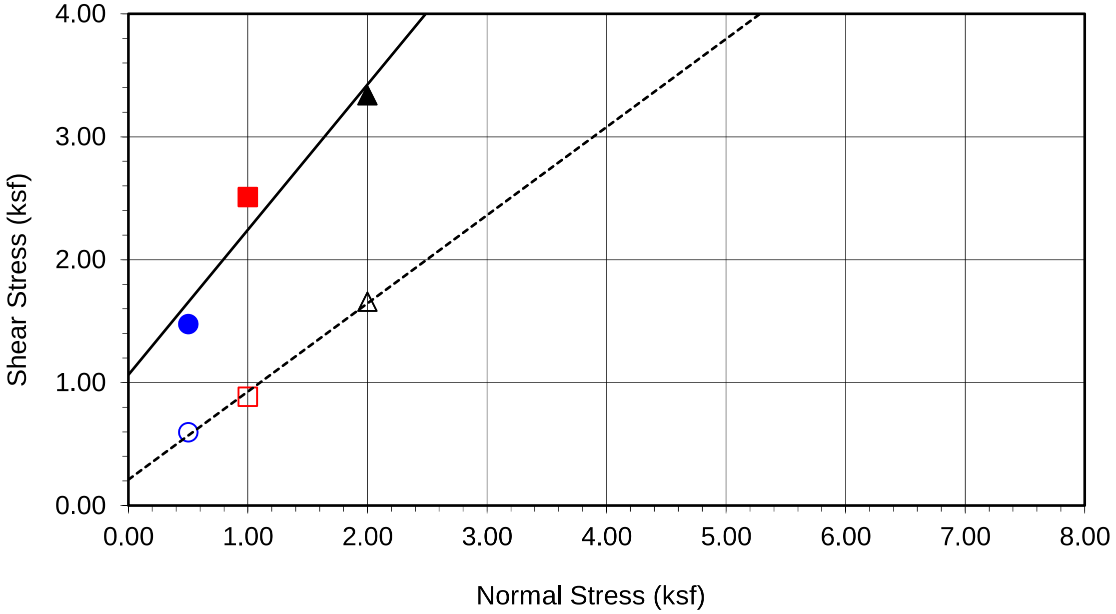
Consolidated Drained - ASTM D 3080

Project No.: 12764.001

Sharp Metro Master Plan Geo

08-20





Boring No.	**B-18**
Sample No.	**R-3**
Depth (ft)	**10.0 - 11.0**
Sample Type:	Ring

Soil Identification:
Silty Sand (SM), Reddish Brown.

Strength Parameters

	C (psf)	ϕ (°)
Peak	1063	50
Ultimate	211	36

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 1.476	■ 2.510	▲ 3.336
Shear Stress @ End of Test (ksf)	○ 0.597	□ 0.886	△ 1.659
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 (%)	11.75	11.75	11.75
Dry Density (pcf)	115.9	117.1	117.5
Saturation (%)	69.8	72.1	73.0
Soil Height Before Shearing (in.)	0.9948	0.9922	0.9755
Final Moisture Content (%)	16.3	14.8	14.4

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12764.001

Sharp Metro Master Plan Geo

08-20

Direct Shear; B-18, R-3 (07-29-20)

Leighton

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Sharp Metro Master Plan Geo
Tested By: L. Parrella
Date: 08/12/20
Project No.: 12764.001
Input By: M. Vinet
Date: 08/18/20
Boring No.: B-2
Depth (ft.): 1.0 - 5.0
Sample No.: B-1
Soil Identification: Silty Sand (SM), Reddish Brown.

Preparation Method: ☒ Moist ☐ Dry

☒ Mechanical Ram ☐ Manual Ram

Mold Volume (ft³) 0.03330

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5631	5707	5622			
Weight of Mold (g)	3521	3521	3521			
Net Weight of Soil (g)	2110	2186	2101			
Wet Weight of Soil + Cont. (g)	1205.8	1207.7	1217.2			
Dry Weight of Soil + Cont. (g)	1178.0	1170.2	1170.6			
Weight of Container (g)	703.2	704.9	712.2			
Moisture Content (%)	5.9	8.1	10.2			
Wet Density (pcf)	139.7	144.7	139.1			
Dry Density (pcf)	132.0	133.9	126.3			

Maximum Dry Density (pcf) 134.5 **Optimum Moisture Content (%)** 7.4

PROCEDURE USED

☒ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

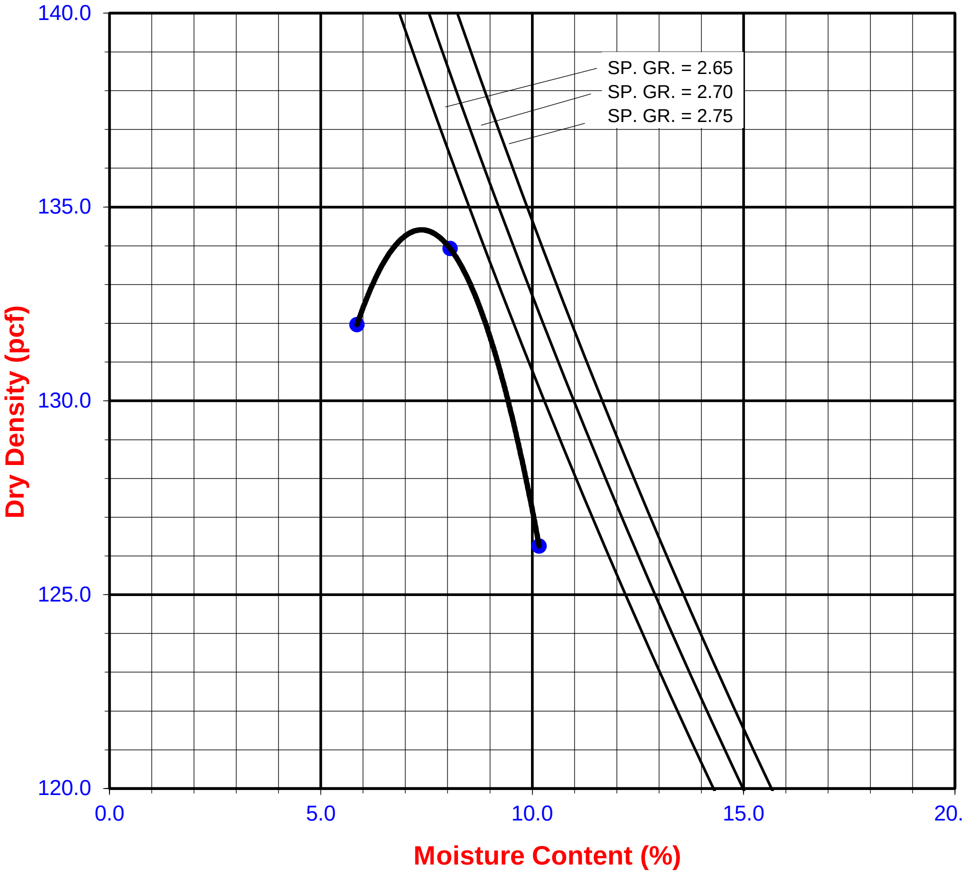
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Leighton

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Sharp Metro Master Plan Geo
Tested By: L. Parrella
Date: 08/14/20
Project No.: 12764.001
Input By: M. Vinet
Date: 08/18/20
Boring No.: B-6
Depth (ft.): 1.0 - 5.0
Sample No.: B-1
Soil Identification: Clayey Sand with Gravel (SC)g, Reddish Brown.

Preparation Method: ☒ Moist ☐ Dry

☒ Mechanical Ram ☐ Manual Ram

Mold Volume (ft³) 0.03330

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5526	5637	5609			
Weight of Mold (g)	3521	3521	3521			
Net Weight of Soil (g)	2005	2116	2088			
Wet Weight of Soil + Cont. (g)	1203.2	1204.0	1208.3			
Dry Weight of Soil + Cont. (g)	1173.2	1163.3	1160.1			
Weight of Container (g)	706.0	704.0	703.5			
Moisture Content (%)	6.4	8.9	10.6			
Wet Density (pcf)	132.7	140.1	138.2			
Dry Density (pcf)	124.7	128.7	125.0			

Maximum Dry Density (pcf) 128.8 **Optimum Moisture Content (%)** 8.5

PROCEDURE USED

☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☒ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

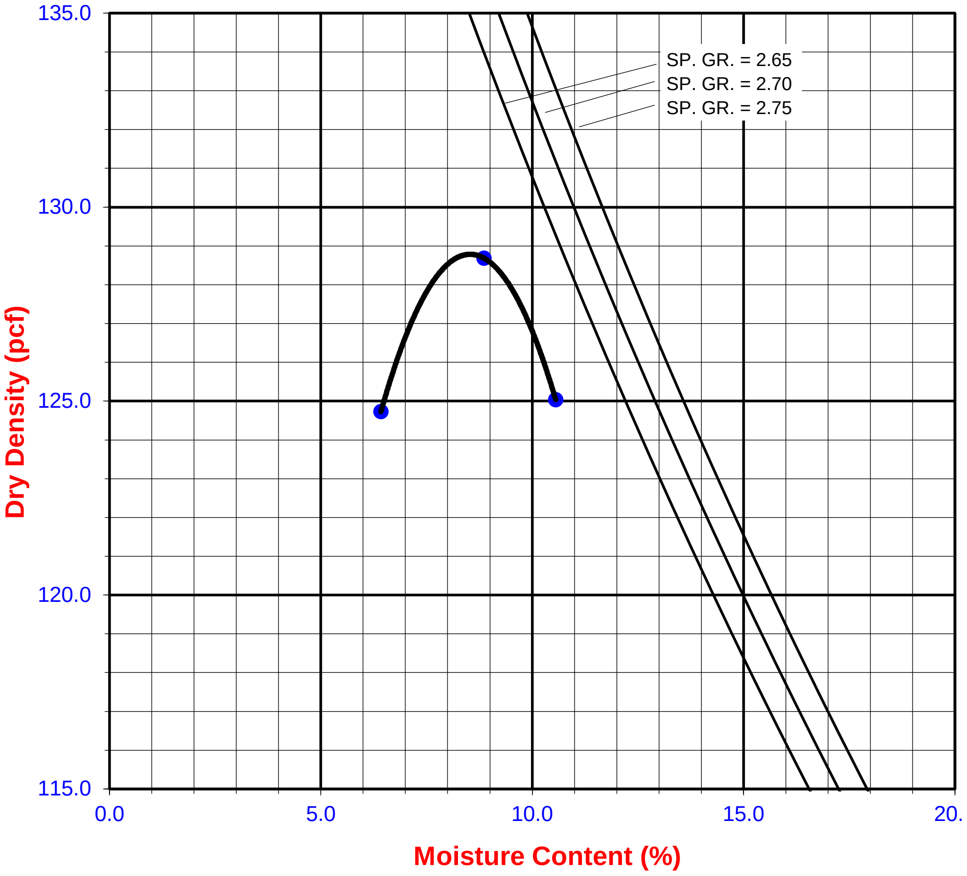
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Appendix D
Seismic Hazard Analysis

Sharp Metropolitan Medical Campus/ Mary Birch Hospital Expansion

Latitude, Longitude: 32.7982, -117.1544

Date	12/8/2020, 3:30:29 PM
Design Code Reference Document	ASCE7-10
Risk Category	IV
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.08	MCE_R ground motion. (for 0.2 second period)
S_1	0.413	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.08	Site-modified spectral acceleration value
S_{M1}	0.573	Site-modified spectral acceleration value
S_{DS}	0.72	Numeric seismic design value at 0.2 second SA
S_{D1}	0.382	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.387	Site amplification factor at 1.0 second
PGA	0.461	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.461	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.08	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.222	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.81	Factored deterministic acceleration value. (0.2 second)
S1RT	0.413	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.442	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.754	Factored deterministic acceleration value. (1.0 second)
PGAd	0.7	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.884	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.935	Mapped value of the risk coefficient at a period of 1 s

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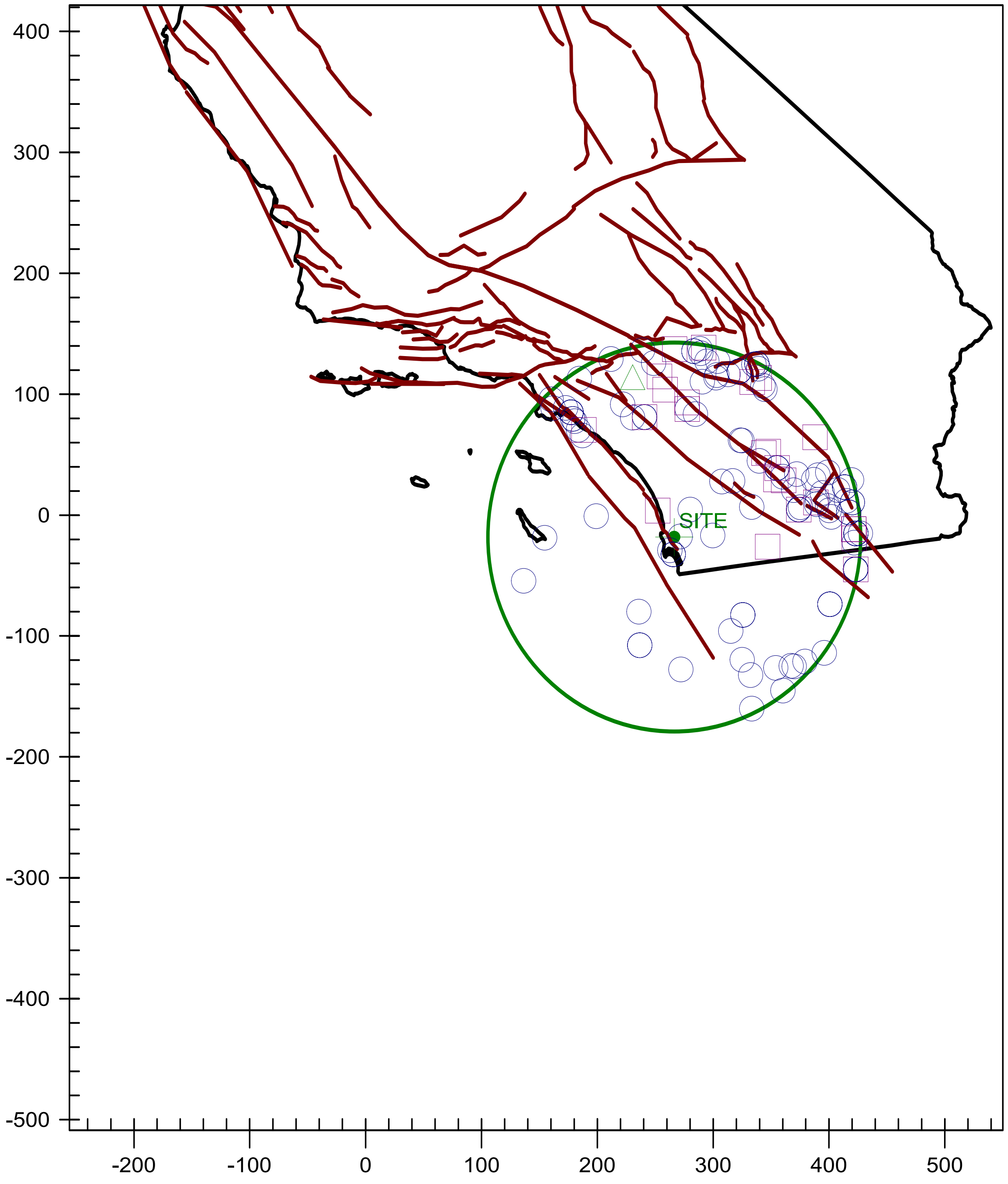
EARTHQUAKE EPICENTER MAP

Sharp MBH



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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: 12764.001
DATE: 09-24-2020
JOB NAME: Sharp MBH
EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT
MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00
SITE COORDINATES:
   SITE LATITUDE:  32.7982
   SITE LONGITUDE:  117.1554
SEARCH DATES:
           START DATE:   1800
           END DATE:   1999
SEARCH RADIUS:
           100.0 mi
           160.9 km
ATTENUATION RELATION:   2) Boore et al. (1997) Horiz. - NEHRP C (520)
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   ASSUMED SOURCE TYPE:  SS [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
                              -------------------------
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
MGI |32.8000|117.1000|05/25/1803| 0 0 0.0|  0.0| 5.00| 0.129 |VIII|  3.2(  5.2)
DMG |32.7000|117.2000|05/27/1862|20 0 0.0|  0.0| 5.90| 0.137 |VIII|  7.3( 11.7)
T-A |32.6700|117.1700|10/21/1862| 0 0 0.0|  0.0| 5.00| 0.074 | VII|  8.9( 14.3)
T-A |32.6700|117.1700|12/00/1856| 0 0 0.0|  0.0| 5.00| 0.074 | VII|  8.9( 14.3)
T-A |32.6700|117.1700|05/24/1865| 0 0 0.0|  0.0| 5.00| 0.074 | VII|  8.9( 14.3)
DMG |33.0000|117.3000|11/22/1800|2130 0.0|  0.0| 6.50| 0.106 | VII| 16.3( 26.2)
MGI |33.0000|117.0000|09/21/1856| 730 0.0|  0.0| 5.00| 0.048 | VI | 16.6( 26.7)
DMG |32.8000|116.8000|10/23/1894|23 3 0.0|  0.0| 5.70| 0.058 | VI | 20.6( 33.2)
DMG |33.2000|116.7000|01/01/1920| 235 0.0|  0.0| 5.00| 0.025 |  V | 38.3( 61.6)
MGI |33.2000|116.6000|10/12/1920|1748 0.0|  0.0| 5.30| 0.027 |  V | 42.5( 68.3)
T-A |32.2500|117.5000|01/13/1877|20 0 0.0|  0.0| 5.00| 0.023 | IV | 42.8( 68.9)
PAS |32.9710|117.8700|07/13/1986|1347 8.2|  6.0| 5.30| 0.027 |  V | 43.1( 69.4)
DMG |33.0000|116.4330|06/04/1940|1035 8.3|  0.0| 5.10| 0.024 | IV | 44.1( 71.0)
DMG |32.7000|116.3000|02/24/1892| 720 0.0|  0.0| 6.70| 0.050 | VI | 50.1( 80.7)
DMG |32.2000|116.5500|11/05/1949| 43524.0|  0.0| 5.10| 0.020 | IV | 54.3( 87.4)
DMG |32.2000|116.5500|11/04/1949|204238.0|  0.0| 5.70| 0.028 |  V | 54.3( 87.4)
DMG |32.0830|116.6670|11/25/1934| 818 0.0|  0.0| 5.00| 0.018 | IV | 57.0( 91.7)
DMG |32.0000|117.5000|06/24/1939|1627 0.0|  0.0| 5.00| 0.018 | IV | 58.7( 94.4)
DMG |32.0000|117.5000|05/01/1939|2353 0.0|  0.0| 5.00| 0.018 | IV | 58.7( 94.4)
DMG |33.3430|116.3460|04/28/1969|232042.9| 20.0| 5.80| 0.027 |  V | 60.1( 96.7)
PAS |33.5010|116.5130|02/25/1980|104738.5| 13.6| 5.50| 0.023 | IV | 61.1( 98.3)
DMG |33.5000|116.5000|09/30/1916| 211 0.0|  0.0| 5.00| 0.017 | IV | 61.5( 99.0)
DMG |33.2000|116.2000|05/28/1892|1115 0.0|  0.0| 6.30| 0.034 |  V | 61.9( 99.6)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.017 | IV | 63.8(102.7)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.029 |  V | 63.8(102.7)
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DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.017 | IV | 63.8(102.7)

DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.017 | IV | 64.3(103.5)

DMG |33.4000|116.3000|02/09/1890|12 6 0.0|  0.0| 6.30| 0.033 |  V | 64.6(104.0)

DMG |33.1900|116.1290|04/09/1968| 22859.1| 11.1| 6.40| 0.035 |  V | 65.3(105.1)

DMG |33.2830|116.1830|03/19/1954| 95429.0|  0.0| 6.20| 0.031 |  V | 65.5(105.4)

DMG |33.2830|116.1830|03/23/1954| 41450.0|  0.0| 5.10| 0.017 | IV | 65.5(105.4)

DMG |33.2830|116.1830|03/19/1954|102117.0|  0.0| 5.50| 0.022 | IV | 65.5(105.4)

DMG |33.2830|116.1830|03/19/1954| 95556.0|  0.0| 5.00| 0.017 | IV | 65.5(105.4)

DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.022 | IV | 65.5(105.4)

DMG |33.2170|116.1330|08/15/1945|175624.0|  0.0| 5.70| 0.024 | IV | 65.9(106.0)

DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.042 | VI | 66.3(106.7)

DMG |33.7500|117.0000|06/06/1918|2232 0.0|  0.0| 5.00| 0.016 | IV | 66.3(106.7)

DMG |33.4080|116.2610|03/25/1937|1649 1.8| 10.0| 6.00| 0.028 |  V | 66.7(107.3)

DMG |32.9670|116.0000|10/22/1942|181326.0|  0.0| 5.00| 0.016 | IV | 68.0(109.4)

DMG |32.9670|116.0000|10/21/1942|162213.0|  0.0| 6.50| 0.036 |  V | 68.0(109.4)

DMG |32.9670|116.0000|10/21/1942|162519.0|  0.0| 5.00| 0.016 | IV | 68.0(109.4)

DMG |32.9670|116.0000|10/21/1942|162654.0|  0.0| 5.00| 0.016 | IV | 68.0(109.4)

DMG |31.8110|117.1310|12/22/1964|205433.2|  2.3| 5.60| 0.022 | IV | 68.2(109.7)

DMG |33.1130|116.0370|04/09/1968| 3 353.5|  5.0| 5.20| 0.018 | IV | 68.3(110.0)

DMG |32.9830|115.9830|05/23/1942|154729.0|  0.0| 5.00| 0.016 | IV | 69.2(111.3)

DMG |32.8170|118.3500|12/26/1951| 04654.0|  0.0| 5.90| 0.026 |  V | 69.3(111.6)

DMG |33.8000|117.0000|12/25/1899|1225 0.0|  0.0| 6.40| 0.033 |  V | 69.7(112.2)

DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.017 | IV | 71.9(115.6)

DMG |31.8670|116.5710|02/27/1937| 12918.4| 10.0| 5.00| 0.015 | IV | 72.8(117.1)

DMG |33.2310|116.0040|05/26/1957|155933.6| 15.1| 5.00| 0.015 | IV | 73.0(117.6)

DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.030 |  V | 73.4(118.2)

MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.015 | IV | 73.8(118.7)

DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.016 | IV | 75.3(121.2)
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                            -------------------------
                            EARTHQUAKE SEARCH RESULTS
                            -------------------------
------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|  0.0| 6.00| 0.025 |  V | 76.1(122.5)
PAS |33.0130|115.8390|11/24/1987|131556.5|  2.4| 6.00| 0.025 |  V | 77.7(125.1)
DMG |33.0000|115.8330|01/08/1946|185418.0|  0.0| 5.40| 0.018 | IV | 77.9(125.4)
DMG |33.0330|115.8210|09/30/1971|224611.3|  8.0| 5.10| 0.015 | IV | 79.0(127.2)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.018 | IV | 80.0(128.7)
DMG |33.1830|115.8500|04/25/1957|222412.0|  0.0| 5.10| 0.015 | IV | 80.1(128.9)
DMG |33.9500|116.8500|09/28/1946| 719 9.0|  0.0| 5.00| 0.014 | IV | 81.4(131.1)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.015 | IV | 81.5(131.2)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.015 | IV | 81.5(131.2)
DMG |31.7500|116.5000|04/29/1935|20 8 0.0|  0.0| 5.00| 0.014 | IV | 81.9(131.7)
PAS |33.0820|115.7750|11/24/1987| 15414.5|  4.9| 5.80| 0.021 | IV | 82.3(132.5)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.027 |  V | 83.2(133.8)
DMG |33.2160|115.8080|04/25/1957|215738.7| -0.3| 5.20| 0.015 | IV | 83.2(133.9)
DMG |32.9830|115.7330|01/24/1951| 717 2.6|  0.0| 5.60| 0.019 | IV | 83.4(134.3)
DMG |32.5000|118.5500|02/24/1948| 81510.0|  0.0| 5.30| 0.016 | IV | 83.6(134.6)
DMG |32.9500|115.7170|06/14/1953| 41729.9|  0.0| 5.50| 0.018 | IV | 84.1(135.3)
DMG |32.9000|115.7000|10/02/1928|19 1 0.0|  0.0| 5.00| 0.014 | III| 84.7(136.3)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.014 | IV | 84.8(136.4)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.016 | IV | 84.8(136.4)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.014 | III| 84.8(136.4)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.014 | IV | 84.8(136.4)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.014 | III| 84.8(136.4)
DMG |33.9760|116.7210|06/12/1944|104534.7| 10.0| 5.10| 0.014 | IV | 85.1(136.9)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 7.00| 0.039 |  V | 85.3(137.3)
DMG |31.7960|116.2690|06/11/1963|152338.3| -2.0| 5.80| 0.020 | IV | 86.4(139.0)
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DMG |33.9940|116.7120|06/12/1944|111636.0| 10.0| 5.30| 0.016 | IV | 86.4(139.1)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.016 | IV | 88.4(142.2)
DMG |33.2330|115.7170|10/22/1942| 15038.0|  0.0| 5.50| 0.017 | IV | 88.5(142.5)
PAS |33.9980|116.6060|07/08/1986| 92044.5| 11.7| 5.60| 0.018 | IV | 88.7(142.7)
DMG |33.9330|116.3830|12/04/1948|234317.0|  0.0| 6.50| 0.029 |  V | 90.1(145.0)
DMG |32.2500|115.7500|12/01/1958| 6 2 0.0|  0.0| 5.50| 0.017 | IV | 90.1(145.1)
DMG |32.2500|115.7500|12/01/1958| 32118.0|  0.0| 5.80| 0.020 | IV | 90.1(145.1)
DMG |32.2500|115.7500|12/01/1958| 350 0.0|  0.0| 5.00| 0.013 | III| 90.1(145.1)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.015 | IV | 90.3(145.3)
GSP |33.8760|116.2670|06/29/1992|160142.8|  1.0| 5.20| 0.014 | IV | 90.4(145.4)
PAS |33.0980|115.6320|04/26/1981|12 928.4|  3.8| 5.70| 0.019 | IV | 90.7(145.9)
GSG |31.8060|116.1280|03/23/1994|025916.2| 22.0| 5.00| 0.013 | III| 91.0(146.5)
T-A |33.5000|115.8200|05/00/1868| 0 0 0.0|  0.0| 6.30| 0.025 |  V | 91.1(146.7)
GSP |33.9020|116.2840|07/24/1992|181436.2|  9.0| 5.00| 0.013 | III| 91.3(146.9)
DMG |34.1000|116.8000|10/24/1935|1448 7.6|  0.0| 5.10| 0.013 | III| 92.2(148.3)
DMG |34.0170|116.5000|07/25/1947| 04631.0|  0.0| 5.00| 0.013 | III| 92.2(148.4)
DMG |34.0170|116.5000|07/25/1947| 61949.0|  0.0| 5.20| 0.014 | IV | 92.2(148.4)
DMG |34.0170|116.5000|07/26/1947| 24941.0|  0.0| 5.10| 0.013 | III| 92.2(148.4)
DMG |34.0170|116.5000|07/24/1947|221046.0|  0.0| 5.50| 0.017 | IV | 92.2(148.4)
DMG |31.8000|116.1000|10/10/1953|1849 6.0|  0.0| 5.00| 0.013 | III| 92.4(148.7)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.016 | IV | 92.8(149.4)
DMG |34.1000|116.7000|02/07/1889| 520 0.0|  0.0| 5.30| 0.015 | IV | 93.6(150.7)
GSP |33.9610|116.3180|04/23/1992|045023.0| 12.0| 6.10| 0.022 | IV | 93.7(150.8)
PAS |33.0140|115.5550|10/16/1979| 65842.8|  9.1| 5.50| 0.016 | IV | 94.0(151.2)
PAS |32.9270|115.5400|10/16/1979| 54910.2| 10.4| 5.10| 0.013 | III| 94.1(151.4)
PAS |32.9280|115.5390|10/16/1979| 61948.7|  9.2| 5.10| 0.013 | III| 94.2(151.5)
DMG |33.1170|115.5670|07/29/1950|143632.0|  0.0| 5.50| 0.016 | IV | 94.6(152.3)
DMG |33.1170|115.5670|07/28/1950|175048.0|  0.0| 5.40| 0.015 | IV | 94.6(152.3)
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                            EARTHQUAKE SEARCH RESULTS
                            -------------------------
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |31.8330|116.0000|05/10/1956|114854.0|  0.0| 5.00| 0.012 | III| 94.8(152.5)
GSP |34.1630|116.8550|06/28/1992|144321.0|  6.0| 5.30| 0.014 | IV | 95.8(154.2)
DMG |32.8000|115.5000|06/23/1915| 456 0.0|  0.0| 6.25| 0.024 | IV | 96.1(154.6)
DMG |32.8000|115.5000|06/23/1915| 359 0.0|  0.0| 6.25| 0.024 | IV | 96.1(154.6)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.012 | III| 96.2(154.8)
DMG |32.7330|115.5000|05/19/1940| 43640.9|  0.0| 6.70| 0.030 |  V | 96.2(154.8)
DMG |34.1800|116.9200|01/16/1930| 034 3.6|  0.0| 5.10| 0.013 | III| 96.4(155.1)
DMG |34.1800|116.9200|01/16/1930| 02433.9|  0.0| 5.20| 0.014 | III| 96.4(155.1)
MGI |32.7000|115.5000|01/01/1927|13 0 0.0|  0.0| 5.30| 0.014 | IV | 96.4(155.1)
DMG |34.2000|117.1000|09/20/1907| 154 0.0|  0.0| 6.00| 0.021 | IV | 96.8(155.8)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.012 | III| 96.9(155.9)
DMG |33.0000|115.5000|12/17/1955| 6 729.0|  0.0| 5.40| 0.015 | IV | 97.0(156.0)
DMG |33.0000|115.5000|02/26/1930| 230 0.0|  0.0| 5.00| 0.012 | III| 97.0(156.0)
DMG |32.7670|115.4830|05/19/1940| 63540.0|  0.0| 5.50| 0.016 | IV | 97.1(156.3)
DMG |32.7670|115.4830|05/19/1940| 63320.0|  0.0| 5.00| 0.012 | III| 97.1(156.3)
DMG |32.7670|115.4830|05/19/1940| 55134.0|  0.0| 5.50| 0.016 | IV | 97.1(156.3)
DMG |32.7670|115.4830|05/19/1940| 455 0.0|  0.0| 5.50| 0.016 | IV | 97.1(156.3)
DMG |31.5000|116.5000|10/17/1954|225718.0|  0.0| 5.70| 0.018 | IV | 97.5(156.9)
GSP |34.0290|116.3210|08/21/1993|014638.4|  9.0| 5.00| 0.012 | III| 97.6(157.1)
DMG |34.2000|117.4000|07/22/1899| 046 0.0|  0.0| 5.50| 0.016 | IV | 97.8(157.4)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.014 | III| 97.8(157.4)
GSP |34.1950|116.8620|08/17/1992|204152.1| 11.0| 5.30| 0.014 | IV | 97.9(157.6)
DMG |31.6250|116.2110|06/10/1969| 34132.7| -2.0| 5.00| 0.012 | III| 98.0(157.7)
DMG |32.5000|115.5000|05/01/1918| 432 0.0|  0.0| 5.00| 0.012 | III| 98.4(158.4)
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MGI |32.5000|115.5000|04/16/1925| 330 0.0|  0.0| 5.00| 0.012 | III| 98.4(158.4)
MGI |32.5000|115.5000|04/16/1925| 520 0.0|  0.0| 5.30| 0.014 | IV | 98.4(158.4)
DMG |32.5000|115.5000|11/07/1923|2357 0.0|  0.0| 5.50| 0.016 | IV | 98.4(158.4)
DMG |32.5000|115.5000|04/19/1906| 030 0.0|  0.0| 6.00| 0.020 | IV | 98.4(158.4)
DMG |32.5000|115.5000|01/01/1927| 81645.0|  0.0| 5.75| 0.018 | IV | 98.4(158.4)
DMG |32.5000|115.5000|09/08/1921|1924 0.0|  0.0| 5.00| 0.012 | III| 98.4(158.4)
DMG |32.5000|115.5000|01/01/1927| 91330.0|  0.0| 5.50| 0.016 | IV | 98.4(158.4)
DMG |32.5000|115.5000|11/05/1923|22 7 0.0|  0.0| 5.00| 0.012 | III| 98.4(158.4)
GSP |34.0640|116.3610|09/15/1992|084711.3|  9.0| 5.20| 0.013 | III| 98.7(158.8)
GSN |34.2030|116.8270|06/28/1992|150530.7|  5.0| 6.70| 0.030 |  V | 98.8(159.0)
DMG |33.1670|115.5000|12/20/1935| 745 0.0|  0.0| 5.00| 0.012 | III| 99.2(159.6)
PAS |32.7660|115.4410|10/15/1979|231930.0|  9.3| 5.20| 0.013 | III| 99.5(160.2)
DMG |34.0670|116.3330|05/18/1940| 55120.2|  0.0| 5.20| 0.013 | III| 99.6(160.3)
DMG |34.0670|116.3330|05/18/1940| 72132.7|  0.0| 5.00| 0.012 | III| 99.6(160.3)
PAS |31.8900|115.8210|05/08/1985|234020.8|  6.0| 5.00| 0.012 | III|100.0(160.8)
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-END OF SEARCH-   145 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  1999

LENGTH OF SEARCH TIME:   200  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 3.2 MILES (5.2 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.137 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.565
  b-value=  0.388
  beta-value=  0.893

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TABLE OF MAGNITUDES AND EXCEEDANCES:
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  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |      145        |   0.72864
     4.5     |      145        |   0.72864
     5.0     |      145        |   0.72864
     5.5     |       56        |   0.28141
     6.0     |       25        |   0.12563
     6.5     |        8        |   0.04020
     7.0     |        1        |   0.00503
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Appendix E
General Earthwork and Grading Specifications for Rough Grading

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.

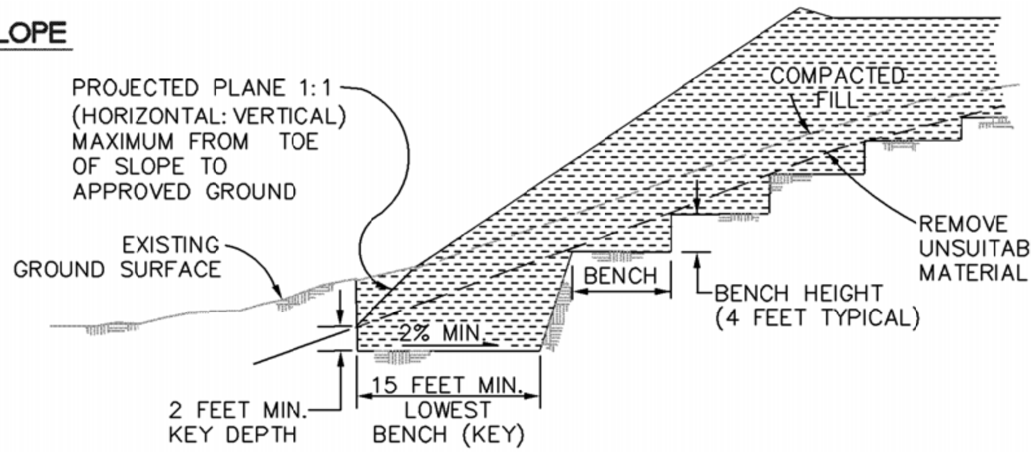
7.3 Lift Thickness

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.

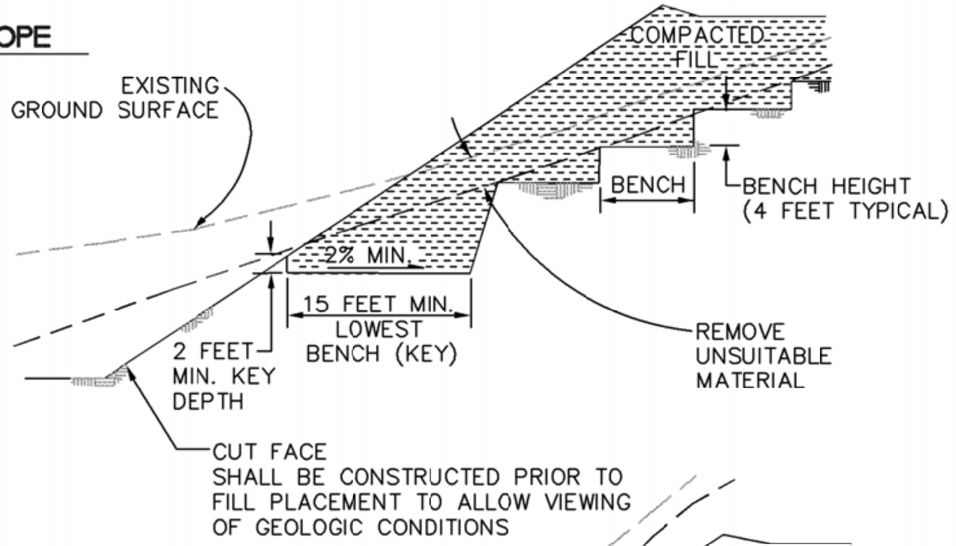
7.4 Observation and Testing

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

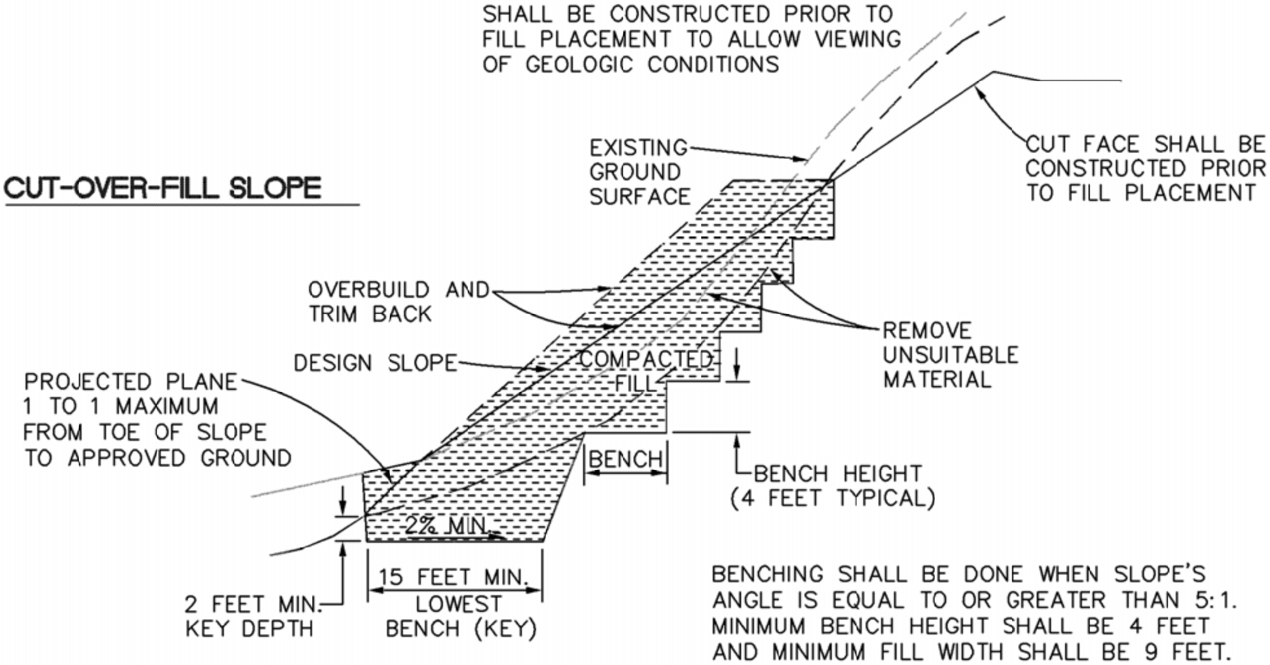
FILL SLOPE



FILL-OVER-CUT SLOPE



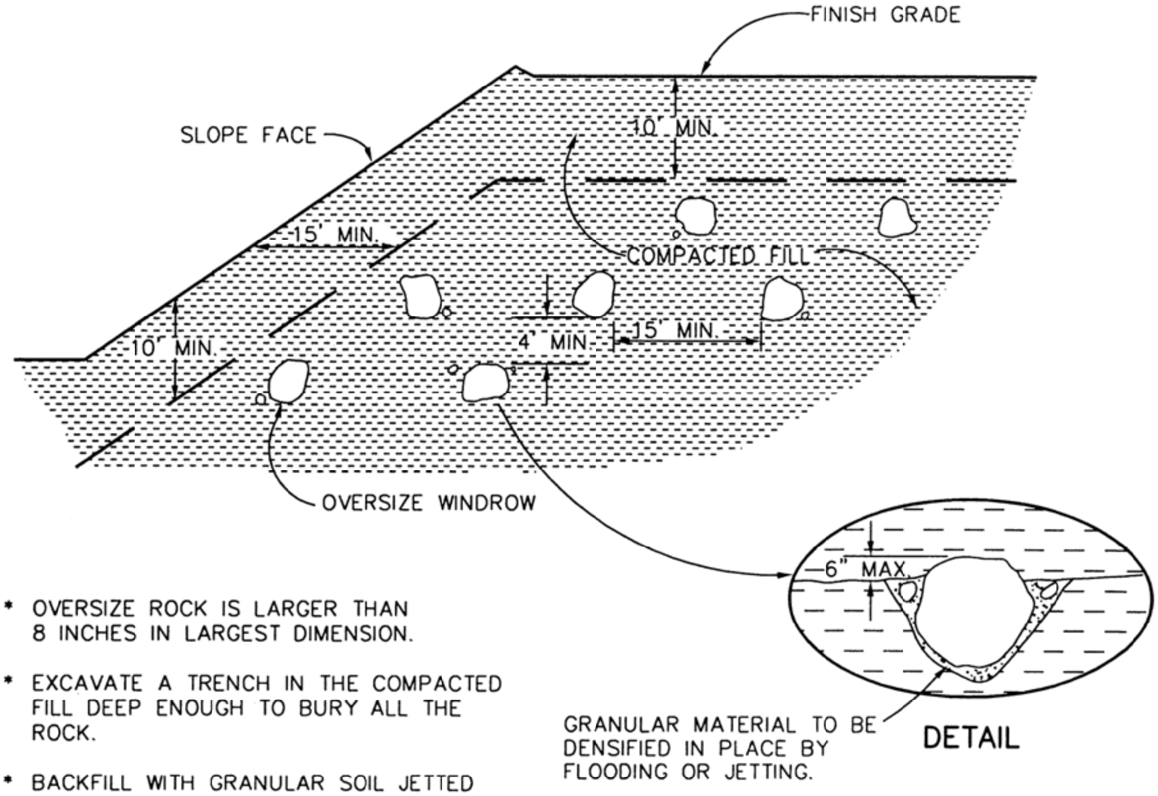
CUT-OVER-FILL SLOPE



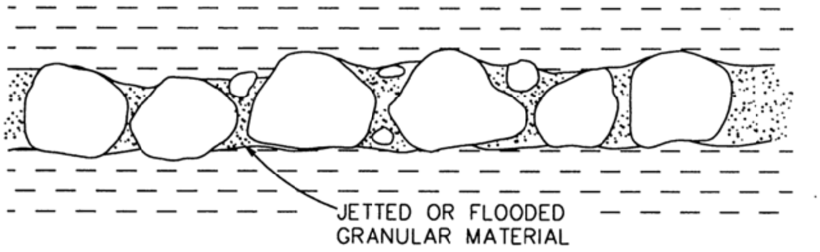
BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A



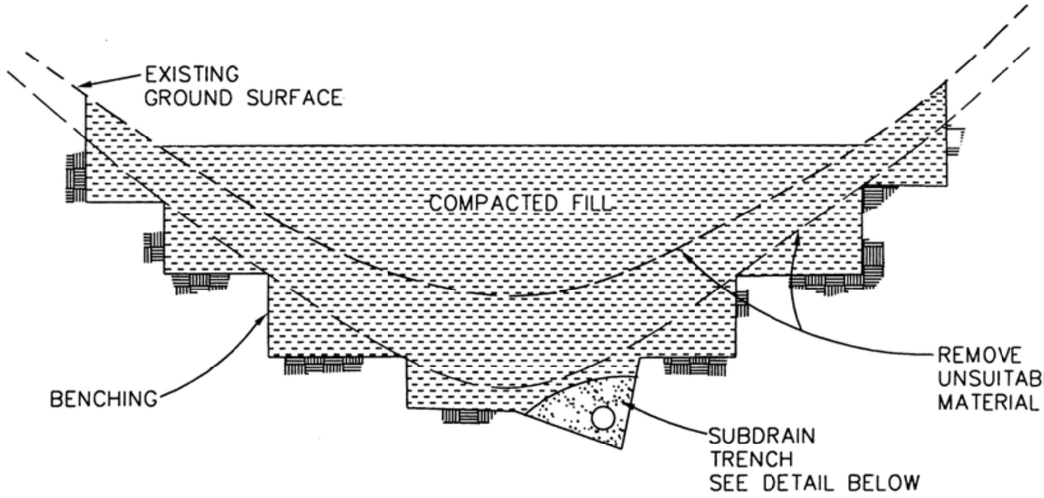
- * OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- * EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- * BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- * DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- * WINDROW OF BURIED ROCK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

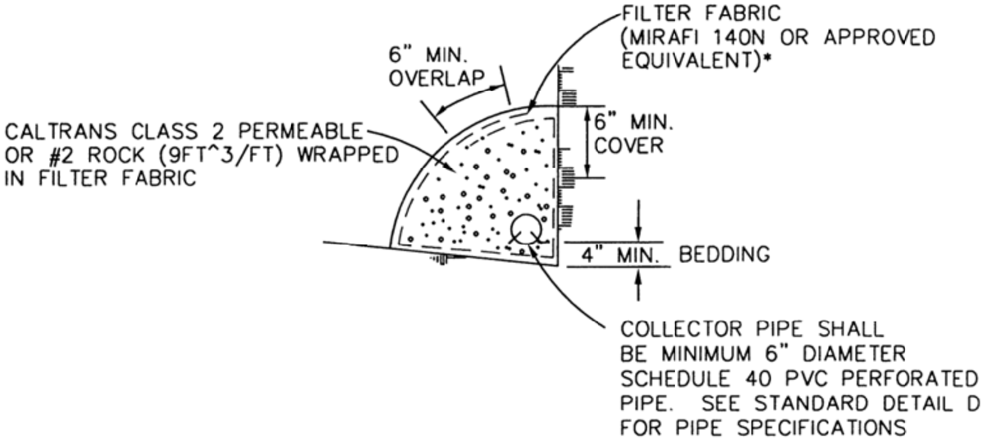


TYPICAL PROFILE ALONG WINDROW

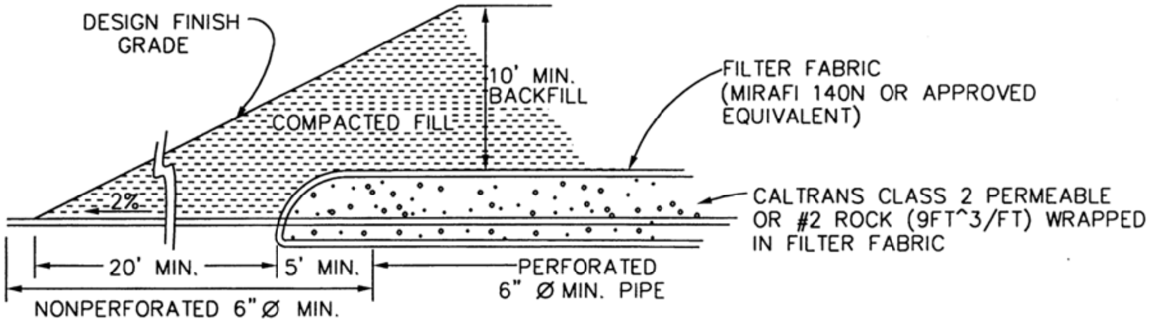
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B





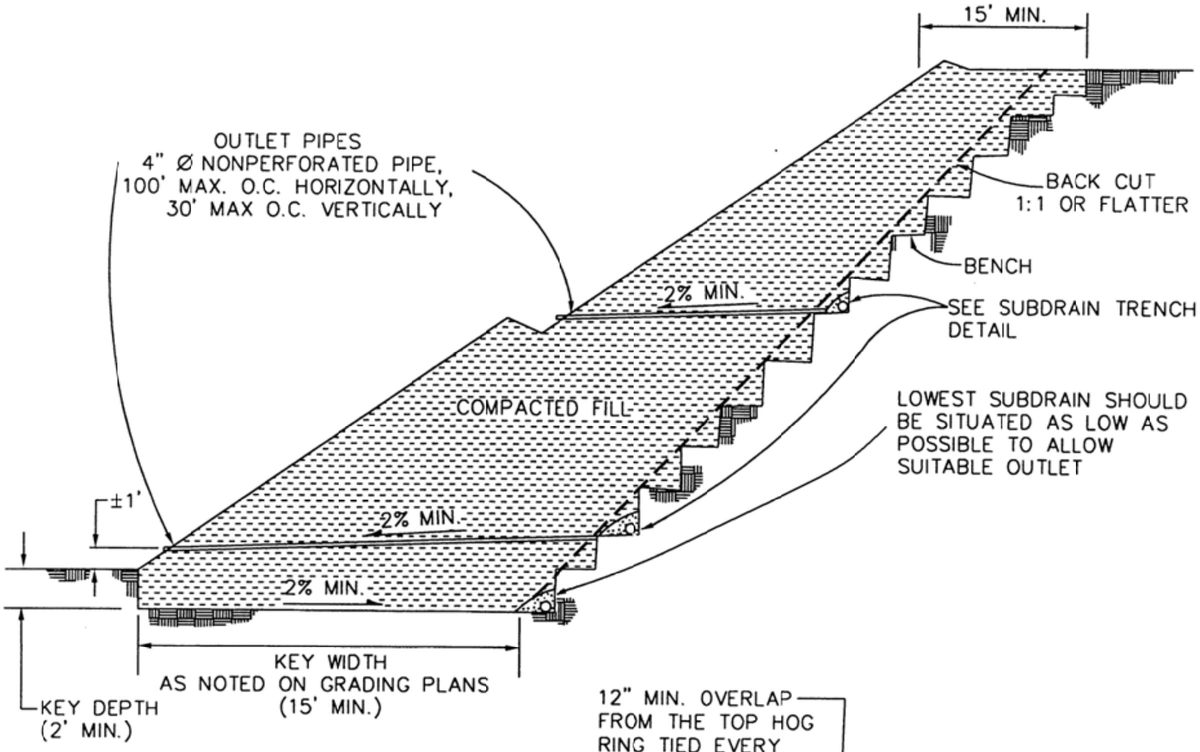
SUBDRAIN DETAIL

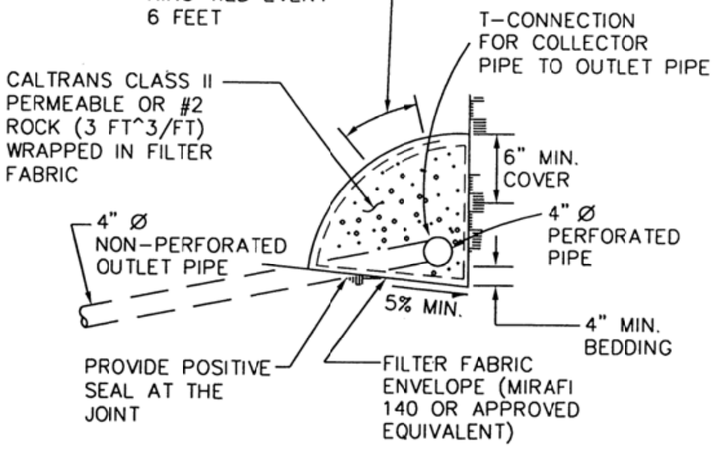


DETAIL OF CANYON SUBDRAIN OUTLET

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.

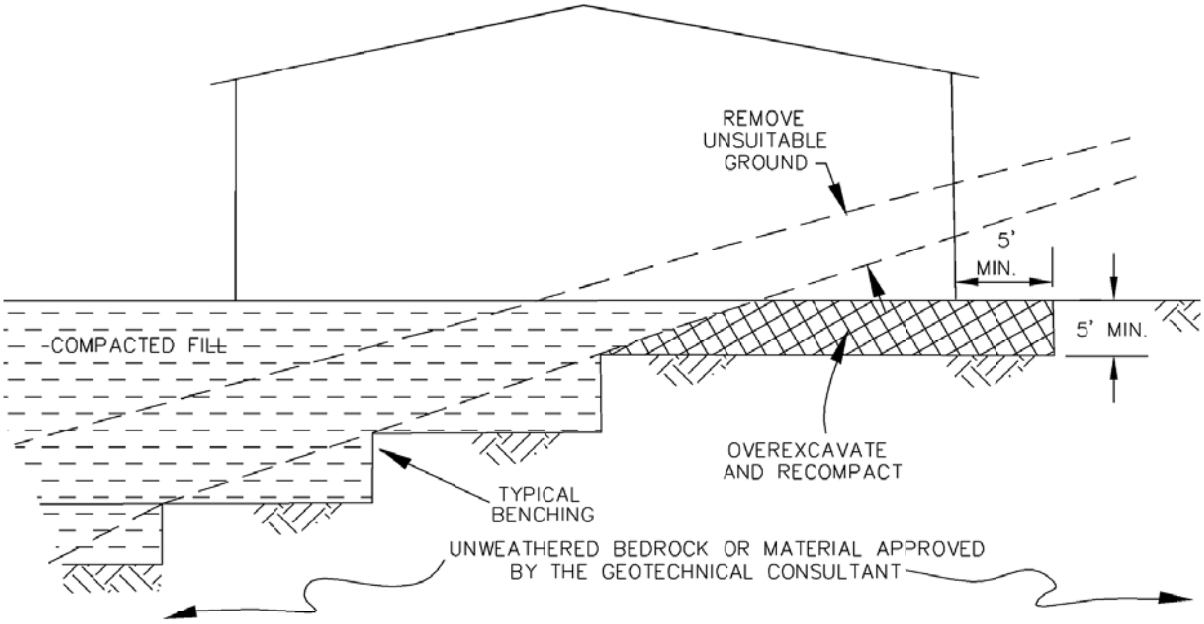
SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

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

BUTTRESS OR REPLACEMENT FILL SUBDRAINS

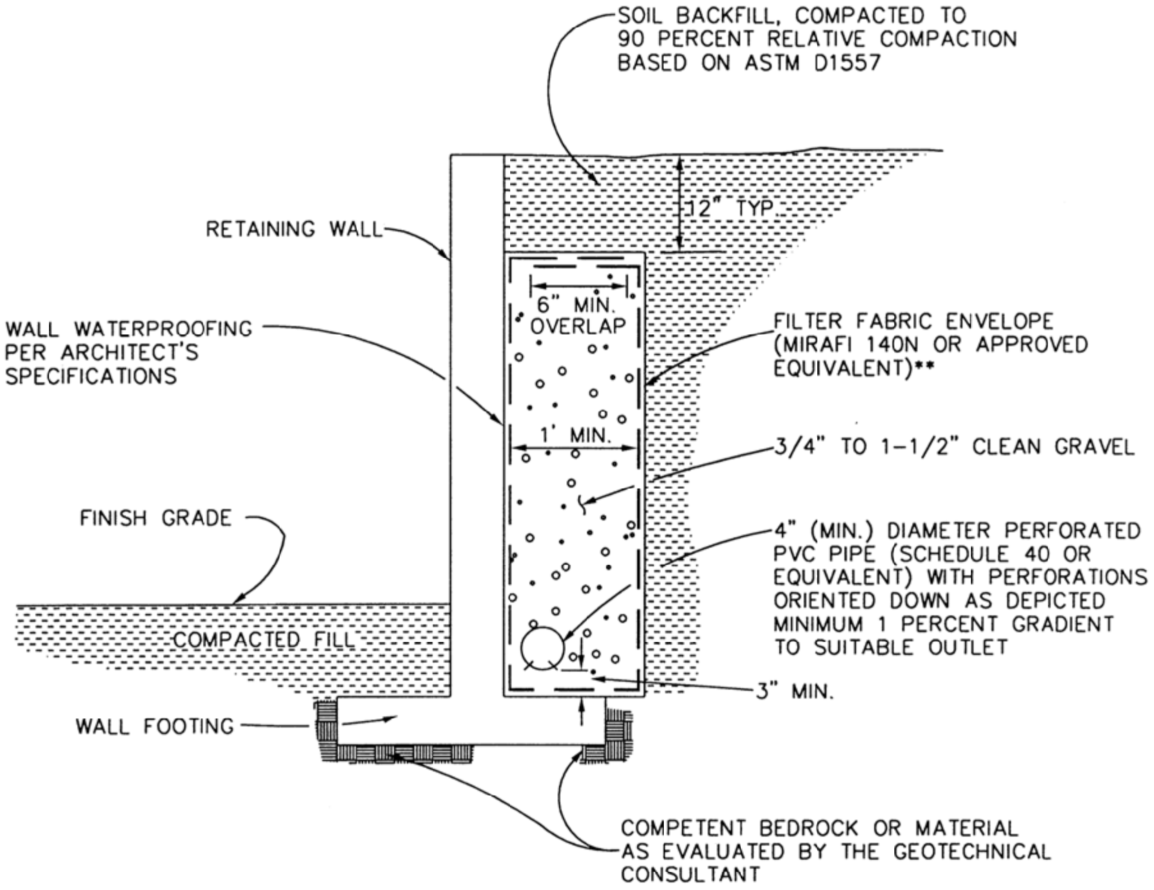
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D

CUT–FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

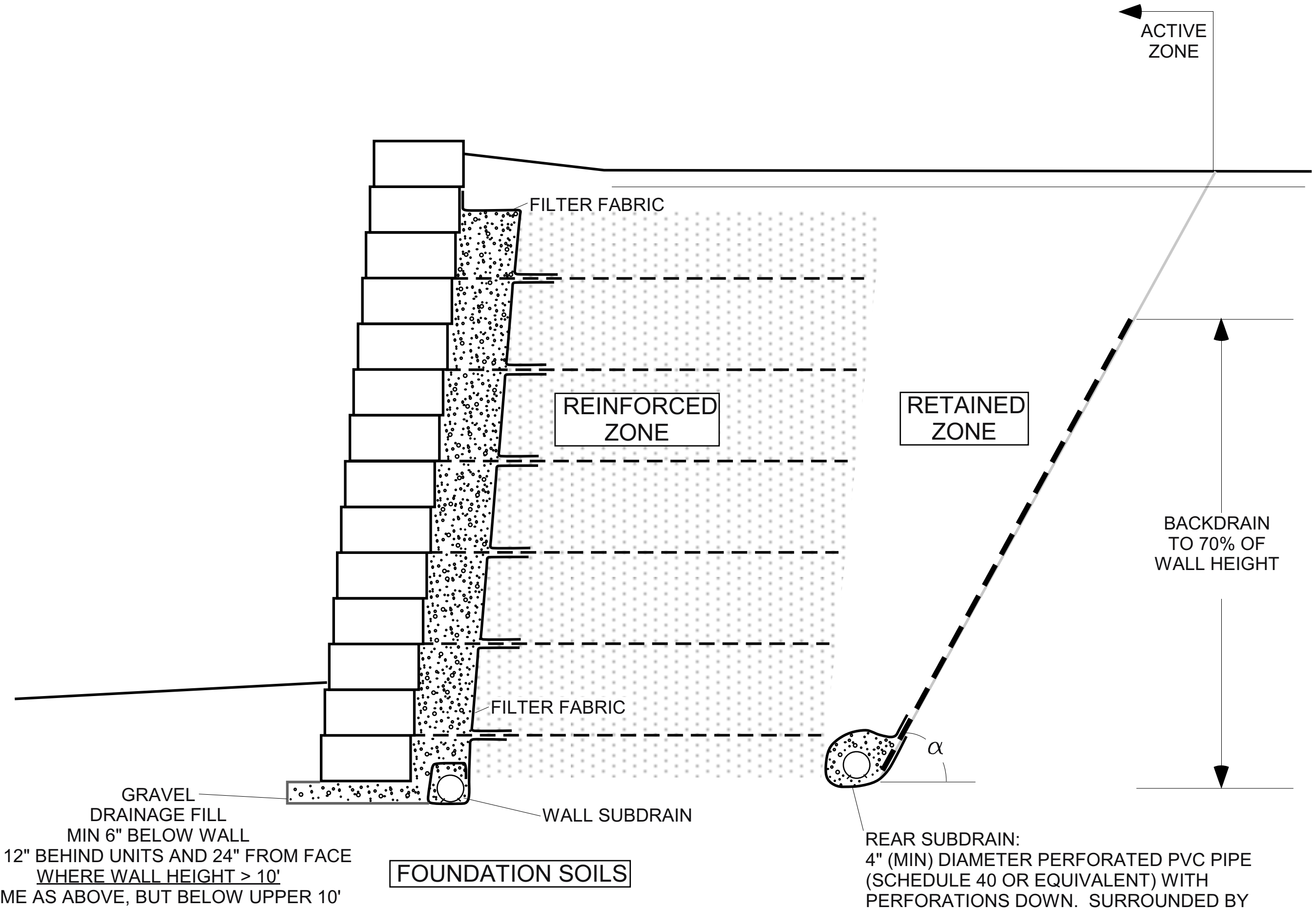
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



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



GRAVEL
DRAINAGE FILL
MIN 6" BELOW WALL
MIN 12" BEHIND UNITS AND 24" FROM FACE
<u>WHERE WALL HEIGHT > 10'</u>
SAME AS ABOVE, BUT BELOW UPPER 10'
INCREASE GRAVEL TO
MIN 24" BEHIND UNIT AND 36" FROM FACE
<u>WHERE WALL HEIGHT > 20'</u>
SAME AS ABOVE, BUT BELOW UPPER 20'
INCREASE GRAVEL TO
MIN 36" BEHIND UNIT AND 48" FROM FACE

REAR SUBDRAIN:
4" (MIN) DIAMETER PERFORATED PVC PIPE
(SCHEDULE 40 OR EQUIVALENT) WITH
PERFORATIONS DOWN. SURROUNDED BY
1 CU. FT/FT OF 3/4" GRAVEL WRAPPED IN
FILTER FABRIC (MIRAFI 140N OR EQUIVALENT)

OUTLET SUBDRAINS EVERY 100 FEET, OR CLOSER,
BY TIGHTLINE TO SUITABLE PROTECTED OUTLET

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 AND LIQUID LIMIT < 40
FOR WALL HEIGHT 10 FEET OR TALLER, PLASTICITY INDEX < 6
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
FOR WALL HEIGHT > 20 FEET, REDUCE ALLOWABLE RANGE % PASSING NO. 200 SIEVE TO 0-15

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 AND UNIT WEIGHT 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

REVISED 11/16