GEOTECHNICAL REPORT

KAISER PERMANENTE

MORENO VALLEY MEDICAL CENTER
DIAGNOSTIC AND TREATMENT (D&T) BUILDING
27300 IRIS AVENUE
MORENO VALLEY, CALIFORNIA

GEOBASE, INC.

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Prepared for:

Kaiser Foundation Health Plan, Inc.
Moreno Valley, California

By:
GEOBASE, INC.
23362 Peralta Drive, Unit 4
Laguna Hills, California 92653
(949) 588-3744

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TABLE OF CONTENTS

Page

COVER PAGE I
TABLE OF CONTENTS ii

I. INTRODUCTION 1
1.1 General 1
1.2 Objectives of the Geotechnical Investigation 1
1.3 Scope of Services 2

II. PREVIOUS RELEVANT REPORT 2

III. SITE AND PROJECT DESCRIPTIONS 3
3.1 Site Description 3
3.2 Project Description 3

IV. SITE INVESTIGATION 3
4.1 Field Program 3
4.2 Laboratory Testing 6

V. GEOLOGIC SETTING 6
5.1 Regional Geology 6
5.2 Site Geology 7

VI. SUBSURFACE CONDITIONS 8
6.1 Subsoil Conditions 8
6.2 Regional Groundwater Conditions 9
6.3 Site Groundwater Conditions 9
6.4 Historic High Groundwater Level 10

VII. SEISMOLOGY 11
7.1 Regional Faulting 11
7.1.1 San Jacinto Fault – San Jacinto Valley Segment 11
7.1.2 San Andreas Fault – San Bernardino Mountains Segment 12
7.1.3 Elsinore Fault – Glen Ivy and Temecula Segments 13
7.2 Historic Earthquakes 14
7.3 Site Accelerations 15
7.3.1 Site Coordinates 15

TABLE OF CONTENTS continued...

Page

VIII. SITE DEVELOPMENT RECOMMENDATIONS continued...

7.3.2 Site Classification 15

7.3.3 Seismic Design Criteria 15

7.3.3.1 Mapped Accelerations Response Spectra 15

7.3.3.2 Seismic Design Category 16

7.3.3.3 Design Spectra Based on Mapped Parameters 16

7.3.3.4 Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Accelerations 17

7.3.3.5 Seismic Hazard Deaggregation 17

7.4 Earthquake Effects 18

7.4.1 Liquefaction 18

7.4.2 Seismically Induced Settlements 18

7.4.3 Seismically Induced Landsliding 19

7.4.4 Ground Surface Rupture 19

7.4.5 Lateral Spreading 19

7.4.6 Subsidence 20

7.4.7 Tsunamis 20

7.4.8 Seiches 20

7.4.9 Flooding 20

VIII. SITE DEVELOPMENT RECOMMENDATIONS 21

8.1 General 21

8.2 Clearing 21

8.3 Subgrade Preparation 22

8.3.1 Building Pad 22

8.3.2 Minor Structures, Walkways, Flatwork and Pavement Areas 22

8.4 Fill Placement 23

8.4.1 Preparation of Bottom of Excavations 23

8.4.2 Compaction 23

8.4.3 Fill Material 23

8.5 Drainage 24

8.6 Temporary Excavations 24

8.6.1 Unsupported Excavations 24

8.6.2 Shored Excavations 25

8.6.2.1 General 25

8.6.2.2 Lateral Earth Pressures 25

8.6.2.3 Design of Soldier Piles 26

Page

VIII. SITE DEVELOPMENT RECOMMENDATIONS continued...
8.6.2.4 Lagging 26
8.6.2.5 Anchor Design 27
8.6.2.6 Anchor Testing 27
8.6.2.7 Monitoring 28
8.7 Trench Backfill 29

IX. FOUNDATION RECOMMENDATIONS 29
9.1 General 29
9.2 Footings 29
9.2.1 Soil Bearing Pressures 29
9.2.2 Footings Adjacent to Trenches or Existing Footings 30
9.2.3 Settlement 30
9.2.4 Lateral Load Resistance 30
9.2.5 Footing Observations 31
9.3 Retaining Walls 31
9.3.1 General 31
9.3.2 Earth Pressures 31
9.3.3 Wall Backfill and Drainage 32
9.4 Minor Structures 32
9.5 Ultimate Values 32
9.6 Floor Slabs 33

X. SOIL CORROSIVITY -- IMPLICATIONS 33

XI. PAVEMENT RECOMMENDATIONS 33
11.1 Asphaltic Concrete Pavement 33
11.2 Rigid Pavement 34

XII. PLAN REVIEW, OBSERVATIONS AND TESTING 35

XIII. LIMITATIONS 35

REFERENCES 38

LIST OF TABLES

TABLE I HIGHEST GROUNDWATER LEVEL OBSERVED AT MONITORING WELLS 10

TABLE II MCE_R MAPPED ACCELERATIONS 16

TABLE III MAPPED DESIGN RESPONSE SPECTRUM 17

TABLE IV COMPACTION REQUIREMENTS 23

TABLE V Load Factors for Ultimate Design 32

TABLE VI ASPHALTIC CONCRETE PAVEMENT SECTIONS 34

TABLE VII PCC PAVEMENT SECTION 35

LIST OF APPENDICES

APPENDIX A

Figure A-1	Site Location Map
Figure A-2	Site, Boring and CPT Location Plan
Figure A-3	Site Topographic Survey Plan
Figure A-4	Existing Foundation Plan
Figure A-5	Regional Geologic Map
Figure A-6	Geologic Cross Section A-A’
Figure A-7	Geologic Cross Section B-B’
Figure A-8	Regional Fault Map
Figure A-9	Vicinity Fault Map
Figure A-10	Historical Earthquakes Map
Figure A-11	Shear Wave Velocity Profiles
Figure A-12	Liquefaction Susceptibility Map
Figure A-13	Subsidence Susceptibility Map
Figure A-14	FEMA Flood Map
Figure A-15	Earth Pressures and Tieback Geometry for Shoring
Figure A-16	Additional Lateral Earth Pressures on Shoring

APPENDIX B

Figure B-1	Explanation of Terms and Symbols
Figure B-2	Log of Boring B-1
Figure B-3	Log of Boring B-2
Figure B-4	Log of Boring B-3
Figure B-5	Log of Boring B-4
Figure B-6	Log of Boring B-5
Figure B-7	Log of Boring B-6
Figure B-8	Log of Boring B-7
Figure B-9	Log of Boring B-8
Figure B-10	Log of Boring B-9
Figure B-11	Log of Boring B-10
Figure B-12	Log of Boring B-11
Figure B-13	Log of CPT-1
Figure B-14	Log of CPT-2
Figure B-15	Log of CPT-3
Figure B-16	Log of CPT-4

APPENDIX B continued...

Figure B-17 Log of CPT-5
Figure B-18 Log of CPT-6
Figure B-19 Log of CPT-7
Figure B-20 Log of CPT-8
Figure B-21 Log of CPT-9
Figure B-22 Log of CPT-10
Figure B-23 Log of CPT-11
Figure B-24 Log of CPT-12
Figure B-25 Log of CPT-13
Figure B-26 Log of CPT-14
Figure B-27 Log of Test Pit

GEOBASE INC (June 2010)

Figure B-28 Log of Boring B-1
Figure B-29 Log of Boring B-2
Figure B-30 Log of Boring B-4
Figure B-31 Log of CPT-3

GeoVision Geophysical Services, Inc. (July 21, 2017)

APPENDIX C

Figure C-1 Summary of Laboratory Test Results
Figure C-2 HAI Laboratory Test Results Transmittal
Figure C-3 Particle-Size Analysis of Soils
Figure C-4 Particle-Size Analysis of Soils
Figure C-5 Particle-Size Analysis of Soils
Figure C-6 Particle-Size Analysis of Soils
Figure C-7 Particle-Size Analysis of Soils
Figure C-3 Particle-Size Analysis of Soils
Figure C-4 Particle-Size Analysis of Soils
Figure C-5 Particle-Size Analysis of Soils
Figure C-6 Particle-Size Analysis of Soils
Figure C-7 Particle-Size Analysis of Soils
Figure C-8 Particle-Size Analysis of Soils
Figure C-9 Particle-Size Analysis of Soils

APPENDIX C continued...

Figure C-10	Particle-Size Analysis of Soils
Figure C-11	Particle-Size Analysis of Soils
Figure C-12	Particle-Size Analysis of Soils
Figure C-13	Particle-Size Analysis of Soils
Figure C-14	Particle-Size Analysis of Soils
Figure C-15	Particle-Size Analysis of Soils
Figure C-16	Particle-Size Analysis of Soils
Figure C-17	Particle-Size Analysis of Soils
Figure C-18	Atterberg Limits
Figure C-19	Expansion Index of Soils
Figure C-20	Expansion Index of Soils
Figure C-21	Expansion Index of Soils
Figure C-22	Expansion Index of Soils
Figure C-23	Consolidation Test Results
Figure C-24	Consolidation Test Results
Figure C-25	Consolidation Test Results
Figure C-26	Consolidation Test Results
Figure C-27	Consolidation Test Results
Figure C-28	Consolidation Test Results
Figure C-29	Consolidation Test Results
Figure C-30	Consolidation Test Results
Figure C-31	Consolidation Test Results
Figure C-32	Consolidation Test Results
Figure C-33	Direct Shear Test Results
Figure C-34	Direct Shear Test Results
Figure C-35	Direct Shear Test Results
Figure C-36	Direct Shear Test Results
Figure C-37	Direct Shear Test Results
Figure C-38	Direct Shear Test Results
Figure C-39	Direct Shear Test Results
Figure C-40	Direct Shear Test Results
Figure C-41	Direct Shear Test Results
Figure C-42	Summary of Other Test Results (EI, SO4, Ch, pH & ER; MP -OMC; and R-Value)
Figure C-43	Corrosivity Series Test Results by Anaheim Test Laboratory
Figure C-44	Corrosivity Series Test Results by M.J. Schiff & Associates
Figure C-45	Laboratory Compaction Test by Modified Effort

APPENDIX C continued...

Figure C-46 Laboratory Compaction Test by Modified Effort
Figure C-47 Laboratory Compaction Test by Modified Effort
Figure C-48 Resistance R-Value by Anaheim Test Laboratory
Figure C-49 Resistance R-Value by LaBelle Marvin, Inc.

APPENDIX D

Figure D-1 Dry Seismic Settlement CPT-1
Figure D-2 Dry Seismic Settlement CPT-4

I. INTRODUCTION

1.1 General

Kaiser Foundation Health Plan, Inc. is planning the construction of a Diagnostic and Treatment (D&T) Building on the Moreno Valley Medical Center (MVMC) campus, located at 27300 Iris Avenue, in the City of Moreno Valley, California. The MVMC campus location is shown on Figure A-1, Appendix A and the proposed D&T Building location is shown on Figure A-2, Appendix A. GEOBASE, INC. (GEOBASE) was retained by Kaiser Foundation Health Plan, Inc. to complete a geotechnical investigation for the proposed D&T Building.

For this geotechnical investigation we were provided with:

- A site plan, prepared by CO Architects, showing the existing Hospital and CUP, and proposed D&T Building. This plan is reproduced herein as Figure A-2, Appendix A, Site, Boring and CPT Locations Plan.
- Topographic Survey Plan prepared by SB&O Inc. dated October 27, 2009 showing the layout of the existing buildings and site features. The location of the proposed D&T Building, borings, CPT's and geophysical survey lines have been added to this plan which is presented herein as Figure A-3, Appendix A, Site Topographic Survey Plan.
- Existing hospital foundation plan in the area where the proposed D&T Building adjoins the hospital. This plan is reproduced herein as Figure A-4, Appendix A.
- Geotechnical reports pertinent to the site (see references).

This geotechnical report incorporates results of the field and laboratory testing, and the geologic-seismic study, as required by the guidelines prepared by the Department of Conservation, California Geological Survey (CGS) and the California Office of Statewide Health and Planning Department (OSHPD). Both general and specific recommendations pertinent to suitable site development and foundation design, respectively, are provided. Construction guidelines related to the geotechnical aspects of the project are also addressed.

1.2 Objectives of the Geotechnical Investigation

The objectives of the geotechnical investigation are to obtain soil parameters and an understanding of site geologic conditions in order to provide recommendations pertinent to suitable site development and foundation design. These recommendations will assist with final design and construction of the project as planned.

1.3 Scope of Services

To achieve the objectives of the geotechnical investigation, stated above, the services provided during the course of this investigation included:

- a review of available published and unpublished geotechnical, geological and seismological reports, and maps pertinent to the site.
- Field exploration program consisting of advancing eleven (11) borings, fourteen (14) Cone Penetration Tests (CPT) and one (1) test pit;
- Logging the borings and test pit, and selection of samples representative of the materials encountered for laboratory testing;
- Field testing consisting of the Standard Penetration Test (SPT) and CPT, including shear wave velocity measurements;
- Field testing consisting of two (2) geophysical survey lines, utilizing multi–channel array surface wave (MASW) methods.
- Selection of appropriate laboratory tests and laboratory testing;
- Evaluation of data obtained from the above, and engineering analyses; and,
- Preparation of this report describing the field investigation, summarizing the results of field testing, laboratory testing and engineering analyses, and providing appropriate recommendations for site development and foundation design.

II. PREVIOUS RELEVANT REPORT

GEOBASE has completed a geotechnical investigation of the existing hospital addition and CUP for Kaiser Foundation Health Plan, Inc. The results of this investigation were presented in a report titled "Geotechnical Investigation, Kaiser Permanente MVCH, Hospital Addition and CUP, 27300 Iris Avenue, Moreno Valley, California" (GEOBASE, 2010). This report was approved by the regulating agencies and the Emergency Room Expansion was built. Relevant field boring logs, CPT's and laboratory test results of the aforementioned geotechnical investigation have been evaluated and are incorporated in this investigation as supplemental data. The locations of the pertinent borings and CPT's are shown on Figures A-2 and A-3, Appendix A. Relevant laboratory test data are presented in Appendices B and C.

III. SITE AND PROJECT DESCRIPTIONS

3.1 Site Description

The Kaiser Permanente - Moreno Valley Medical Center (MVMC) site is located on an approximately twenty (20) acre site at 27300 Iris Avenue, in the City of Moreno Valley, California. The MVMC site is bounded by medical office buildings to the east and west, Iris Avenue to the south, and an empty/vacant lot to the north. The site is gently sloping to the north and is occupied by the Hospital, the CUP, a medical office building (MOB), and at-grade parking and driveways.

3.2 Project Description

The proposed location of the D&T Building connected to the existing Hospital is shown on the Site, Boring and CPT Locations Plan, Figure A-2, Appendix A.

The east wing, at-grade portion, of the existing hospital will be demolished. The proposed D&T will be connected to the hospital at its southwest corner with finish floor elevation matching the lowest level floor elevation of the hospital at 1523.45 above-mean-sea-level (amsl). The south wall of the proposed D&T Building will retain approximately fifteen (15) feet of soil, and the height of soil retained by the east and west walls gradually decreases towards the north as the elevation changes to near at-grade along the north face of the building (Figure A-3, Appendix A).

Column loads were not available at the time of writing this report.

IV. SITE INVESTIGATION

4.1 Field Program

The field investigation for the proposed MVMC site was carried out on June 07, 08, 09 and 22, 2017 by advancing eleven (11) borings using a truck-mounted CME-75 drill rig fitted with hollow-stem augers, fourteen (14) CPT's and one (1) test pit. The borings, CPT's and test pit were located in the field by utilizing a Trumeter 550SE (roll-a-tape) and elevations were estimated from Site, Boring, CPT Locations Plan and Site Topographic Survey Plan (Figures A-2 and A-3, respectively, Appendix A). Therefore, the locations and elevations should be considered accurate only to the degree implied by the methods used.

Geophysical survey lines, utilizing multi-channel array surface wave (MASW) methods, were conducted by GeoVision Geophysical Services, Inc. on July 10, 2017.

Four (4) borings (B-2 thru B-5, inclusive) and two (2) CPT's (CPT-1 and CPT-2) advanced during this investigation are considered relevant to the proposed D&T Building. All borings and CPT's were advanced to maximum penetration depths of seventy-one and one-half (71.5) feet and seventy-five (75) feet, respectively, except for CPT-2 and CPT-5 locations where refusal was obtained at shallow depths. In this respect, the test pit was excavated at CPT-5 location and advanced beyond the depth at which refusal was obtained to confirm that refusal was due to a hard soil layer. Two (2) seismic CPT's (SCPT-4 and SCPT-12) were advanced to a depth of 100 feet to determine shear wave velocities of the subsoils. All borings were hand-augered in the upper five (5) feet.

The Log of Borings, together with the Explanation of Terms and Symbols used are shown on Figures B-1 thru B-12, inclusive, CPT plots are presented on Figures B-13 thru B-26, inclusive, and the Log of Test Pit on Figure B-27, Appendix B. Relevant borings and CPT's from a previous investigation (GEOBASE, 2010) are presented herein as Figures B-28 thru B-31, inclusive, Appendix B.

Field testing consisted of: Standard Penetration Test (SPT); Cone Penetration Tests (CPT's), including Seismic Cone Penetration Testing at two (2) CPT locations (SCPT-4 and SCPT-12) to determine the shear wave velocities of the subsoils; and, geophysical survey lines to determine shear wave velocities of the subsoils.

- The SPT test (ASTM D 1586) involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, two (2) inches outside diameter and one and three-eights (1-3/8) inches inside diameter, is driven eighteen (18) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value, or SPT blow count. The driving energy is provided by a 140-pound weight dropping thirty (30) inches.

- The Cone Penetration Tests (CPT's) were performed in accordance with ASTM D 3441. The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil, and a continuous record of cone tip resistance, friction resistance and pore water pressures versus depth is obtained in digital form at the ground surface. A specially designed truck is used to transport and house the test equipment and to provide a ten (10) ton reaction to the thrust of the hydraulic rams. Near-continuous CPT records provide: approximate correlations with soil classification; relatively accurate definition of the thickness of various soil layers; subsoils data for liquefaction and seismic settlement analyses; and, engineering properties of the subsoils for static settlement analyses.

- Shear wave velocity measurements were carried out at five (5) foot intervals at two (2) CPT locations, SCPT-4 and SCPT-12.

- Two (2) geophysical survey lines utilizing multi-channel array surface wave (MASW) methods were completed to obtain the shear wave velocity profile of the subsoils. A discussion of field procedures, geophysical techniques, data processing and interpretation, and the results of the geophysical survey are given in Appendix B.

Sampling consisted of:

- Collection of bulk samples at selected locations retrieved from the auger;

- Collection of samples retrieved from the Standard Penetration Test (SPT) split spoon sampler; and,

- Collection of soil samples at selected locations using a Modified California Sampler. The soil samples were retained in a series of brass rings, each having an inside diameter of 2.41 inches and a height of one (1) inch. These ring samples were placed in close- fitting, moisture-tight containers for shipment to the laboratory.

4.2 Laboratory Testing

The samples obtained during the field program were returned to the laboratory for visual examination and testing. The soils were classified in accordance with ASTM D 2487 and D 2488.

The laboratory testing program consisted of the following:

- Laboratory determination of water (moisture) content of soils, rock, and soil-aggregate mixtures (ASTM D 2216), and dry density (ASTM D 2937);
- Particle size analysis of soils (ASTM D 422);
- Standard test methods for amount of material in soils finer than the No. 200 Sieve (ASTM D 1140); and,
- Atterberg Limits (ASTM D 4318);
- Direct shear test of soils (ASTM D 3080);
- Consolidation tests (ASTM D 2435);
- Maximum dry density and optimum moisture content (ASTM D 1557);
- Expansion potential of soils (ASTM D 4829);
- Resistance R-Value (CT 301); and,
- Water soluble sulfate content of soils (CT 417); pH and electrical resistivity (CT 643); and water soluble chlorides (CT 422).

The laboratory test results from this investigation and previous investigation (GEOBASE, 2010) are presented on the Log of Borings, Figures B-2 thru B-12, inclusive, and B-28 and B-29, Appendix B, where applicable and in Appendix C.

V. GEOLOGIC SETTING

5.1 Regional Geology

The MVMC site is located in the Northern portion of the Peninsular Ranges Physiographic Province of California on a structural unit known as the Perris Block (CGS, 2002). The Perris

Block is bounded on the northeast by the San Jacinto Fault Zone, on the southwest by the Elsinore Fault Zone, and on the north by the Cucamonga Fault Zone. The southern boundary of the Perris Block is not as distinct, but is believed to coincide with a complex group of faults trending southeast from the Murrieta, California area (Kennedy, 1977 and Mann, 1955). The Peninsular Ranges are characterized by northwest trending elongated alluvial valleys and by elevated Mesozoic age intrusive rock masses of the California batholith, flanked by metavolcanic and metasedimentary rocks that form the mountainous portions of the province. Various thicknesses of alluvial sediments derived from the erosion of the elevated portions of the region fill the low-lying areas such as the Moreno Valley where the site is located. According to Morton and Matti (2001), the sediments that infill the Moreno Valley have been differentiated into Holocene and late Pleistocene age young alluvial fan and alluvial valley deposits and into very old alluvial fan deposits of early Pleistocene age. Maximum depths of valley fill in the area are reported to reach approximately 900 feet in the western and northern portions of the San Jacinto Groundwater Basin, where the site is located, but may exceed 5,000 feet in the eastern part of the same basin between the Casa Loma and Claremont faults (CDWR, 2006). Morton and Matti (2001) indicate that the young alluvial fan and valley deposits consist predominantly of sandy materials with silty, gravelly and cobbly interbeds. The very old alluvial fan deposits are reported to consist of mostly well-dissected, well-indurated sand deposits that typically flank the bedrock outcrops in the immediate vicinity. Very old alluvium underlies the subject site whereas Cretaceous age quartz diorite constitutes the hilly areas of the Perris State Recreational area to the south. The alluvial sequence at the site is inferred to rest unconformably on Cretaceous age crystalline bedrock. Figure A-5, Appendix A, presents the Regional Geology Map.

5.2 Site Geology

The MVMC is located near the foothills of the mountains that constitute the Perris State Recreational area to the south. The site is located at an approximate elevation of 1,530 feet above mean sea level (amsl) on a gently northwest sloping surface that grades down towards the Moreno Valley (Figures A-1 and A-5, Appendix A). Drainage at the site area is presently controlled by storm run-off sewers, street and/or natural drainages.

GEOBASE advanced four (4) exploratory soil borings and three (3) cone penetration tests (CPT's) at the site in 2010, and an additional eleven (11) borings, fourteen (14) CPT's and one (1) test pit in June 2017 (Figure A-2, Appendix A, Site, Boring and CPT Locations Plan). Soil borings were drilled to a maximum depth of seventy-one and one-half (71.5) feet, whereas the CPT's had a total depth that ranged up to 100 feet.

All the soil borings and CPT's advanced by GEOBASE to a maximum depth of seventy-one and one-half (71.5) and 100 feet below ground surface (bgs), respectively, confirm that the site is underlain by unconsolidated Quaternary alluvial fan deposits covered by a thin mantle of man-made fill (Figures B-2 thru B-31, inclusive, Appendix B). The man-made fill materials consist of approximately up to eight (8.0) feet of predominantly brown, silty sands (SM) at the boring locations. The unconsolidated alluvium consists predominantly of medium-grained brown silty sands with a five (5.0) to ten (10.0) foot thick orange to brown, silt (ML) interbed in the upper twenty-five (25) feet. This silt (ML) interbed was not encountered at soil boring location B-4. The density of the alluvial materials at the site generally increases with depth. Unconsolidated alluvial materials were encountered to the total depth of penetration of all the soil borings that have been advanced at the site.

Our interpreted surface distribution of geologic materials encountered during the site investigations is illustrated in Figure A-2, Appendix A. Geologic Sections A-A' and B-B' across the D&T Building site are given on Figures A-6 and A-7, Appendix A, respectively.

VI. SUBSURFACE CONDITIONS

6.1 Subsoil Conditions

At the boring and CPT locations within paved areas, the pavement section consisted of approximately four (4) to six (6) inches of asphaltic concrete overlying approximately four (4) to five (5) inches of aggregate base.

The generalized stratigraphic profile at the boring locations relevant to the D&T Building consisted of up to six (6) feet of fill soils overlying native silty sands and sands with traces of gravel to the maximum depth of exploration, seventy-one and one-half (71.5) feet. The fill soils may be thicker at other locations. Unless a compaction report is made available, these fills are considered "undocumented fills". Notwithstanding the preceding, SPT test results and CPT data indicate that the existing fills possess a "very loose" to "medium dense" consistency. A five (5) to eleven (11) foot thick silt layer was also encountered at varying depths in the upper twenty-five (25) feet, except at boring B-2 location. At boring B-3 location, a silt layer was also observed at a depth of fifty-five (55) to sixty (60) feet below ground surface.

The SPT test results and CPT data indicate that the native silty sands can be generally inferred to be in a "dense" to "very dense" state; however, very loose silts and silty sands were encountered at shallow depths. The native sandy silts are inferred to have a “stiff” to “very hard” consistency.

The silty samples tested showed non-plastic behavior, and the soil natural moisture contents ranged from three (3) to thirteen (13) percent, with the higher values measured in the siltier samples. Expansion potential of the samples tested showed “very low” potential for expansion(Expansion Indices = 8 at D&T Building location; and, 0 to 12 at the MVMC site).

6.2 Regional Groundwater Conditions

The MVMC site is located in the western portion of the San Jacinto Groundwater Basin. The San Jacinto Groundwater Basin underlies San Jacinto, Perris, Moreno, and Menifee Valleys in western Riverside County. This basin is bounded by the San Jacinto Mountains on the east, the San Timoteo Badlands on the northeast, the Box Mountains on the north, the Santa Rosa Hills and Bell Mountain on the south, and unnamed hills on the west. The valleys are drained by the San Jacinto River and its tributaries.

According to the CDWR (2006), groundwater in the western portion of the San Jacinto Basin occurs under confined conditions. The primarily source of recharge for the confined aquifers is found where the San Jacinto River and the Baustita Creek enter the San Jacinto Valley CDWR (2006). Percolation of water stored in Lake Perris has been an additional source of recharge along with reclaimed water percolation by means of storage ponds administered by Eastern Municipal Water District.

6.3 Site Groundwater Conditions

During our exploratory investigations, groundwater was not encountered to the maximum depth of boring penetration, seventy-one and one-half (71.5) feet. The exploratory soil borings drilled by GEOBASE at the MVMC site did not encounter groundwater; that is in general agreement with the conditions reported by the CDWR (2017).

6.4 Historic High Groundwater Level

Historical groundwater level data was obtained online from the Water Data Library operated by the CDWR (2017). There are five (5) monitoring wells within a two (2) kilometer radius of the site. Monitoring well locations are shown on Figure A-5, Appendix A, and pertinent data is summarized in Table I, below.

TABLE I
HIGHEST GROUNDWATER LEVEL OBSERVED AT MONITORING WELLS

Point	Well No.	Period of Measurements	Date of Highest Recorded Groundwater (mm/dd/yr)	Highest Recorded Groundwater Below Existing Grade (ft.)	Ground Elevation* (ft.)	Groundwater Elevation Above Mean Sea Level (ft)
1	EMWD12077	10/04/2011 to 04/11/2017	04/11/2017	34.9	1507.4	1472.5
2	EMWD25696	11/07/2011 to 04/11/2017	04/11/2017	41.0	1506.2	1465.2
3	EMWD25695	11/07/2011 to 04/11/2017	04/11/2017	44.5	1507.4	1462.9
4	EMWD10141	11/03/2011 to 04/11/2017	04/07/2017	59.8	1545.8	1486.0
5	03S03W15F001S	05/29/1951 to 09/15/1986	04/01/1952	99.8	1539.0	1439.2

* Existing Ground Surface Elevation at the Well Location

Reference : California Department of Water Resources (CDWR); http://www.well.water.ca.gov/cgi-shl/gwater.

Groundwater level reading for water well number EMWD12077 are available for the time period of 2011 to 2017. Ground surface elevation for this well is reported to be 1,507.4 feet above mean sea level (amsl), whereas the approximate elevation for the MVMC site was estimated at 1,530 feet amsl (an approximate difference in elevation of 23 feet). The shallowest ground water level condition of 1,472.5 feet amsl (depth of 34.9 below ground surface [bgs]) at this well occurred on April 11, 2017. Therefore, it can be concluded that the MVMC site is located on a confined aquifer that appears to have been recharged since 2014. No historical groundwater data is available prior to 2011. Well number 03S03W115F001S has historical data dating back to 1951. Unfortunately, the data ends in 1986.

Projecting the higher groundwater elevation noted above across the MVMC site, the highest groundwater elevation is obtained to be at is approximately fifty-eight (58) feet bgs based on current well data. For design purposes, historic highest groundwater level in excess of fifty (50) feet bgs shall be considered for the site.

VII. SEISMOLOGY

7.1 Regional Faulting

The two principal seismic considerations for most properties in Southern California are ground surface rupture along fault traces and damage to structures due to seismically induced ground shaking. The fault classification system adopted by the California Geological Survey (CGS), relative to the State legislation, delineates Earthquake Fault Zones along active or potentially active faults (Alquist-Priolo Act). Such Earthquake Fault Zones are in turn used to establish setbacks of structures from active fault zones. An active fault is defined by the CGS as a "sufficiently active and well defined fault" that has exhibited surface displacement within Holocene time (approximately the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago). Any fault proven not to have moved within the last 1.6 million years is considered inactive.

The closest known active faults to the site are the San Jacinto, San Andreas and Elsinore faults. A California Fault Map, showing the geographic relationship of these faults to the site is presented as Figures A-8 and A-9, Appendix A. A brief description of these faults is provided below.

7.1.1 *San Jacinto Fault – San Jacinto Valley Segment*

The San Jacinto Fault is one of the most active faults in California, having been an important source of moderate- to large-magnitude earthquakes during this century. What makes the San Jacinto Fault of extreme interest to scientists and state building engineers is that the fault is remarkably long and has a potential of hundreds of kilometers of rupture length, thus creating larger magnitude earthquakes and potentially affecting larger areas. This fault, over approximately 210 kilometers in total length, extends to the southern border of California and joins the San Andreas Fault west of the city of San Bernardino. The sense of movement is right-lateral strike-slip. According to the Southern California Earthquake Center (SCEC,1995), slip is regularly released on this fault in the form of small earthquakes (M_L 3 and 4). Historically, this fault has experienced numerous medium sized earthquakes (M_L of upper 4's and 5's) and several large earthquakes (larger than M_L 6). In the early 1900s large earthquakes in the Hemet and San Jacinto areas produced surface rupture. Using information on fault geometry, historical seismicity, and slip-rate data, Petersen et al (1996) divided this fault into eight segments. These segments, from north to south are: San Bernardino Valley, San Jacinto Valley, Anza, Coyote Creek, Borrego Mountain, Superstition Hills, Superstition Mountains, and Imperial.

The closest active fault segment of the San Jacinto Fault to the MVMC site is the northwest-trending, right-lateral strike-slip San Jacinto Valley fault segment, located approximately 4.8 kilometers (km) to the northeast of the site. The San Jacinto Valley fault segment extends approximately 43.0 km from the northern end of the San Jacinto Valley to the junction of the Claremont and Casa Loma faults to the south.

The San Jacinto Valley segment may have been the source of the December 25, 1899 and April 21, 1918 earthquakes with magnitudes of 6.4 and 6.8 that occurred on the Casa Loma and Claremont faults, respectively (SCEC, 1995 and Treiman and Lundbergh, 1999). Petersen et al (1996) and SCEC (1995) assigned a slip-rate of 12+/- 6 millimeters/year (mm/yr), a M_W 6.9 and a recurrence interval of sixty-five (65) to ninety-eight (98) years. Similarly, the estimate of characteristics displacement was assigned at 1.0 +/- 0.2 meters (m).

7.1.2 *San Andreas Fault – San Bernardino Mountains Segment*

The San Andreas Fault extends for several hundred miles from the Gulf of California in the south to Cape Mendocino in northern California and it is the main element of the boundary between the Pacific and North American tectonic plates. The San Andreas Fault extends as a continuous trace from Cape Mendocino to San Bernardino, bends eastward, and continues southeast near Indio. The central and southern San Andreas Fault was divided by SCEC (1995) and Petersen et al (1996) into the following five (5) fault segments: Cholame, Carrizo, Mojave, San Bernardino Mountains, and Coachella Valley. It is important to emphasize that although these segments are treated as independent sources of earthquakes, historical and paleoseismological observations show that ruptures may overlap and that some segments may both produce their own earthquakes and fail when large ruptures nucleate in an adjacent segment and propagate into them. The fault segments are composed of numerous subparallel right-lateral, strike-slip faults that range from 0.5 to 11 km in length. The Fort Tejon earthquake of approximately Mw 8, one of the greatest earthquakes ever recorded in the United States, occurred along the San Andreas Fault in January 9, 1857 and produced a surface rupture of approximately 350 km in length from

Cholame on the north to the Cajon Pass on the south.

The closest significant San Andreas Fault segment to the MVMC site is the northwest-trending, right-lateral strike-slip San Bernardino Mountains segment, located approximately 23.7 km to the northeast of the site. The San Bernardino Mountains segment is approximately 103 km long and extends from a few kilometers northwest of Cajon Creek southeast to the area between Thousand Palms and Myoma. The San Bernardino Mountains segment is characterized by a large left-restraining step between the Mojave segment to the northwest and the Coachella segment to the southeast. The San Andreas Fault Zone is very complex in this restraining step, consisting of dextral strike-slip, thrust, and oblique slip faults (Bryant and Lundbergh, 2002). According to the SCEC (1995), the past five ground surface rupture events at Wrightwood occurred approximately in 1812, 1693, 1587, 1452, and 1192 of the current era. In addition, displacements of 4 m during the 1812 event, and a cumulative offset of 7 to 8 m of right slip for the 1812 and 1693 earthquakes, have been measured in the Cajon Pass area. Therefore, based on paleoseismic studies, the San Bernardino Mountains segment is believed to have last ruptured in 1812. The Wrightwood site has averaged one surface-rupturing earthquake every 124 years since 1192. The most recent three events have been closer together, averaging 112 years between events.

Petersen et al (1996) and the SCEC (1995) assigned a slip rate of 24+/- 6 mm/yr, a M_w 7.5, and a recurrence interval of 14 (+91, -60) years to this segment.

7.1.3 *Elsinore Fault – Glen Ivy and Temecula Segments*

The Elsinore fault zone forms the northeast boundary of the Santa Ana Mountains and extends nearly 200 km from Whittier to the Mexican border. Individual segments within the Los Angeles region are three (3) to forty (40) km long and display reverse right oblique, right-lateral strike-slip, and normal-right-oblique-slip late Quaternary or Holocene offsets. Petersen et al (1996) divided this fault into six segments which from north to south are: Whittier, Glen Ivy, Temecula, Julian, Coyote Mountain, and Laguna Salada. In addition, several of the fault segments possess locally their own names. For example, the Glen Ivy North and Glen Ivy South branches are located Northwest of Lake Elsinore. Heading southeast from Lake Elsinore, the two parallel fault strands are denominated Wildomar Fault (the more easterly) and Willard Fault. At its northern end, the Glen Ivy segment splays into two (2) fault segments, the Chino – Central Avenue and the Whittier faults.

The closest significant Elsinore Fault segments to the MVMC site are the northwest-trending, right-lateral strike-slip Glen Ivy and Temecula segments, located approximately 32.1 km to the southwest of the site.

The Glen Ivy fault segment extends for approximately 38 km. According to the SCEC (1995), this segment at Glen Ivy marsh shows that five (5) and probably six (6) earthquakes have disrupted the sediments there since approximately 1060, yielding an average recurrence interval of 150 to 200 years. These events occurred in 1910, post-1660, 1360 to 1660, about 1300, 1260, and about 1060. The most recent surface rupture is associated with the 1910 Temescal Valley earthquake with an estimated magnitude MW6.0 (Ziony and Jones, 1989). The surface displacement in this event was approximately 250 to 300 millimeters (mm). This fault segment has been assigned a probable MW6.8 with a slip rate of 5 mm/yr and a recurrence interval of 340 years (Petersen et al, 1996).

The Temecula Fault segment extends for approximately 62 km. Trenching across the Wildomar Fault in the Temecula segment has yielded a late Holocene slip rate for the principal strand. A fluvial channel, dated by C-14 at about 2000 to 2400 years, is laterally displaced approximately 10+/- 1 m and yields a slip rate of about 4.2 mm/yr (SCEC, 1995). This rate is considered as minimum since several minor strands of the fault also have a geomorphic expression. Nevertheless, it is similar to the rates determined at other locations along the Elsinore Fault. SCEC (1995) concluded a maximum average recurrence interval of between 250 and 600 years and a slip rate of 5.0+/- 2.0 mm/yr for this segment. Because no measurements of characteristic displacements are available, SCEC (1995) calculated a value of 1.2+/- 0.3 m using the segment length and empirical relations postulated by Wells and Coppersmith in 1994. According to SCEC (1995), this yields an average recurrence interval of 240 (+260, -111) years.

7.2 Historic Earthquakes

A map of recorded earthquake epicenters is provided as Figure A-10, Appendix A. This map can be accessed online by the Southern California Earthquake Data Center at Cal Tech. The Southern California Earthquake Data Center identifies three major earthquakes magnitude 6.0 or greater that have occurred on the San Jacinto fault since 1899, within a fifty (50) mile radius of the subject site: North San Jacinto Fault Earthquake near Loma Linda occurred July 22, 1923 with a magnitude of 6.3; the San Jacinto Earthquake just east of Hemet occurred April 21, 1918 with a magnitude of 6.8; and, the San Jacinto Fault (Terwilliger Valley) Earthquake also known as the Borrego Springs Fault, occurred in 1937 with a magnitude of 6.0.

The only large historical earthquake that can be attributed to the Elsinore Fault is a magnitude 6.0 that occurred in 1910 in the Temescal Valley area.

Four (4) other earthquakes of magnitude 4.0 or greater are identified within this fifty (50) mile radius: the Anza Gap Earthquake M 4.8; the White Wash Earthquake east of Anza occurred on February 25, 1980, M 5.5; the Chino Hills Earthquake in 2008, M 5.4; and, the Upland Earthquake of 1990, M 5.4.

7.3 Site Accelerations

7.3.1 *Site Coordinates*

The site latitude and longitude are 33.898 degrees north and 117.186 degrees west, respectively.

7.3.2 *Site Classification*

The site classification procedure recommended by CBC 2016, subsection 1613A.3.2, which references ASCE 7-10, Chapter 20, was adhered to.

The Cone Penetration Tests (CPT's) and geophysical surveys results provided measured average shear wave velocities at a minimum 402 m/s within the top 100 feet. The shear wave velocity profiles of the CPT's and geophysical surveys presented on Figure A-11, Appendix A, show good correlation. Based on the aforementioned measured shear wave velocities, to develop seismic design criteria, the site subsoils within the top 100 feet are judged to be Site Class C.

7.3.3 *Seismic Design Criteria*

Based on CBC 2016, subsection 1616A.1.3, which references and modifies ASCE 7-10, subsection 11.4.7, since the structure is assigned to Seismic Design Category D and S_1 is less than 0.75g (see subsection 7.3.3.2), a site-specific GMHA was not completed. The following subsections present the seismic design parameters based on mapped parameters.

7.3.3.1 Mapped Accelerations Response Spectra

Mapped, risk-targeted maximum considered earthquake, MCE_R, spectral response accelerations for 0.2 and 1.0 second periods are provided in maps published in the ASCE 7-10, which is the reference used in the CBC 2016. These maps are prepared by the USGS and the California portion of the map was prepared jointly with the CGS. These maps use results of seismic hazard analyses from both probabilistic and deterministic procedures, and are applicable to Site Class B and five (5) percent of critical damping. The mapped site accelerations are adjusted for site class effects using parameters Fa and Fv, which are functions of site class and mapped site spectral accelerations.

The mapped design horizontal spectral accelerations were evaluated in accordance with ASCE 7-10, using the US Seismic Design Maps Application (USGS, 2017) available at the USGS website: http://geohazards.gov/designmaps/us/application.php. This web application requires the inputs of site location (coordinates) and site soil classification.

The project site is Site Class C and coefficient values Fa and Fv of 1.0 and 1.3, respectively, are obtained for the site. Mapped MCE_R accelerations obtained for the project site are summarized in Table II, below.

TABLE II

MCE_R MAPPED ACCELERATIONS

PERIOD (SECONDS)	MAPPED ACCELERATION PARAMETERS (g)	Site Class C	
		MCE_R ACCELERATIONS ADJUSTED FOR SITE CLASS EFFECTS (g)	RISK COEFFICIENTS
0.2	S_s: 1.673	1.673	C_{RS} = 1.008
1.0	S_1: 0.729	0.948	C_{R1} = 0.976

Based on Table II, the mapped spectral response accelerations, adjusted for Site Class C, S_{MS} and S_{M1} are 1.673g and 0.948g, respectively.

7.3.3.2 Seismic Design Category

The mapped spectral response acceleration parameter at one (1) second period (S_1) is 0.729g which is less than 0.75g. The design spectral response acceleration coefficients S_{DS} and S_{D1} are 1.115 and 0.632g, respectively. Therefore, a Seismic Design Category D should be used for the design of the proposed structure per Section 1613A.3.5 of CBC 2016.

7.3.3.3 Design Spectra Based on Mapped Parameters

Section 11.4.5 of ASCE 7-10 describes a procedure to obtain a design response spectra curve for use in cases where a design response spectrum is required by the ASCE 7-10 standard, and site-specific ground motion procedures are not used. This procedure is based on the use of the mapped spectral response accelerations adjusted for site class effects in the determination of the design response spectra curve. Using this procedure, numerical values of the design spectral response accelerations based on the mapped parameters for the project site are provided in Table III, below.

TABLE III

MAPPED DESIGN RESPONSE SPECTRUM

Period (Seconds)	Mapped Design Spectral Response Acceleration (g)
0.00	0.446
0.113	1.115
0.20 (S_{DS})	1.115
0.500	1.115
0.566	1.115
0.700	0.903
0.800	0.790
0.900	0.702
1.00 (S_{D1})	0.632
2.00	0.316
3.00	0.211
4.00	0.158
5.00	0.126

7.3.3.4 Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Accelerations

From Figure 22-7 of ASCE 7-10, PGA = 0.657g is multiplied by the site coefficient F_{PGA} = 1.0 (Table 11.8-1) to obtain the mapped MCE Geometric Mean Peak Ground Acceleration (PGA_M). For Site Class C, $PGA_M = F_{PGA}$ x PGA. Therefore, PGA_M = 0.657 may be used for evaluation of liquefaction, lateral spreading, seismic settlement and soil-related issues.

7.3.3.5 Seismic Hazard Deaggregation

Relative contributions of various combinations of earthquake magnitudes and distances to a particular seismic hazard at a site are determined using deaggregation of the seismic hazards. Magnitude-distance deaggregation, obtained from the Unified Hazard Tool “Dynamic: Conterminous US 2008 (V.3.3.1)” edition that is available on the USGS website, indicates that the deaggregated mode magnitude and distance for the peak ground acceleration at the project site are M7.5 and 7.0 kilometers, respectively.

7.4 Earthquake Effects

7.4.1 *Liquefaction*

Liquefaction occurs when the pore pressures generated within a soil mass equals the overburden pressure. This results in a loss of strength and the soil then possesses a certain degree of mobility.

Factors considered to evaluate liquefaction potential include groundwater conditions, soil type, particle size distribution, earthquake magnitude and acceleration, and soil density obtained through the Standard Penetration Test (SPT) or Cone Penetration Test (CPT). Soils subject to liquefaction comprise saturated fine-grained sands to low-plasticity silts and clays. Coarser-grained soils are considered free-draining and therefore dissipate excess pore pressures, while fine-grained soils possess undrained shear strength and are therefore less subject to liquefaction.

The liquefaction susceptibility map, Figure A-12, Appendix A, of the County of Riverside General Plan, indicates that the project site is located in an area that is subject to "low" liquefaction potential. Furthermore, the subsoils are considered "dense" to "very dense" or "stiff" to "hard" with a historic highest groundwater table at a depth greater than fifty (50) feet; therefore, the site is 11considered to possess a "very low" potential for liquefaction.

7.4.2 *Seismically Induced Settlements*

Based on an examination of the subsoils conditions, seismic settlement analyses were conducted at CPT-1 and CPT-4 locations. For these analyses, a PGA_M of 0.657g and an earthquake magnitude of 7.5 based on the deaggregation results were used. Seismic settlements for the unsaturated cohesionless soils were estimated using the Tokimatsu and Seed (1987) Method. The results of the seismic settlement analyses are provided in Appendix D.

Based on our evaluation of the analyses results at the CPT locations, seismically induced settlements at the site are not anticipated to exceed one-half (0.5) inch for the D&T Building.

7.4.3 *Seismically Induced Landsliding*

Due to the relatively flat existing topographic conditions, the MVMC site is not located within a designated area where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacement such that mitigation would be required (RCIT, 2017). In addition, based on our field reconnaissance and field investigations, there are no known landslides near or at the MVMC site, nor is the site on the path of any known or potential landslides.

7.4.4 *Ground Surface Rupture*

Ground surface displacement along a fault, although more limited in area than the ground shaking associated with it, can have disastrous consequences when structures are located straddling the fault or near the fault zone. Fault displacement involves forces so great that in most cases it is not practically feasible (structurally or economically) to design and build structures to accommodate rapid displacement and remain intact. Amounts of movement during a single earthquake can range from several inches to tens of feet. Another aspect of fault displacement comes not from the violent movement associated with earthquakes, but the barely perceptible movement along a fault called "fault creep". Damage by fault creep is usually expressed by the rupture or bending of buildings, fences, railroad tracks, streets, pipelines, curbs, and other linear features.

No faulting was observed during our field reconnaissance. In addition, active, potentially active, and other major inactive faults noted on regional geologic and fault maps do not cross nor project toward the site. Furthermore, the site is not located within any APEQFZ Map as designated by the CGS (Bryant and Hart, 2007; CDMG, 2000 and CGS, 2017). The County of Riverside (RCIT, 2017) and the USGS (2017) indicate that the closest active fault to the site is the San Sacinto Fault Zone located approximately 4.8 km to the northeast. Cracking due to shaking from distant events is not considered a significant hazard, although it is a possibility at any site.

7.4.5 *Lateral Spreading*

Seismically induced lateral spreading involves primarily movement of earth materials due to ground shaking. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Such spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shear stress biases on essentially horizontal ground. The potential for liquefaction at the site is considered very low. Therefore, the potential for lateral spreading of the subject site is very low.

7.4.6 *Subsidence*

Subsidence refers to the sudden sinking or gradual downward settling and compaction of soil and other surface material with little or no horizontal motion. It may be caused by a variety of human and natural activities, including changes in groundwater level, soil moisture and earthquakes. Alluvial valley regions are especially susceptible and according to RCIT (2017), the site is located within an area that is susceptible to subsidence (Figure A-13, Appendix A) .

7.4.7 *Tsunamis*

A tsunami is a sea wave generated by a submarine earthquake, landslide, or volcanic event. The MVMC is not located within a coastal area; instead, it is located several tens of miles inland from the Pacific Ocean at an approximate elevation of 1525 feet amsl (GoogleEarth, 2017). Therefore, a tsunami hazard at the property is considered negligible.

7.4.8 *Seiches*

A seiche is an earthquake-induced wave in a confined body of water, such as a lake, reservoir, or bay. Resulting oscillations could cause waves up to tens of feet high, which in turn could cause extensive damage along the shoreline. The most serious consequence of a seiche would be the overtopping and failure of a dam. Based on Figure 5.5-2, Floodplains and High Fire Hazard Areas, included in the Moreno Valley General Plan (2006), the site is not located downstream of any large bodies of water that could adversely affect the site in the event of earthquake-induced failures or seiches.

7.4.9 *Flooding*

According to the Federal Emergency Management Agency (FEMA, 2017) flood map

06065C0770G, Figure A-14, Appendix A, the City of Moreno Valley (2006a) and RCIT (2017), the MVMC is located within a “Zone X”, which corresponds to an area determined to be outside of a 0.2 percent annual chance of floodplain (FEMA, 2017).

It should be noted that the northwestern corner of the property is located within “Zone A”, which corresponds to a 1.0 percent annual chance of flood hazard (FEMA, 2017), areas of flooding sensitivity (RCIT, 2017) and a 100-year flood plain (City of Moreno Valley, 2006a). The extent of the affected area varies according to the different agencies.

VIII. SITE DEVELOPMENT RECOMMENDATIONS

8.1 General

The proposed development, described in subsection 3.2, is feasible from a geotechnical engineering standpoint. Project plans and specifications should take into account the appropriate geotechnical features of the site and conform to the geotechnical recommendations.

8.2 Clearing

All surface vegetation, asphaltic concrete, trash, debris, underground pipes, and concrete pieces after demolishing the existing structures should be cleared and removed from the proposed site. Topsoil and soils with organic inclusions are ***not*** considered suitable for reuse as structural fill, but may be stockpiled for future use in landscape areas.

Underground facilities such as utilities, pipes or underground storage tanks may exist at the site. Removal of underground tanks is subject to state law as regulated by County or City Health and/or Fire Department agencies. If storage tanks containing hazardous or unknown substances are encountered, the proper authorities must be notified prior to any attempts at removing such objects.
Septic tanks should be removed in their entirety. Cesspools or seepage pits should be pumped of their contents and backfilled with a minimum two-sack sand-cement slurry. Any water wells, if encountered during construction, should be exposed and capped in accordance with the requirements of the regulating agencies.

Depressions resulting from the removal of buried obstructions, existing building foundations, tunnels and pipes should be backfilled with properly compacted material.

8.3 Subgrade Preparation

8.3.1 *Building Pad*

In the D&T Building area, undocumented fills and "very loose" to "medium dense" silty sands to sandy silts layers were observed at the boring locations and can be observed on the data from relevant CPT's as well. These materials are not suitable for structural support and they extend to approximate elevations 1518 to 1515 amsl, as shown on Figures A-6 and A-7, Appendix A. These materials may also extend deeper at other locations and, where encountered, should be removed and replaced as properly compacted fill. Notwithstanding the aforementioned, a compacted fill blanket, a minimum of five (5) feet in thickness, should be constructed below the footing bottoms. The lateral extent of overexcavation beyond the footing limits should be at least equal to the depth of fill; however, where the D&T Building adjoins the existing hospital, understood to be supported on piles (Figure A-4, Appendix A), lateral extent of overexcavation will be limited to the existing hospital.

Exposed bottoms of overexcavation should be observed by GEOBASE to verify the removal of all unsuitable materials.

8.3.2 *Minor Structures, Walkways, Flatwork and Pavement Areas*

In order to minimize the potential for excessive settlement of minor structures which are structurally separated from the D&T Building, the footing subgrade areas should be over excavated to provide a uniform compacted fill blanket a minimum three (3) feet in thickness below adjacent grade, or at least two (2) feet below footing bottoms, whichever is greater. The lateral extent of removal beyond the footing limits should be equal to at least the depth of overexcavation. The fill should be compacted to a minimum of ninety (90) percent relative compaction (ASTM D 1557).

The subsoils within the concrete walkways, flatwork and parking areas, and within two (2) feet of their proposed limits, should be over excavated at least two (2) feet and replaced as properly compacted fills.

The above subgrade preparation recommendations may only be considered if future maintenance as a result of settlement of underlying undocumented fills can be tolerated. Alternatively, all undocumented fills should be removed and replaced as properly compacted fills.

8.4 Fill Placement

8.4.1 *Preparation of Bottom of Excavations*

Prior to placing any fill, the exposed soils at the bottom of excavations should be scarified to a minimum depth of six (6) to eight (8) inches, moisture conditioned (wetted or dried) to at least optimum moisture content and compacted to a minimum of ninety (90) percent relative compaction, based on ASTM D1557.

8.4.2 *Compaction*

Cohesive soils should be placed in loose lifts not exceeding six (6) inches, moisture-conditioned to approximately two (2) to four (4) percentage points above optimum, and compacted to the minimum relative compaction listed in Table IV below.

Granular fill materials should be placed in loose lifts of six (6) to eight (8) inches, moisture-conditioned to near optimum, and compacted to the minimum relative compaction listed in Table IV.

TABLE IV

COMPACTION REQUIREMENTS

Type of Fill/Area	Relative Compaction (ASTM D1557) Minimum Percent
Fills within building pad area	95
All other structural fill	90

8.4.3 *Fill Material*

The upper ten (10) feet of on-site soils are predominantly "very low" expansive soils (EI = 0-12). These soils may be reused as compacted fill provided they are free of organics, deletious materials, debris and particles over six (6) inches in largest dimension.

Any soils imported to the site for use as fill for subgrade materials should be predominantly granular and "very low" expansive (Expansion Index less than twenty [20])and should contain sufficient fines (approximately twenty [20] percent passing the No. 200 sieve) so as to be relatively impermeable when compacted. The imported soils should be approved by GEOBASE prior to importing.

8.5 Drainage

To enhance future site performance, it is recommended that all pad drainage be collected and directed away from proposed structures and slopes to disposal areas off site. For soil areas, we recommend that a minimum of five (5) percent gradient away from foundation elements be maintained. It is important that drainage be directed away from foundations and that proper drainage patterns be established at the time of construction and maintained through the life of the structures. Roof gutter discharge should be directed away from the building to suitable discharge points.

All slopes should be properly drained and maintained to help control erosion. Care should be exercised in controlling surface runoff onto temporary slopes. The area back of the slope crest should be graded such that water will not be allowed to flow freely onto the slope face. If excavations of temporary slopes are carried out in the rainy season, appropriate erosion protection measures may be required to minimize erosion of the slope cuts.

8.6 Temporary Excavations

The following subsections address unsupported and shored excavations.

8.6.1 *Unsupported Excavations*

Temporary excavations to depths of approximately four (4) feet below grade may be cut vertically without shoring. Where the necessary space is available, temporary unsurcharged excavations up to fifteen (15) feet high in level ground surface may be sloped back at 1H:1V (Horizontal:Vertical) or flatter in native soils. No surcharge loads should be permitted within a horizontal distance equal to the height of cut from crest of the excavation unless the cut is properly shored. Adjacent to existing buildings, the bottom of unshored excavations should not extend below a plane drawn at 1H:1V (Horizontal:Vertical) downward from the foundations of the existing buildings and underground pipelines unless the cut is properly shored. Where space is not available, the recommendations for design of temporary shoring presented in subsection 8.6.2

should be used.

The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing.

All excavations and shoring systems should meet, as a minimum, the requirements given in the State of California Occupational Safety and Health Administration (OSHA) and Trench Safety Standards. Stability of temporary slopes is the responsibility of the contractor.

8.6.2 *Shored Excavations*

In areas where stability or space considerations do not permit sloped excavations, temporary shoring may be used to support vertically cut excavations. In the following paragraphs, recommendations are provided for the design of both cantilevered and braced/tied back shoring.

8.6.2.1 General

All shoring systems should meet minimal requirements given in the State of California Occupational Safety and Health Standards.

A cantilevered shoring system may be used only in areas where lateral movement of soils behind the wall and associated wall movement (at least 0.01 radian angular deflection) can be tolerated. A braced or tie-back shoring system, or at-rest earth pressures should be used in areas where the performance of adjacent structures are affected by movements.

8.6.2.2 Lateral Earth Pressures

For the design of cantilevered shoring, where lateral movement of soils behind the wall can be tolerated, a triangular distribution of lateral earth pressures may be used as shown in Figure A-12, Appendix A. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of thirty-five (35) pounds per cubic foot. Where movements cannot be tolerated, a lateral pressure equal to that developed by a fluid with a density of fifty-five (55) pounds per cubic foot (at-rest earth pressures) may be used.

For the design of tied-back or braced shoring, a rectangular distribution of earth pressures as shown in Figure A-15, Appendix A, is recommended for retained soils with a level surface. In this figure, the maximum pressure is equal to 24H in pounds per square foot, where H is the height of the shoring in feet.

When shoring is used to support surcharge loads, the diagram given in Figure A-16, Appendix A, may be used to determine additional lateral earth pressures. It is recommended that surcharges be included in the design of shoring where loads due to normal street traffic or heavy equipment such as cranes or trucks are anticipated within a distance equal to wall height from the top of the shoring.

Where the shoring system is adjacent to any existing buildings, the lateral surcharge pressure from the building foundations should be considered in the shoring design, or the foundations should be underpinned prior to excavations.

8.6.2.3 Design of Soldier Piles

Lateral resistance for soldier piles may be assumed to be provided by passive pressures below the bottom of excavation equivalent to a fluid pressure of 350 pounds per cubic foot may be used for soldier piles embedded in the natural on-site soils. The aforementioned allowable passive pressures are for soldier piles spaced not less than two (2) diameters center-to-center and includes the doubling effect for isolated piles.

Provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils such that full lateral pressures can be developed.

Adequate bearing capacity should be provided for anchored soldier piles. The design vertical load will be a function of the anchor loads and their inclination. These piles may be designed for vertical loads using an allowable unit skin friction of 300 pounds per square foot where depth of undisturbed on-site soil in contact with the pile is greater than fifteen (15) feet. The unit skin friction may be applied to the full pile surface area below the base of excavation.

8.6.2.4 Lagging

Spaces between the soldier piles should be covered by continuous lagging as excavation progresses. The soldier piles and anchors should be designed for the full anticipated lateral pressure; however, the pressure transferred to the lagging will be less due to arching of the soil. The lagging can be designed for the recommended earth pressures but this pressure may be limited to a maximum value of 400 pounds per square foot. Any void between the back of lagging and the excavation should be backfilled with a two-sack sand-cement slurry.

All lumber to be left in the ground should be pressure-treated in accordance with the specifications of the American Wood Preservers Association (AWPA).

8.6.2.5 Anchor Design

Tie-back friction anchors may be used to resist lateral loads. The capacities of grouted anchors should be determined by testing of the initial anchors as outlined in the following section. For design purposes, it may be estimated that anchors will develop an average allowable friction value of 300 psf, provided that the average depth of bonded length is at least fifteen (15) feet below ground surface. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least six (6) feet on center, no reduction in the capacity of the anchors need to be considered due to group action.

A bond length sufficient to support the anticipated earth and surcharge loads should be installed behind a line rising at fifty-five (55) degrees from the horizontal starting at the base of the pile, as shown on Figure A-12, Appendix A. The anchors may be installed at angles between fifteen (15) degrees to forty-five (45) degrees below the horizontal. If caving occurs in the drilled shafts, casing should be used prior to concrete pour, but casing must be pulled as the shaft is poured. Structural concrete should be placed in the bonded length. Pouring concrete should be done by pumping the concrete through a tremie or pipe extending to the bottom of the shaft. The anchor shaft between the failure plane and the face of the shoring may be backfilled with sand-cement slurry after concrete placement.

8.6.2.6 Anchor Testing

GEOBASE should select at least two (2) percent of the anchors or a minimum of two (2) anchors, whichever is more, for twenty-four (24) hour 200 percent tests, and at least an additional five (5) percent of the anchors for quick 200 percent tests. The purpose of the 200 percent tests is to verify the friction value used in design. Where satisfactory test results are not achieved on the initial anchors, the anchor diameter and/or length should be increased on subsequent anchors until satisfactory test results are obtained.

The total elongation at anchor head during the twenty-four (24) hour 200 percent tests should not exceed twelve (12) inches during loading. The anchor deflection should not exceed 0.75 inch after anchor lock-off and during the twenty-four (24) hour period, measured after the 200 percent test load is applied. If the anchor movement after the 200% load has been applied for twelve (12) hours is less than one-half (0.5) inch, and the movement over the previous four (4) hours has been less than 0.1 inch, the twenty four (24) hour test may be terminated.

For the quick 200 percent tests, the 200 percent test load should be maintained for thirty (30) minutes. The deflection after the 200 percent test load has been applied should not exceed 0.25 inch during the thirty (30) minute period for the anchor to be approved for the design loading.

All of the production anchors should be proof tested to at least 150 percent of the design load. The rate of creep under the 150 percent load should not exceed 0.1 inch over a fifteen (15) minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than ten (10) percent from the design load, the load should be reset until the anchor is locked-off within ten (10) percent of the design load.

It is recommended that the plans and specifications for the proposed shoring system be reviewed by GEOBASE. The installation of the anchors and the testing of the completed anchors should be observed by GEOBASE.

8.6.2.7 Monitoring

Inspection, survey monitoring and observations of the shoring system shall be in accordance with CBC 2016, subsection 1812A.6. Monitoring of the existing structure shall be in accordance with CBC 2016, subsection 1812A.6.

It is recommended that a licensed surveyor be retained to establish monuments on the shoring, the surrounding ground and adjacent structures prior to excavations. Such monuments should be monitored for horizontal and vertical movement during construction on a daily basis. Results of the monitoring program should be provided immediately to the project structural (shoring) engineer and GEOBASE for review and evaluation.

8.7 Trench Backfill

Underground utility trenches could be backfilled and properly compacted by mechanical means. Pipe bedding, shading, and trench backfill should conform to the requirements of appropriate utility authorities.

If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, other methods of utility trench compaction may also be appropriate as approved by GEOBASE at the time of construction. Jetting or flooding of backfill material is not recommended.

IX. FOUNDATION RECOMMENDATIONS

9.1 General

The following recommendations have been formulated from visual, physical and analytical considerations of the existing site conditions and are believed to be applicable for the proposed development.

The on-site soils have a "very low" expansion potential. The recommendations presented in the following subsections are based on a "very low" expansion potential for the subgrade soils. Foundations and slab reinforcement configurations should meet, as a minimum, the requirements of the regulating agencies and the 2016 CBC.

9.2 Footings

Spread or continuous footings may be used for support of the proposed D&T Building. Footings should be based a minimum of three (3) feet below the lowest adjoining grade.

9.2.1 *Soil Bearing Pressures*

Footings with a minimum width of two (2) feet and maximum width of eleven (11) feet, founded on a minimum of five (5) feet of compacted fill (subsection 8.3.1), may be designed for an allowable bearing pressure of 4,000 psf. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed the aforementioned allowable bearing value.

Footings placed closer than one (1) width apart should be structurally tied.

9.2.2 *Footings Adjacent to Trenches or Existing Footings*

Where footings are located adjacent to utility trenches, they should extend below a one-to-one plane projected upward from the inside bottom corner of the trench. Footing excavations adjacent to the footings of existing buildings should be carried out such that the existing footings are not undermined.

9.2.3 *Settlement*

For allowable dead-plus-live load bearing pressures of 4,000 psf, the total and differential settlements of the footings are not anticipated to exceed one (1.0) inch and one-half (0.5) inch, respectively. Total seismic settlements are anticipated not to exceed one-half (0.5) inch and differential seismic settlements are estimated at three-tenths (0.3) of an inch over a distance of thirty (30.0) feet.

Where the D&T Building joins the existing hospital, minor separation cracks are anticipated as the subgrade soils adjust to the newly established loading and moisture conditions. Such cracks, if any, may be repaired a year or two after completion of construction.

Notwithstanding the preceding, the static settlement of the footings foundation system should be reviewed by GEOBASE once the configuration of the footings are finalized.

9.2.4 *Lateral Load Resistance*

Lateral loads (wind or seismic) against structures may be resisted by friction between the bottom of foundations and the supporting soils. An allowable friction coefficient of 0.35 between spread footing and the underlying compacted soil or soil replaced by mixing is recommended. An allowable lateral bearing pressure equal to an equivalent fluid weight of 200 pounds per cubic foot to a maximum of 3,000 pounds per square foot acting against the foundations may also be used, provided the foundations are poured tight against compacted fill.

9.2.5 *Footing Observations*

All foundation excavations should be observed by GEOBASE prior to the placement of forms, reinforcement, or concrete, for verification of conformance with the intent of these recommendations and confirmation of the bearing capacities. All loose or unsuitable materials should be removed prior to the placement of concrete. Materials from footing excavations should not be spread in slab-on-grade areas unless compacted.

9.3 Retaining Walls

9.3.1 *General*

The south wall of the proposed D&T Building will retain approximately fifteen (15) feet of soil. The following subsections provide earth pressures and other parameters required for the design of retaining walls.

9.3.2 *Earth Pressures*

Wall backfill is anticipated to consist of "very low" expansive soils. These walls should be designed to resist lateral pressures imposed by the surrounding soils and surcharge loads. It is recommended that for static loading condition: walls which are free to rotate at the top (at least 0.01radian angular deflection) should be designed to resist a lateral pressure imposed by an equivalent fluid weighing thirty-five (35) pounds per cubic feet; and, walls that are structurally braced against movement at the top should be designed to resist a lateral pressure equivalent to that imposed by a fluid weighing fifty-five (55) pounds per cubic foot. In addition, a uniform pressure equal to one-third (1/3) and one-half ( ) of any vertical pressure adjacent to the basement wall should be assumed to act on the free and braced walls, respectively. These aforementioned pressures assume that positive drainage will be provided as recommended in subsection 9.3.3.

For seismic loading conditions, where appropriate, the dynamic loading increment of active earth pressures may be taken as sixteen (16) psf per foot of wall height distributed in an inverted triangular distribution. For restrained walls, seismic earth pressure increment of twenty-six (26) psf per foot of wall height distributed in an inverted triangular distribution may be used, where appropriate.

9.3.3 *Wall Backfill and Drainage*

The backfill for basement walls shall be granular soils as described in subsection 8.4.3 and walls should be provided with backdrains to relieve possible hydrostatic pressures on the wall. A pre-fabricated drainage system such as Miradrain, Eakadrain or equivalent, installed in accordance with the manufacturer's recommendations, may be used. Alternatively, the wall should be designed to withstand hydrostatic pressures.

The basement walls below existing grade should be waterproofed to prevent moisture build-up on the interior sides of the walls as a result of water migration from the soils in contact with the walls. The water proofing should be applied for the full height of the basement walls and walls below existing grade, and meet, as a minimum, the requirements of the CBC 2016.

9.4 Minor Structures

Minor structures may be designed using the presumptive load-bearing values outlined in CBC 2016, provided that the risk of future settlements and associated maintenance can be tolerated.

9.5 Ultimate Values

The recommended design values presented in this report are for use with loading determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the factors given in Table V.

TABLE V
LOAD FACTORS FOR ULTIMATE DESIGN

Foundation Loading	Ultimate Design Loading
Bearing Value	3.0
Passive Pressure	1.33
Coefficient of Friction	1.25

In no event, however, should the footing sizes be reduced from those required for support of dead-plus-live loads when using the working stress values.

9.6 Floor Slabs

Concrete slab-on-grade may be used for the proposed D&T Building. The subgrade of the slab-on-grade should be prepared in accordance with the recommendations provided in subsections 8.3 and 8.4.

In moisture sensitive areas, as a minimum, the floor slabs should be damproofed per CBC 2016, subsection 1805A.2; specific recommendations can be provided by a Waterproofing Consultant.

A subgrade modulus of 150 pounds per cubic inch may be used for slab design. The slab should be designed by the Structural Engineer using applicable CBC requirements, and the various anticipated loading conditions including shrinkage, temperature stresses, construction and operation conditions.

X. SOIL CORROSIVITY -- IMPLICATIONS

Electrical conductivity, pH, chloride and water soluble sulfate tests were conducted on representative samples by Anaheim Test Labs, and the results are provided in Appendix C. The tests results indicate that the subsoils at the site have a "low" corrosive potential with respect to concrete and "corrosive" potential with respect to steel and other metals. Therefore, Type II Portland Cement may be used for construction of concrete structures in contact with subgrade soils.

XI. PAVEMENT RECOMMENDATIONS

11.1 Asphaltic Concrete Pavement

Based on an R-value of fifty (50), the following alternative preliminary minimum pavement sections may be used. The traffic index assumed in Table VI, below, **should be confirmed by the Civil Engineer** and R-value tests should be performed during grading, prior to finalizing the pavement sections.

TABLE VI

ASPHALTIC CONCRETE PAVEMENT SECTIONS

PAVEMENT UTILIZATION	TRAFFIC INDEX	ASPHALTIC CONCRETE (INCHES)	CLASS II BASE (INCHES)
Automobile parking areas	5	3	3
Truck and bus loading/unloading areas and driveways	6	4	3

The upper twelve (12) inches of subgrade soils, below the aggregate base, should be scarified, moisture conditioned and recompacted to a minimum of ninety-five (95) percent relative compaction, at to slightly above optimum moisture content, based on ASTM D 1557.

The aggregate base must meet CALTRANS "Class 2 Base" specifications and should be compacted to at least ninety-five (95) percent relative compaction based on ASTM D 1557. Asphaltic concrete should be compacted to at least ninety five (95) percent of the density obtained with the California Kneading Compactor (CAL 304).

11.2 Rigid Pavement

A Portland Cement concrete (PCC) pavement may also be used. In the design of the PCC pavement section shown in Table VI, below, the following design parameters were used:

- Modulus of subgrade reaction of the soil, k -- 240 pci
- Modulus of rupture of concrete, MR -- 500 psi
- Traffic Category, TC -- C
- Average daily truck traffic, ADTT -- 100

The traffic category and average daily truck traffic should be confirmed by the civil engineer and R-value tests should be performed during grading, prior to finalizing PCC thickness.

Based on the design parameters presented above, the following rigid pavement section, calculated in general conformance with the procedure recommended by ACI 330R-01, may be used.

TABLE VII
PCC PAVEMENT SECTION

PAVEMENT UTILIZATION	PCC Minimum Thickness (inches)
Truck loading/unloading areas (TC = C)	6

The upper twelve (12) inches of subgrade soils below the PCC should be scarified, moisture conditioned and recompacted to a minimum of ninety-five (95) percent relative compaction, at to slightly above optimum moisture content, based on ASTM D 1557.

The PCC pavement reinforcement should be designed by the structural engineer for shrinkage, temperature stresses and loading conditions including vehicular traffic. A thickened edge should be constructed on the outside of concrete pavements subject to wheel loads. Control joints should be included in the design of the PCC by the structural engineer at a maximum spacing of fifteen (15) feet each way.

XII. PLAN REVIEW, OBSERVATIONS AND TESTING

Post-investigation services are an important and integrated part of this investigation and should be carried out by GEOBASE. The project foundation and grading plans, and specifications should be forwarded to GEOBASE for review for conformance with the intent of the soils recommendations.

Geotechnical observations of excavation bases should be carried out prior to fill placement. Observations and testing of all fill placement should be carried out on a continuous basis to verify the design assumptions and conformance with the intent of the recommendations. Observations of footings bases should be carried out prior to concrete pour.

XIII. LIMITATIONS

This investigation was performed in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is intended for use by the client and its representatives, and with regard to the specific project discussed herein. Any changes in the design or location of the proposed new structure, however slight, should be brought to our attention so that we may determine how they may affect our conclusions. The conclusions and recommendations contained in this report are based on the data relating only to the specific project and location discussed herein. This report does not relate any conclusions or recommendations about the potential for hazardous and/or contaminated materials existing at the site.

The analyses and recommendations submitted in this report are based upon the observations noted during drilling of the borings, interpretation of laboratory test results, and geological evidence. This report does not reflect any variations which may occur away from the borings and which may be encountered during construction. If conditions observed during construction are at variance with the preliminary findings, we should be notified so that we may modify our conclusions and recommendations, or provide alternate recommendations, if necessary.

The recommendations presented herein assume that the plan review, observations and testing services, outlined in Section XII of the report, will be provided by GEOBASE. During execution of the aforementioned services, GEOBASE can finalize the report recommendations based on observations of actual subsurface conditions evident during construction. GEOBASE cannot assume liability for the adequacy of the recommendations if another party is retained to observe construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project, and incorporated into the plans and specifications. In this respect, it is recommended that we be allowed the opportunity to review the project plans and the specifications for conformance with the geotechnical recommendations.

This office does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for other than our own personnel on the site. Therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein to be unsafe.

This report is subject to review by the appropriate regulating agencies.

Respectfully submitted
GEOBASE, INC.

H. D. Nguyen, P.E.
R.C.E. 82460
Associate Engineer

J-M. Chevallier, P.E., G.E.
R.C.E. 39198; G.E. 2056
Managing Principal

S. Gutierrez
P.G. 8835, C.E.G. 2652
Associate Geologist

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APPENDIX A

Figure A-1	Site Location Map
Figure A-2	Site, Boring and CPT Location Plan
Figure A-3	Site Topographic Survey Plan
Figure A-4	Existing Foundation Plan
Figure A-5	Regional Geologic Map
Figure A-6	Geologic Cross Section A-A’
Figure A-7	Geologic Cross Section B-B’
Figure A-8	Regional Fault Map
Figure A-9	Vicinity Fault Map
Figure A-10	Historical Earthquakes Map
Figure A-11	Shear Wave Velocity Profiles
Figure A-12	Liquefaction Susceptibility Map
Figure A-13	Subsidence Susceptibility Map
Figure A-14	FEMA Flood Map
Figure A-15	Earth Pressures and Tieback Geometry for Shoring
Figure A-16	Additional Lateral Earth Pressures on Shoring

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SITE LOCATION MAP
Kaiser Permanente MVMC – D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

FIGURE A-1

SITE PLAN
07/05/17

CO ARCHITECTS

5055 Wilshire Boulevard, 9th Floor
Los Angeles, California 90036
323.525.0500 phone, 323.525.0955 fax

C.314.81.00

KAISER PERMANENTE

Site, Boring and CPT Locations Plan

MORENO VALLEY MEDICAL CENTER

PROJECT No. K0227

Figure A-2

SD-10

SB&O INC.
PLANNING ENGINEERING SURVEYING
41689 Enterprise Circle North, Suite 126
Temecula, Ca. 92590
951-695-8900
951-695-8901 Fax

KAISER HOSPITAL
CITY OF MORENO VALLEY
TOPOGRAPHIC SURVEY

OCTOBER 27, 2009 JN 68222

C.314.81.00

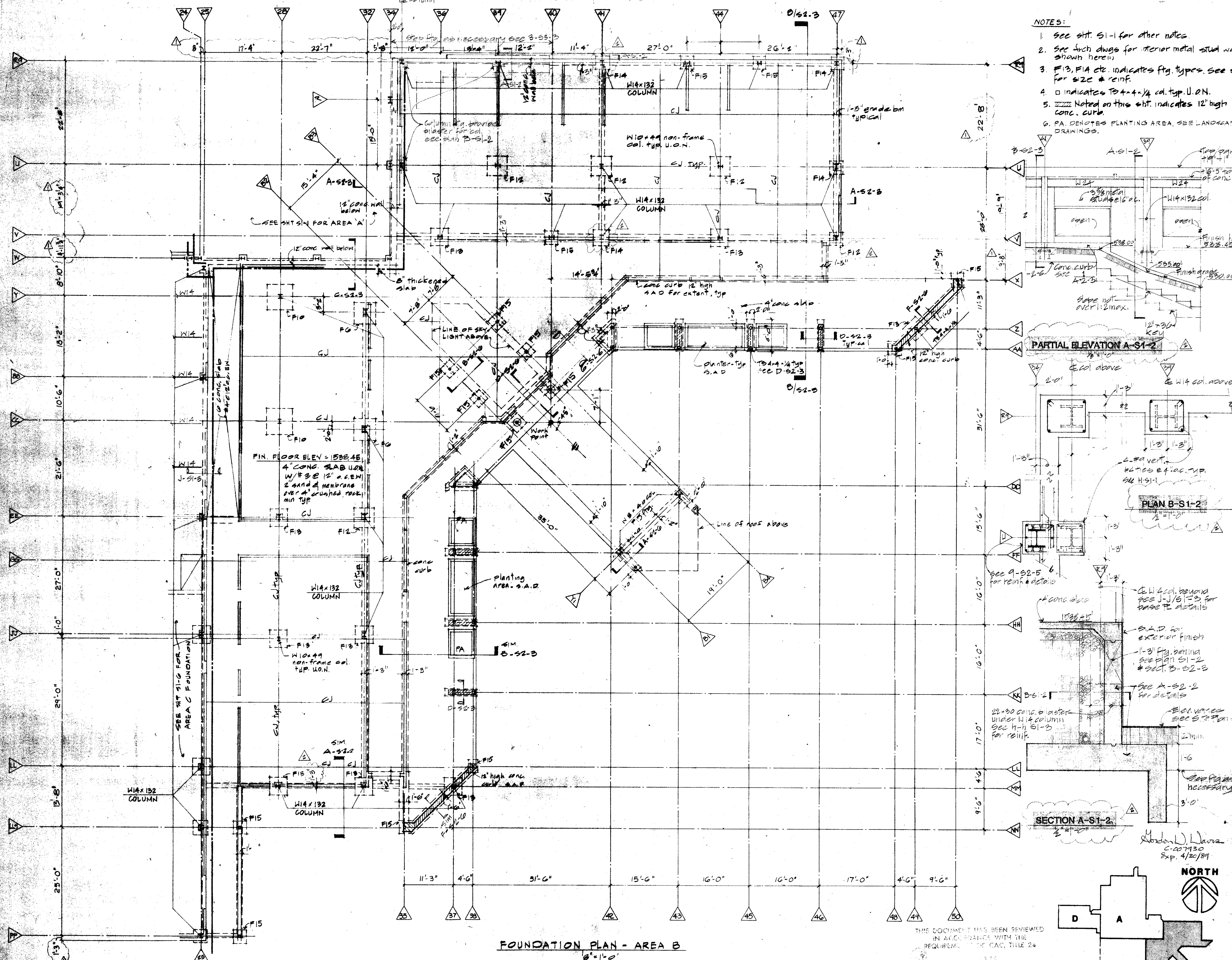
SITE TOPOGRAPHIC SURVEY PLAN

FIGURE A-3

C.314.81.00

EXISTING FOUNDATION PLAN

FIGURE A-4
PAGE 1 OF 2



NOTES:

1. See sht. S1-1 for other notes.
2. See Arch dwgs for interior metal stud walls not shown herein.
3. F13, F14 etc. indicates ftg. types. See sht. S1-3 for size & reinf.
4. □ indicates TS 4x4x1/4 col. typ. U.O.N.
5. ▨ Noted on this sht. indicates 12" high conc. curb.
6. PA. DENOTES PLANTING AREA. SEE LANDSCAPE DRAWINGS.

Henningson, Durham & Richardson, Inc.
A Centerra Company

Leong/ Razzano & Associates, Inc.
Consulting Civil and Structural Engineers

MORENO VALLEY HOSPITAL

MORENO VALLEY CALIFORNIA

HEMET VALLEY HOSPITAL DISTRICT

FOUNDATION PLAN - AREA B

Project Number	00501-001-031
Date	JUNE 1988
Project Manager	G. DAVIS
Design	D. BATTRICK
Structural	LEONG / RAZZANO
Mechanical	K. GILL
Electrical	M. HAWKINS
Drawn By	

Revisions
1. 4 NOVEMBER 1988
2. 1-20-89

Gordon W. Razzano
C-007930
Exp. 4/30/89

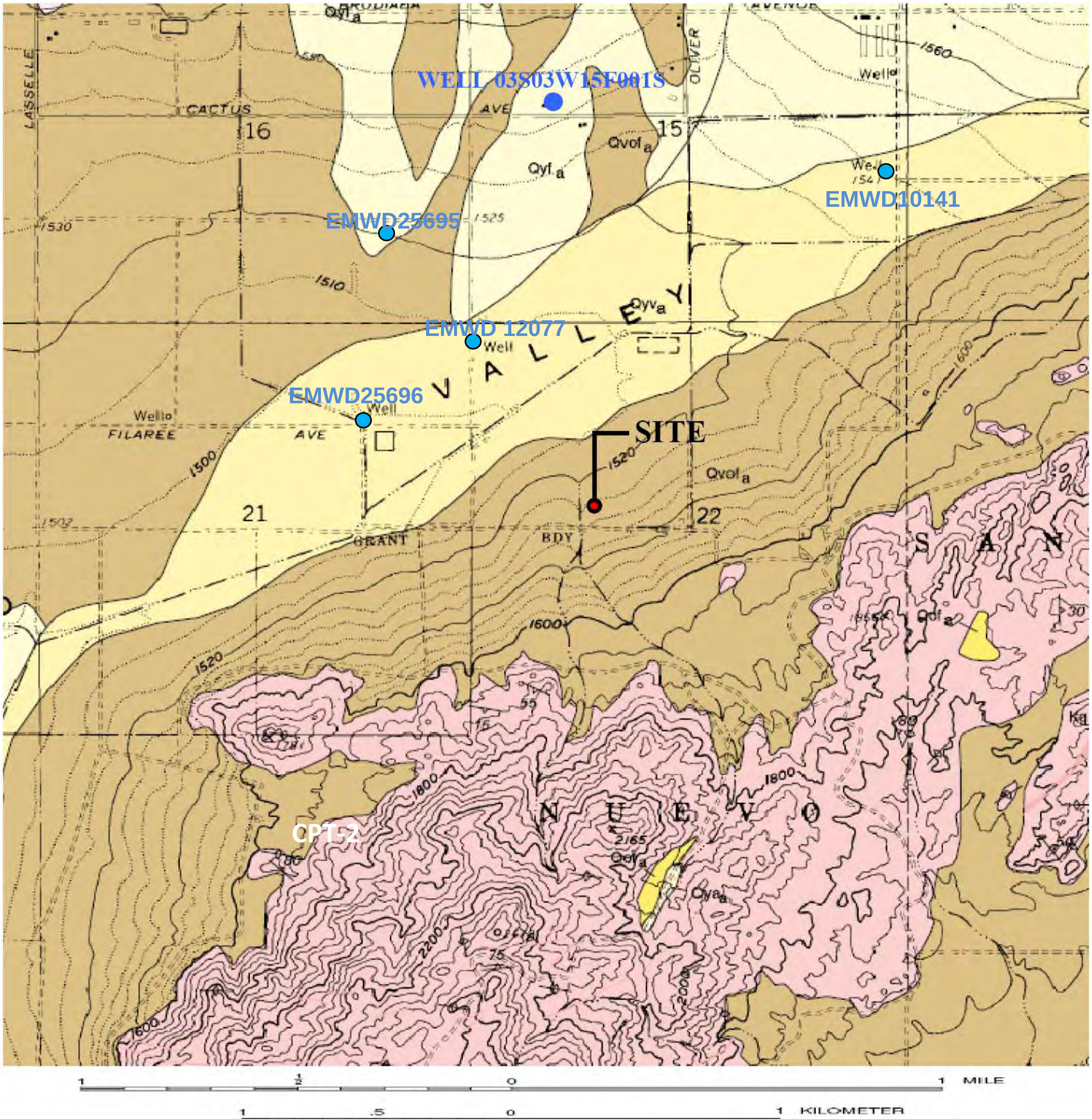
THIS DOCUMENT HAS BEEN REVIEWED IN ACCORDANCE WITH THE REQUIREMENTS OF CAC TITLE 24

Sheet Number S1-2

C.314.81.00

EXISTING FOUNDATION PLAN

FIGURE A-4
PAGE 2 OF 2



EXPLANATION

Qyf	Qyf - Quaternary Young Alluvial Fan Deposits (Holocene and late Pleistocene)
Qyv	Qyv - Quaternary Young Alluvial Valley Deposits (Holocene and late Pleistocene)
Qvof	Qvof - Quaternary Very Old Alluvial Fan Deposits (early Pleistocene)
Kqd	Kgd - Quatz Diorite, Undifferentiated (Cretaceous)

Source: Morton, D. M., and Matti, J C., 2001, Geologic Map of the Sunnymead Quadrangle, Riverside County, California: Version 1.0, Scale 1:24,000. Open File Report 01-450, Published by the United States Geological Survey in Cooperation with the California Geological Survey and the United States Air Force.

GEOBASE

REGIONAL GEOLOGIC MAP

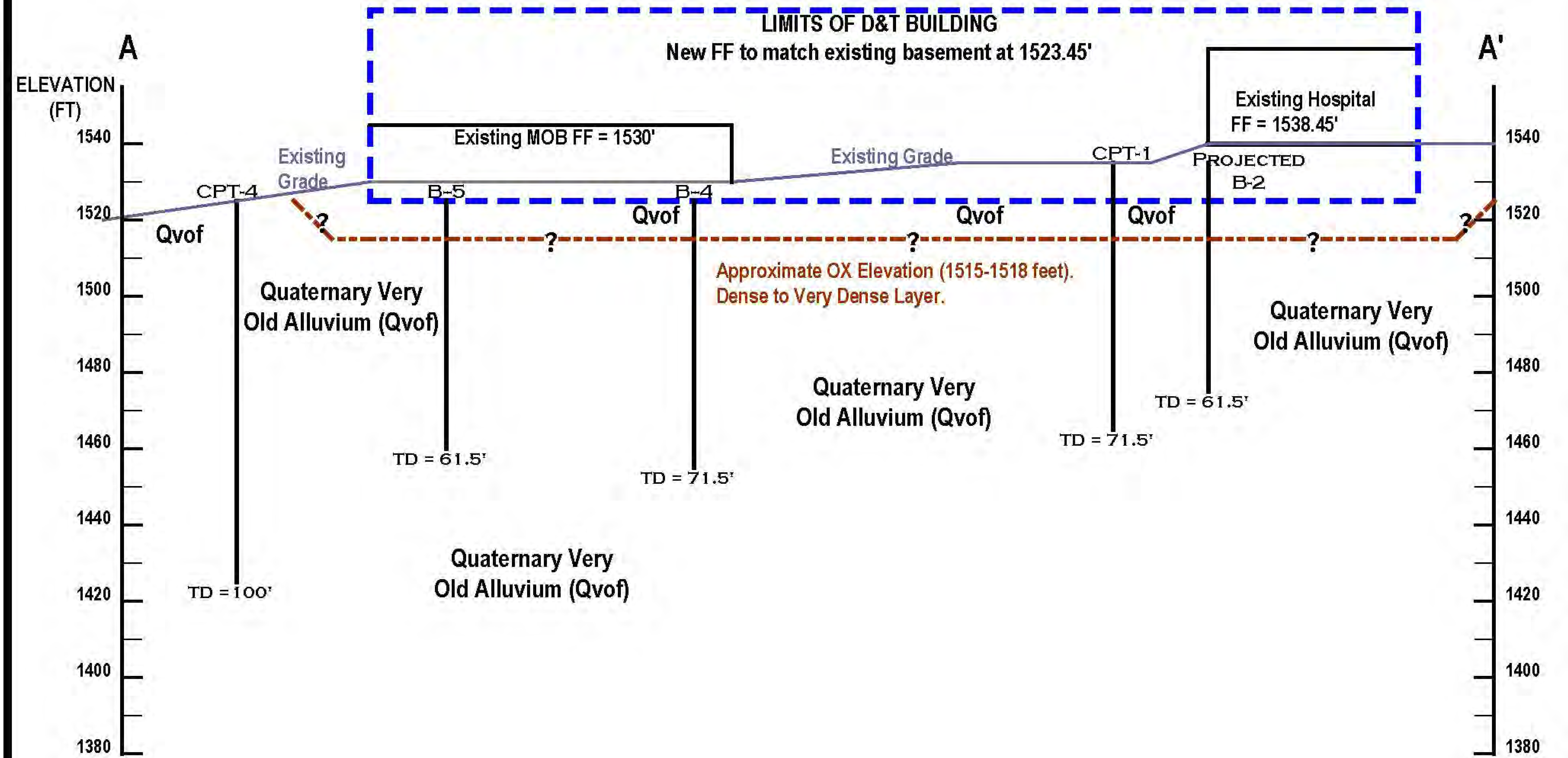
Kaiser Permanente MVMC – D&T BUILDING

27300 Iris Avenue

Moreno Valley, California

C.314.81.00

FIGURE A-5



SCALE AS SHOWN:
HORIZONTAL 1 IN. = 40 Feet
VERTICAL 1 IN. = 40 Feet

NORTH

EXPLANATIONS:
Qvof -- Quaternary Very Old Alluvium
TD -- Total depth of borehole
dense to very dense layer, overexcavation bottom

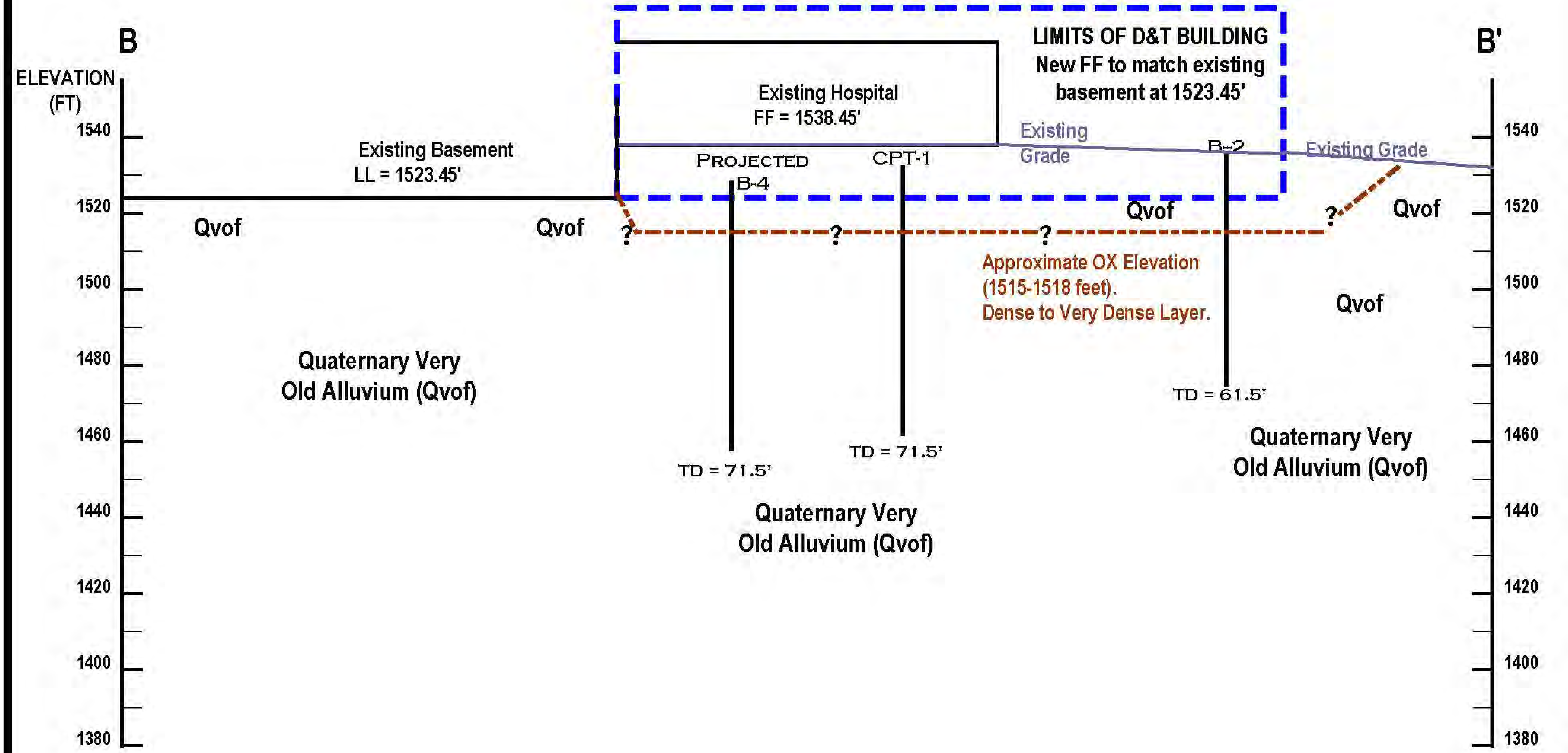
NOTES:
1. Locations of the cross section are shown on Figures A-2 & A-3
2. Soil profiles are known with accuracy only at the locations observed. The subsoil condition between borehole locations has been inferred from geological or geotechnical evidence and may vary from that shown.

GEOBASE

GEOLOGIC CROSS-SECTION A-A'
Kaiser Permanente MVMC -- D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

FIGURE A-6



SCALE AS SHOWN:
HORIZONTAL 1 IN. = 40 Feet
VERTICAL 1 IN. = 40 Feet

EXPLANATIONS:
Qvof -- Quaternary Very Old Alluvium
TD -- Total depth of borehole
dense to very dense layer, overexcavation bottom

NOTES:
1. Locations of the cross section are shown on Figures A-2 & A-3
2. Soil profiles are known with accuracy only at the locations observed. The subsoil condition between borehole locations has been inferred from geological or geotechnical evidence and may vary from that shown.

NORTH

GEOBASE

GEOLOGIC CROSS-SECTION B-B'
Kaiser Permanente MVMC -- D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

FIGURE A-7

EXPLANATION

Fault along which historic (last 200 years) displacement has occurred.

Holocene fault displacement (during past 10,000 years).

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Late Cenozoic faults within the Sierra Nevada.

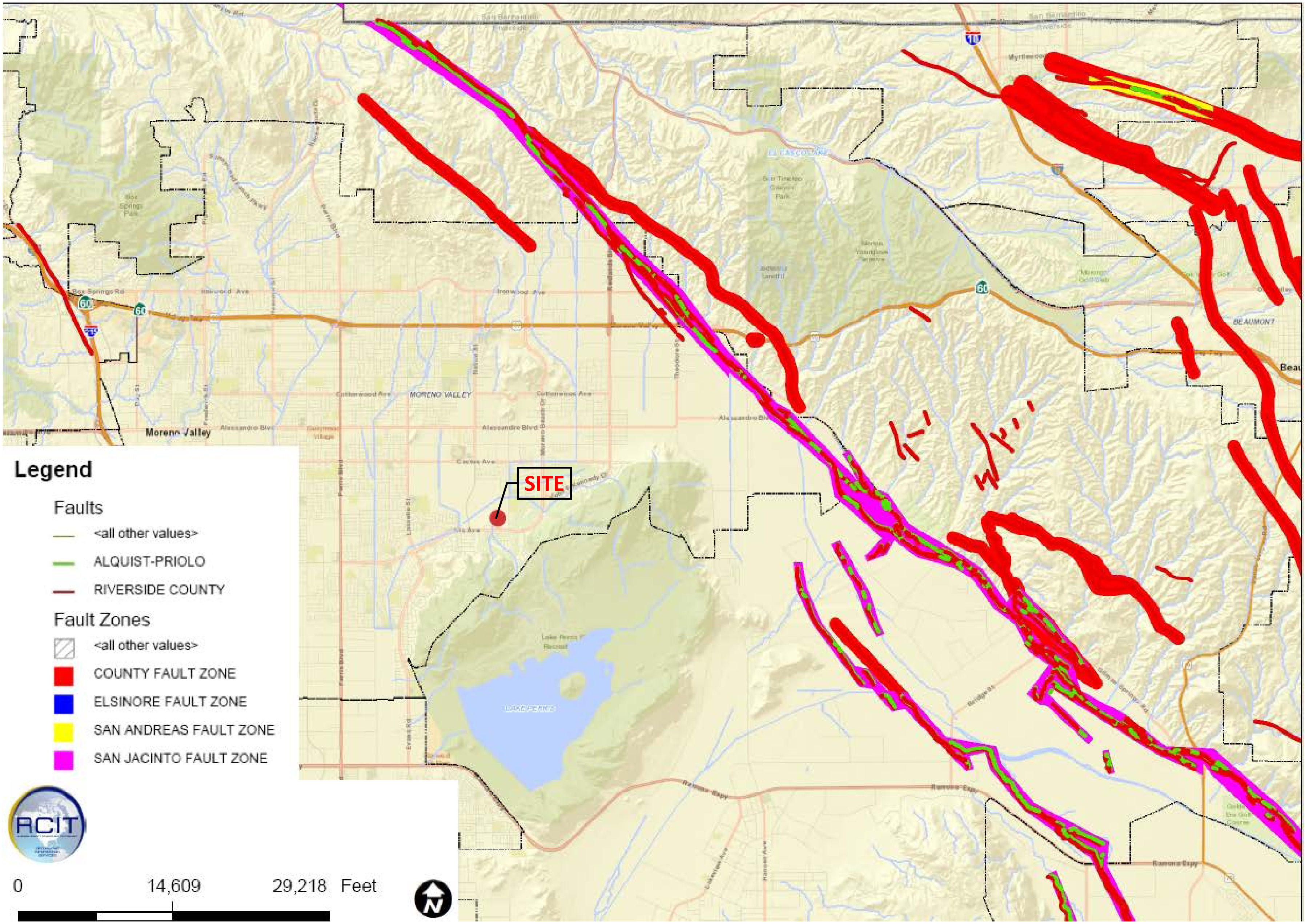
Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement.

Pink band added to emphasize location of historic fault displacement.

Approximate Scale 1 Inch Equals 10.89 Miles

Source: Jennings, C.W., 1994, Fault Activity map of California and Adjacent Areas with Location and Ages of Recent Volcanic Eruptions: California Division of Mines and Geology, Geologic Data Map Series, Map No. 6, Scale 1 : 750,000.

GEOBASE	**REGIONAL FAULT MAP** Kaiser Permanente MVMC – D&T BUILDING 27300 Iris Avenue Moreno Valley, California
	C.314.81.00 **FIGURE A-8**

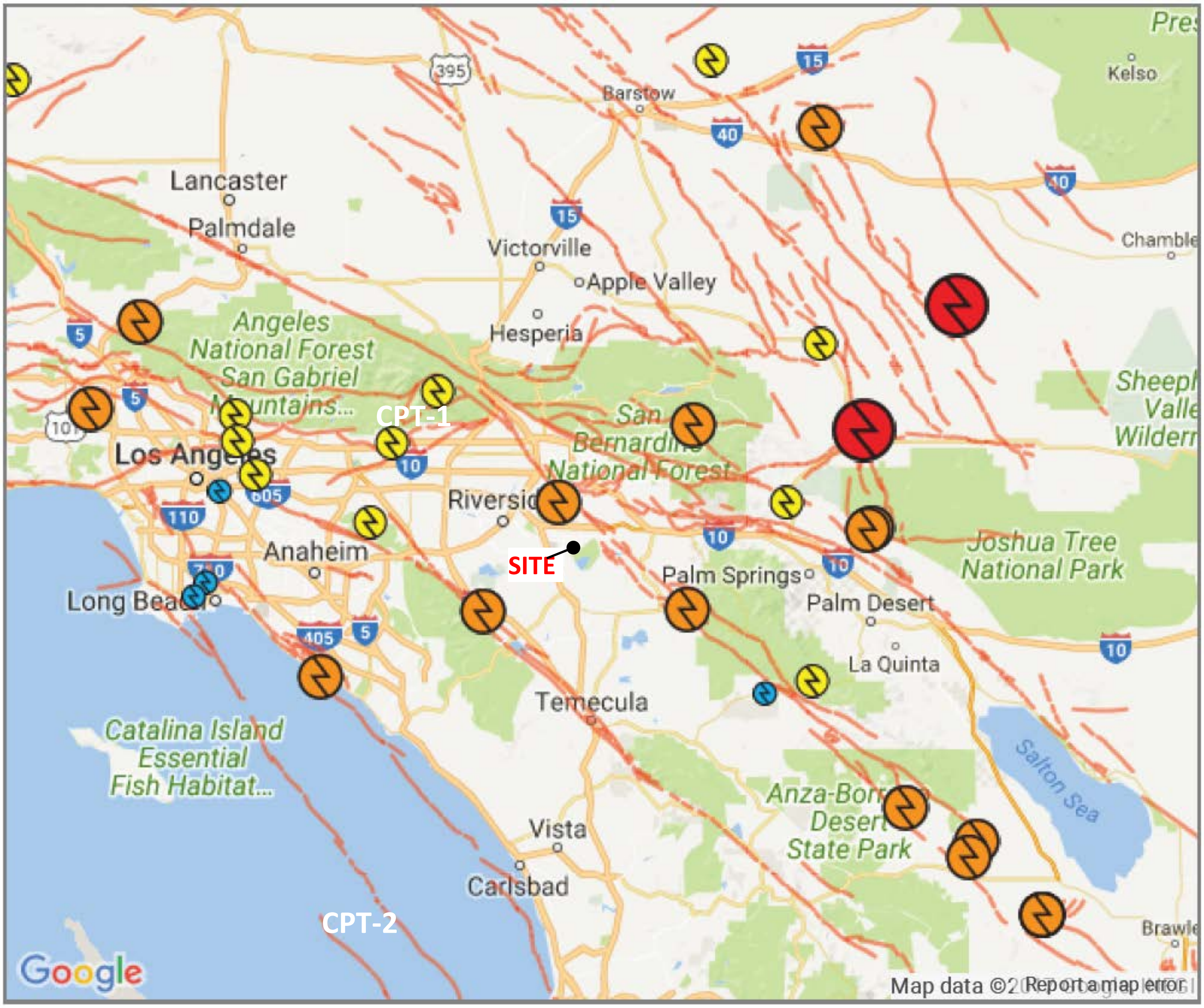


GEOBASE

VICINITY FAULT MAP
Kaiser Permanente MVMC – D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

FIGURE A-9



Sources: Southern CA Earthquake Center, Division of Geological and Planetary Sciences | California Institute of Technology.

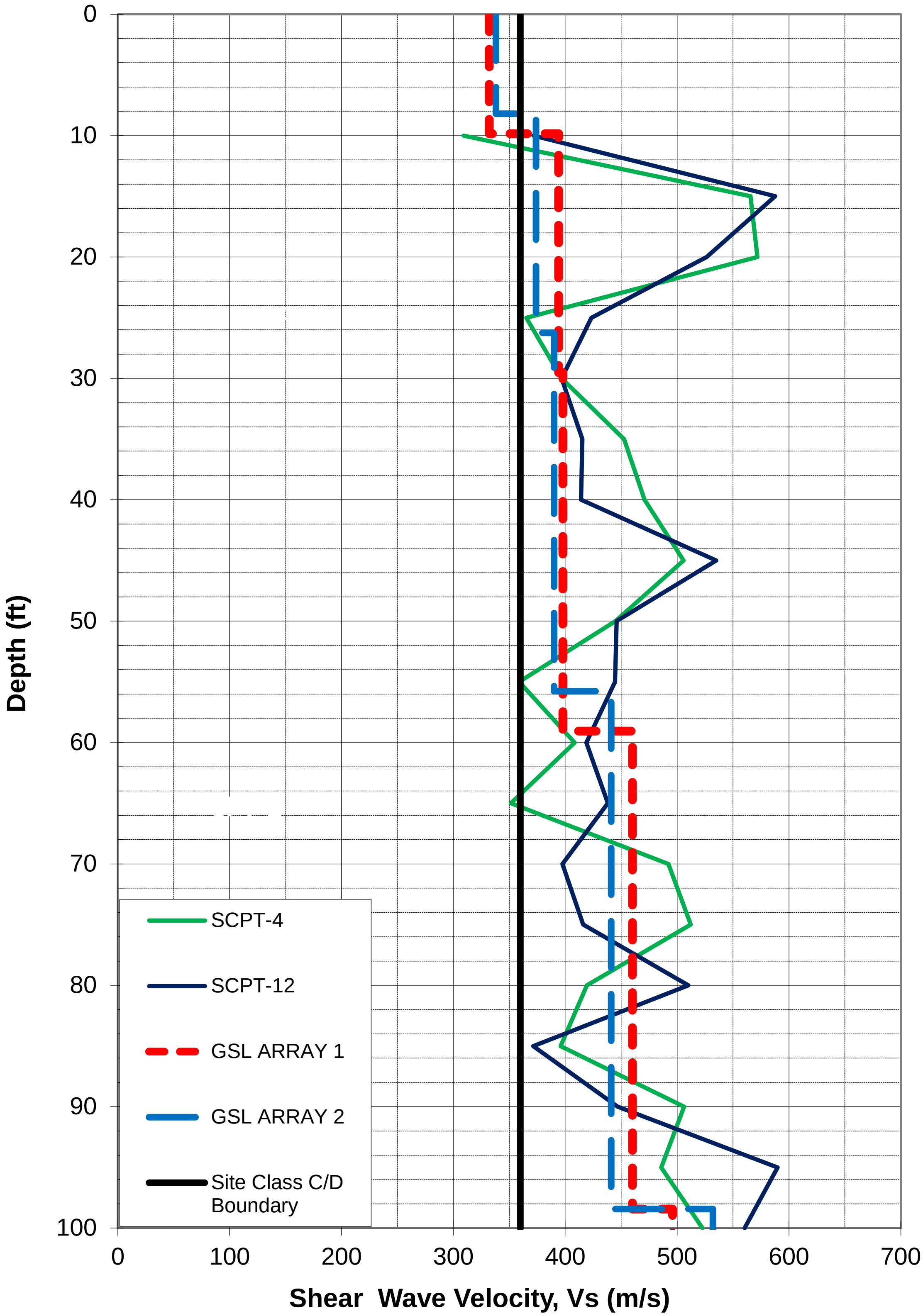
Note: Fault traces are in red as shown.

GEOBASE

HISTORICAL EARTHQUAKES MAP
Kaiser Permanente MVMC – D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

FIGURE A-10

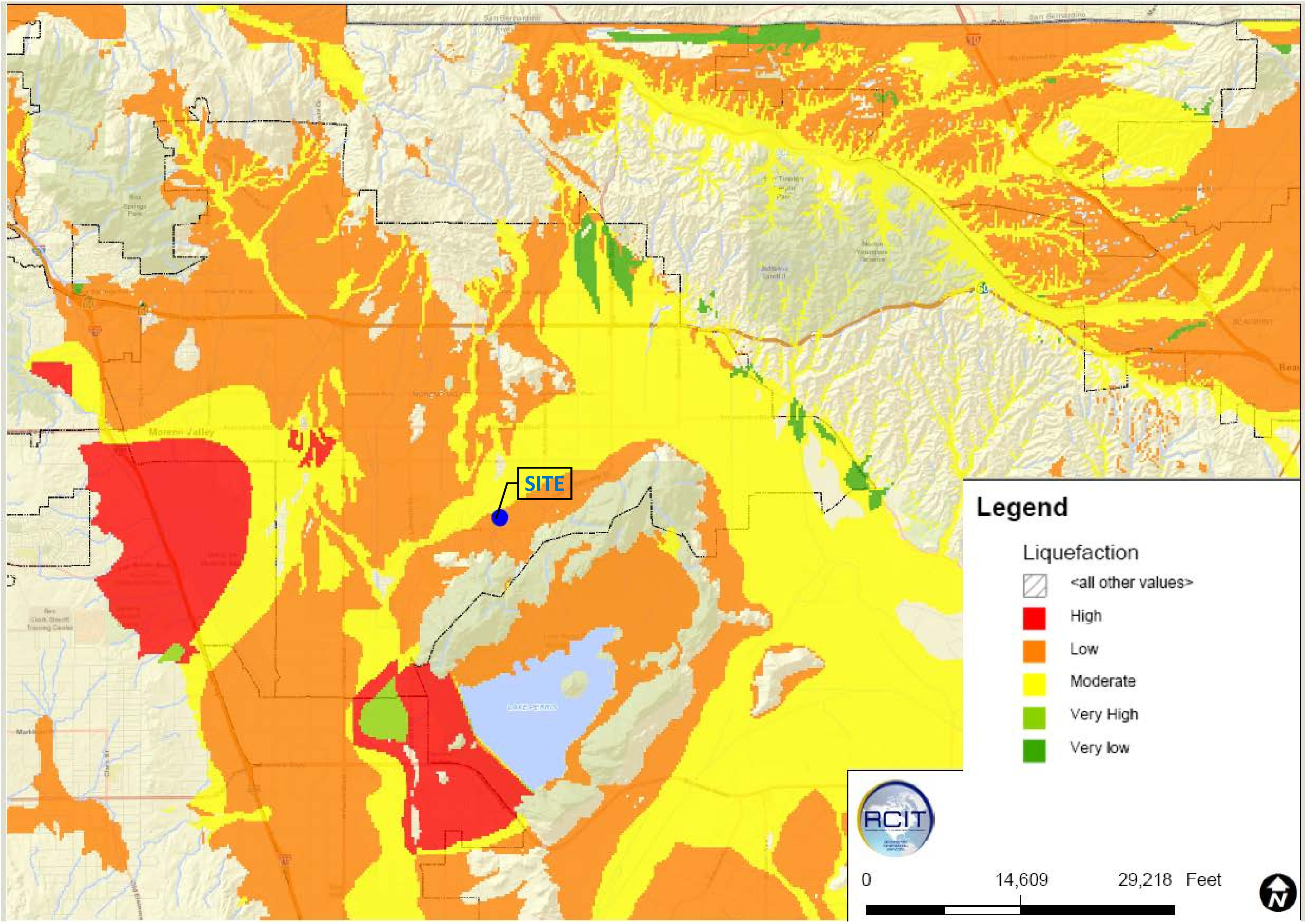


GEOBASE

SHEAR WAVE VELOCITY PROFILES
Kaiser Permanente MVMC – D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

FIGURE A-11

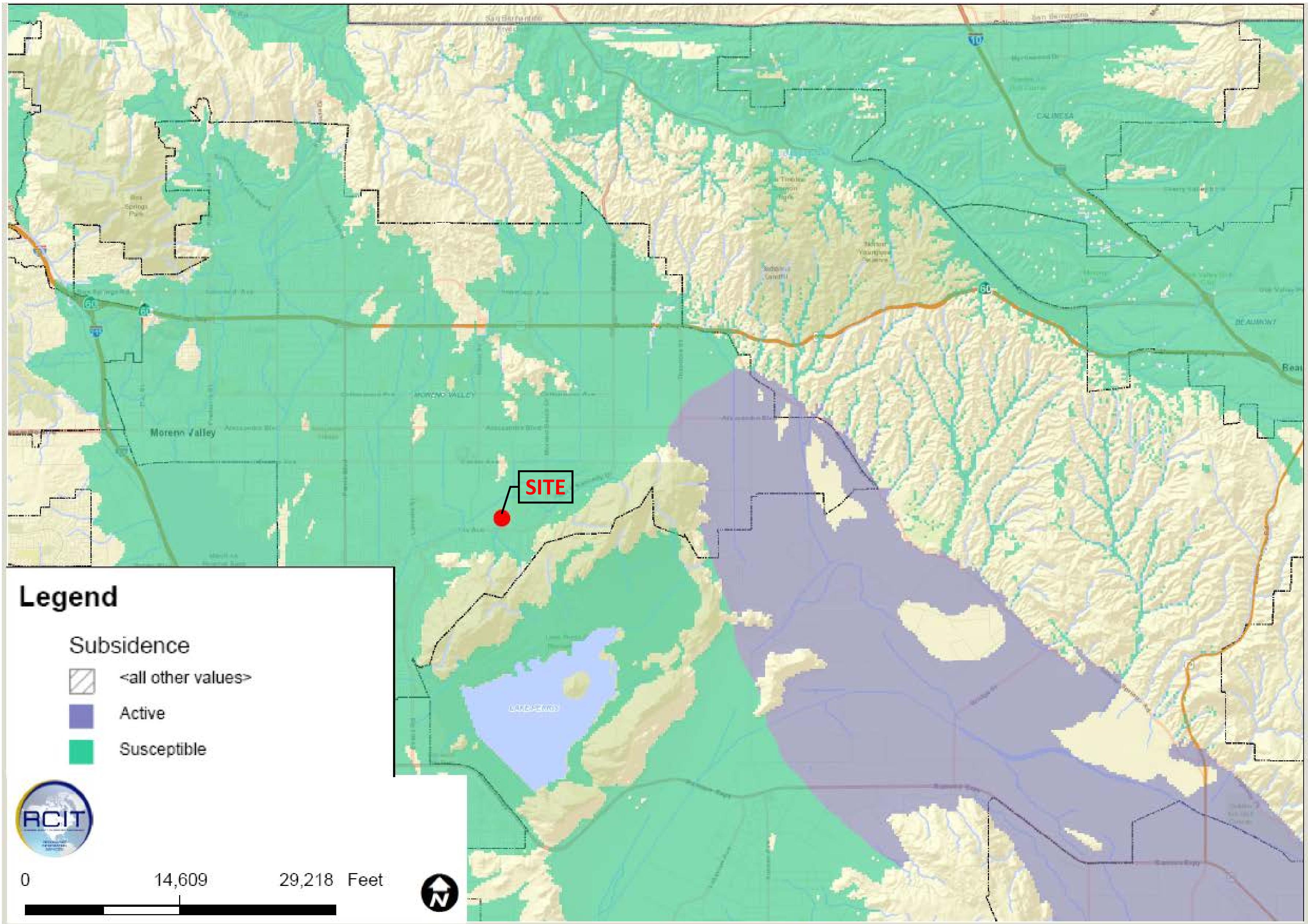


GEOBASE

LIQUEFACTION SUSCEPTIBILTY MAP
Kaiser Permanente MVMC – D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

FIGURE A-12

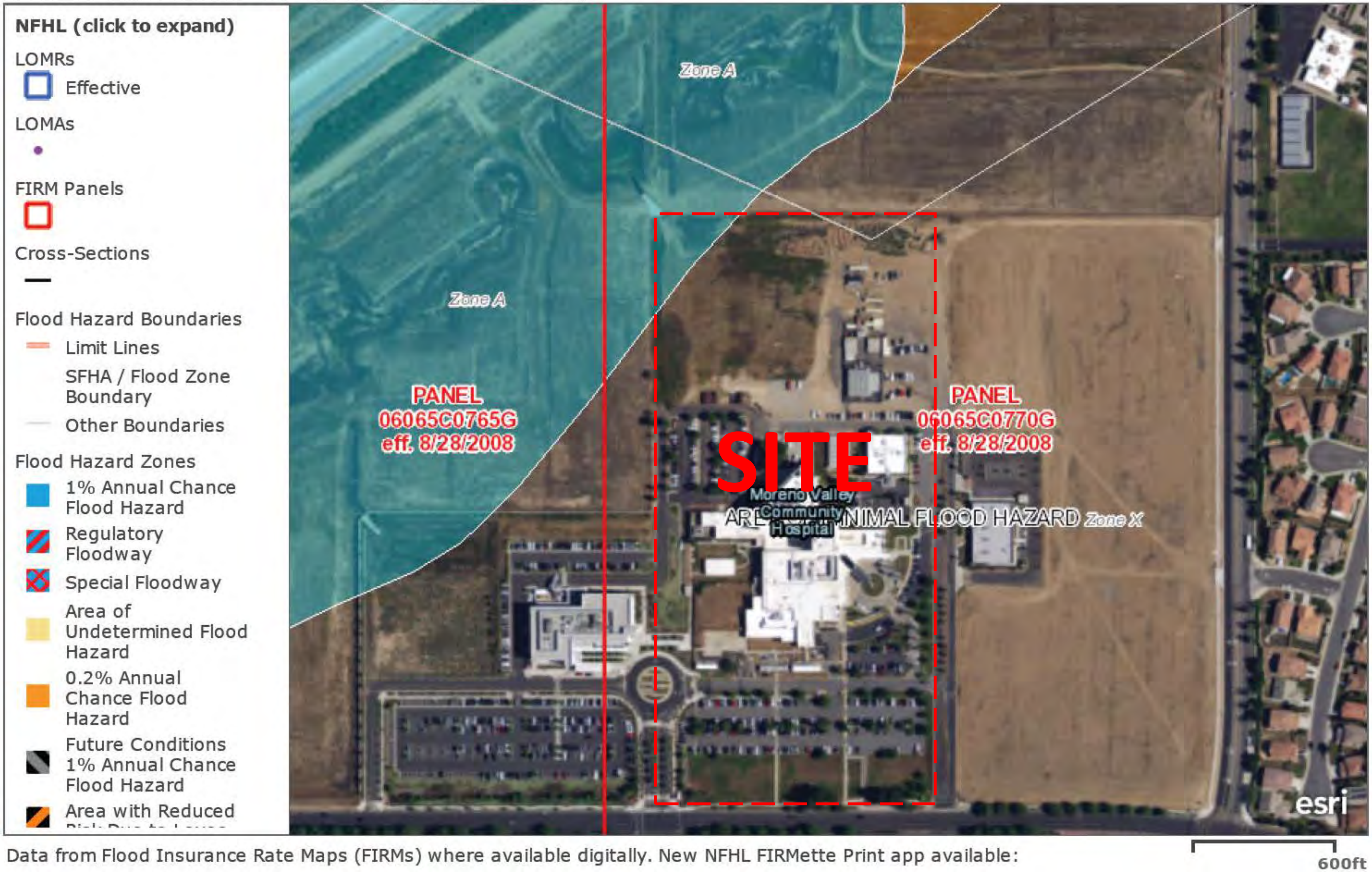


GEOBASE

SUBSIDENCE SUSCEPTIBILTY MAP
Kaiser Permanente MVMC – D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

C.314.81.00

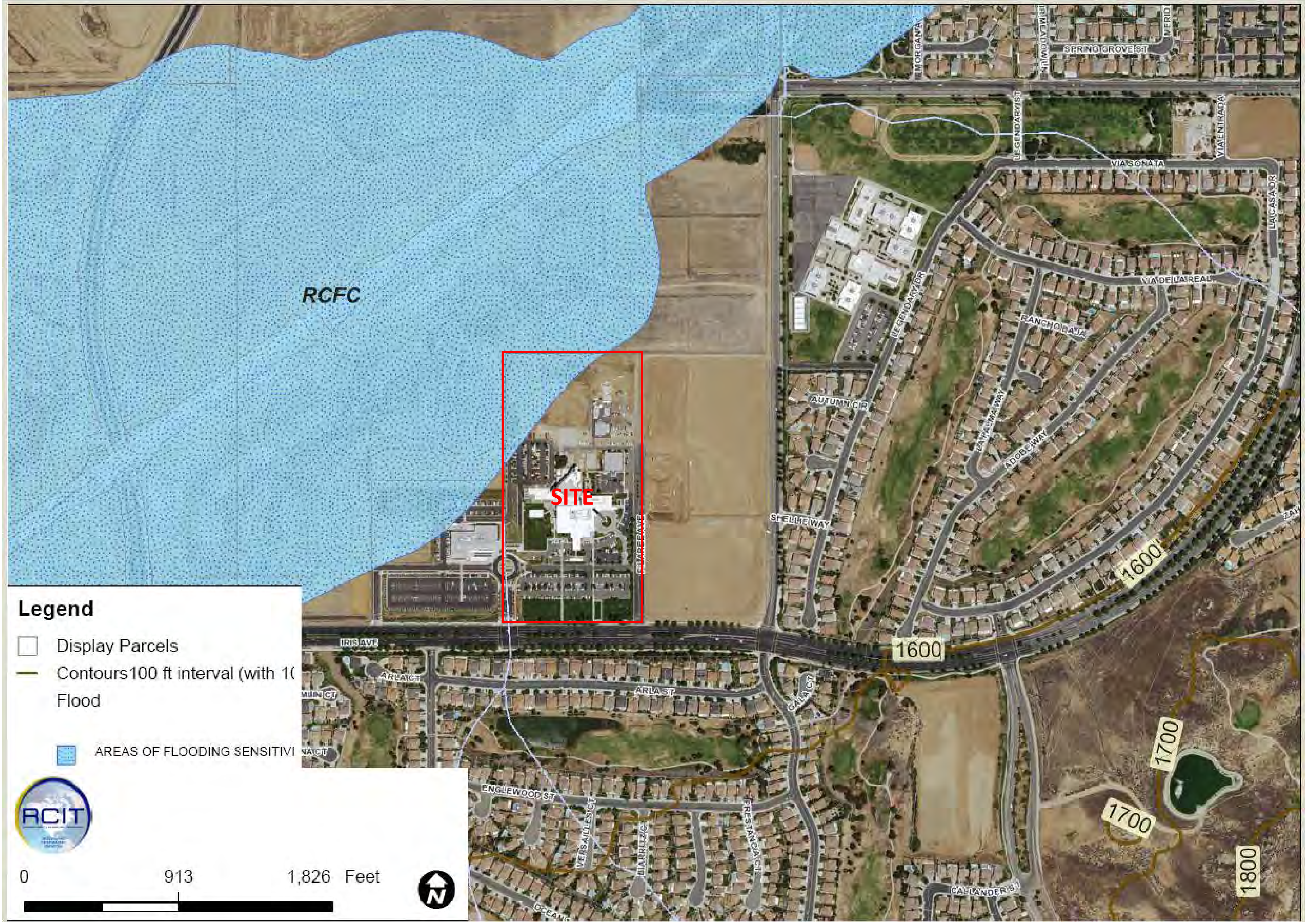
FIGURE A-13



Data from Flood Insurance Rate Maps (FIRMs) where available digitally. New NFHL FIRMette Print app available:

The **SITE** is in Zone X – Area determined to be outside of 0.2% annual chance of floodplain.
Zone A – 1% Annual Chance Flood Hazard

GEOBASE	C.314.81.00	**FEMA FLOOD MAP** **Kaiser Permanente MVMC – D&T BUILDING** **27300 Iris Avenue** **Moreno Valley, California**	**FIGURE A-14** **Page 1 of 2**

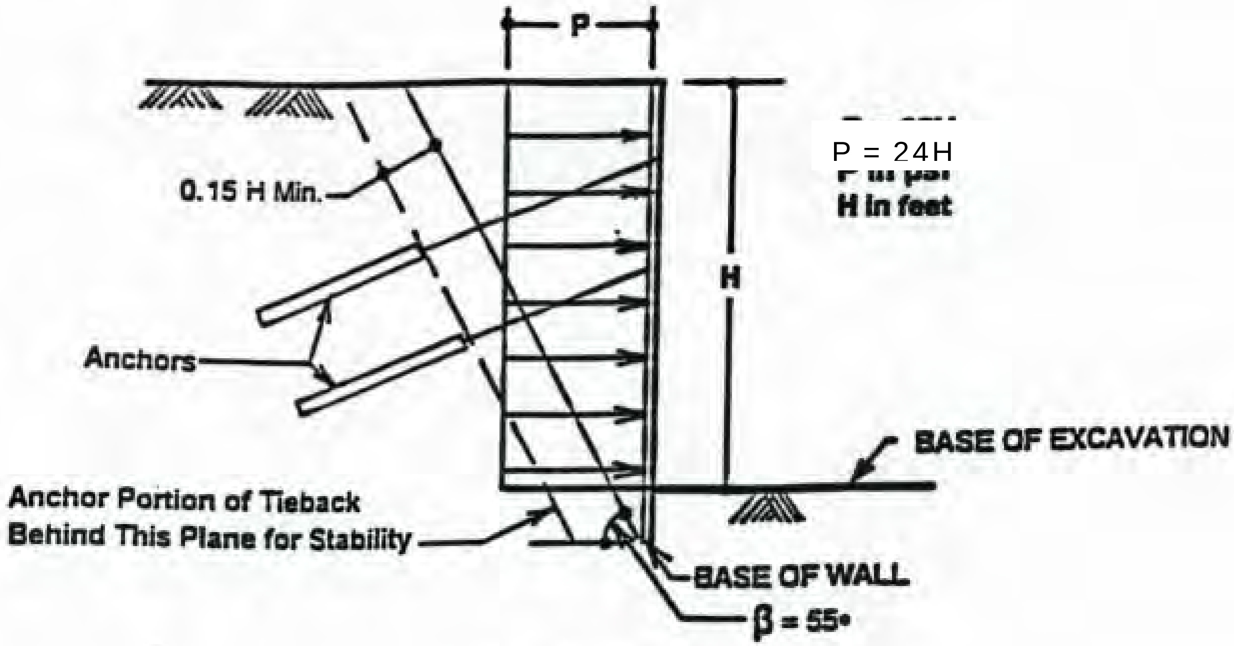


GEOBASE

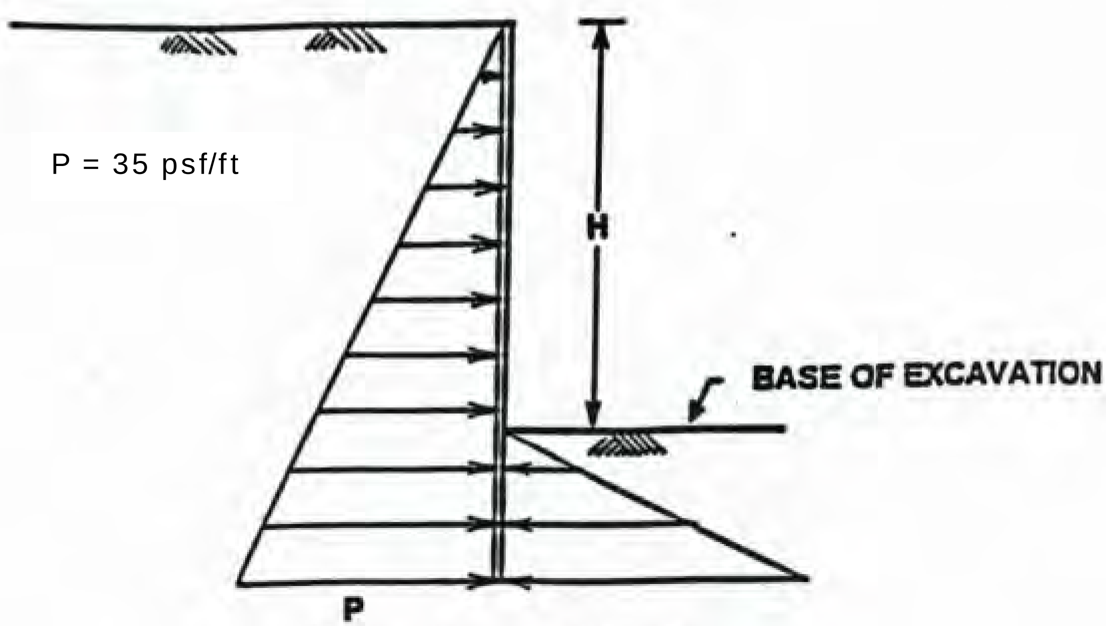
C.314.81.00

RCIT FLOOD MAP
Kaiser Permanente MVMC – D&T BUILDING
27300 Iris Avenue
Moreno Valley, California

FIGURE A-14
Page 2 of 2



a) Tieback Shoring



b) Cantilevered Shoring

GEOBASE

Earth Pressures and Tieback Geometry for Shoring

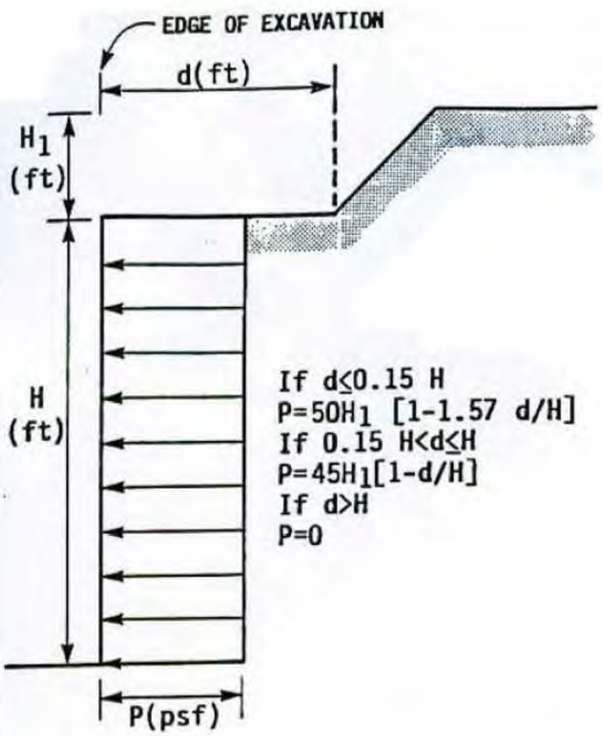
Kaiser Permanente MVMC – D&T BUILDING

27300 Iris Avenue

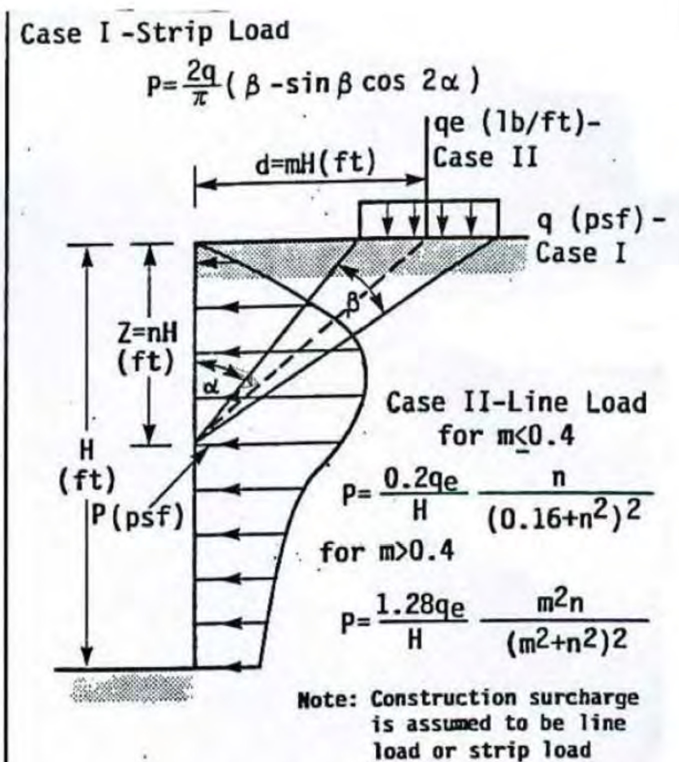
Moreno Valley, California

C.314.81.00

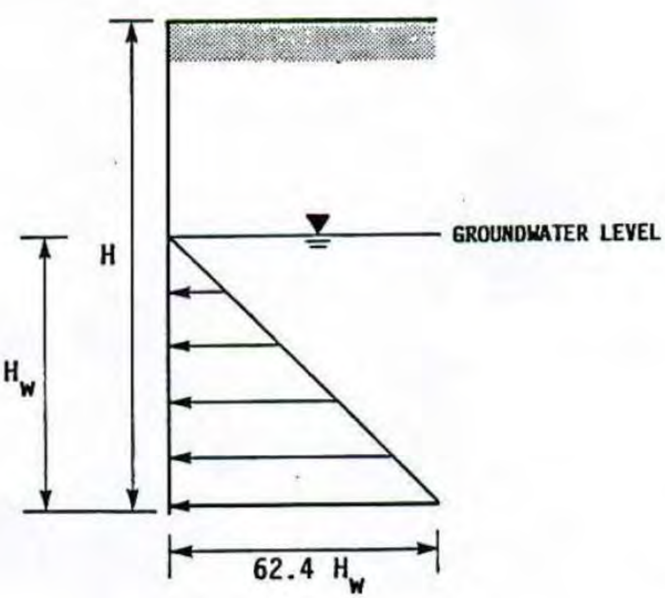
FIGURE A-15



(a) SLOPED EXCAVATION SURCHARGE



(b) CONSTRUCTION AND TRAFFIC SURCHARGE



(c) HYDROSTATIC PRESSURE

GEOBASE	ADDITIONAL LATERAL EARTH PRESSURES ON SHORING Kaiser Permanente MVMC – D&T BUILDING 27300 Iris Avenue C.314.81.00 Moreno Valley, California FIGURE A-16

APPENDIX B

Figure B-1	Explanation of Terms and Symbols
Figure B-2	Log of Boring B-1
Figure B-3	Log of Boring B-2
Figure B-4	Log of Boring B-3
Figure B-5	Log of Boring B-4
Figure B-6	Log of Boring B-5
Figure B-7	Log of Boring B-6
Figure B-8	Log of Boring B-7
Figure B-9	Log of Boring B-8
Figure B-10	Log of Boring B-9
Figure B-11	Log of Boring B-10
Figure B-12	Log of Boring B-11
Figure B-13	Log of CPT-1
Figure B-14	Log of CPT-2
Figure B-15	Log of CPT-3
Figure B-16	Log of CPT-4
Figure B-17	Log of CPT-5
Figure B-18	Log of CPT-6
Figure B-19	Log of CPT-7
Figure B-20	Log of CPT-8
Figure B-21	Log of CPT-9
Figure B-22	Log of CPT-10
Figure B-23	Log of CPT-11
Figure B-24	Log of CPT-12
Figure B-25	Log of CPT-13
Figure B-26	Log of CPT-14
Figure B-27	Log of Test Pit

GEOBASE INC (June 2010)

Figure B-28	Log of Boring B-1
Figure B-29	Log of Boring B-2
Figure B-30	Log of Boring B-4
Figure B-31	Log of CPT-3

GeoVision Geophysical Services, Inc. (July 21, 2017)

The terms and symbols used on the Log of Borings to summarize the results of the field investigation and subsequent laboratory testing are described in the following:

It should be noted that materials, boundaries, and conditions have been established only at the boring locations, and are not necessarily representative of subsurface conditions elsewhere across the site.

A. PARTICLE SIZE DEFINITION (ASTM D2487 AND D422)

Boulder	-- larger than 12-inches	Sand, medium	-- No.40 to No. 10 sieves
Cobble	-- 3-inches to 12-inches	Sand, fine	-- No.200 to No. 40 sieves
Gravel, coarse	-- 3/4-inch to 3-inches	Silt	-- 5µm to No. 200 sieves
Gravel, fine	-- No.4 sieve to 3/4 -inch	Clay	-- smaller than 5 µm
Sand, coarse	-- No.10 to No.4 sieve		

B. SOIL CLASSIFICATION

Soils and bedrock are classified and described according to their engineering properties and behavioral characteristics. The soil of each stratum is described using ASTM D2487 and D2488.

The following adjectives may be employed to define percentage ranges by weight of minor components:

trace	--	1-10%	some	--	20-35%
little	--	10-20%	“and” or ”y”	--	35-50%

The following descriptive terms may be used for stratified soils:

parting	--	0 to 1/16-in. thickness;	layer	--	½-in. to 12-in. thickness;
seam	--	1/16 to ½-in. thickness;	stratum	--	greater than 12-in. thickness.

C. SOIL DENSITY AND CONSISTENCY

The density of coarse grained soils and the consistency of fine grained soils are described on the basis of the Standard Penetration Test:

COARSE GRAINED SOILS		FINE GRAINED SOILS		
DENSITY	SPT BLOWS PER FOOT	ESTIMATED CONSISTENCY	SPT BLOWS PER FOOT	ESTIMATED RANGE OF UNCONFINED COMPRESSIVE STRENGTH (TSF)
very loose	less than 4	very soft	less than 2	less than 0.25
loose	5 to 10	soft	2 to 4	0.25 to 0.50
medium	11 to 30	firm (medium)	5 to 8	0.50 to 1.0
dense	31 to 50	stiff	9 to 15	1.0 to 2.0
very dense	over 50	very stiff	16 to 30	2.0 to 4.0
		hard	over 30	over 4.0

D. STANDARD PENETRATION TEST (SPT) -- D1586

The SPT test involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, 2-inches outside diameter and 1 3/8-inches inside diameter, is driven eighteen (18) inches. The sampler is seated in the first six (6) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value or SPT blow count. The driving energy is provided by a 140 pound weight dropping thirty (30) inches.

E. ABBREVIATION OF LABORATORY TEST DESIGNATIONS

C	Consolidation	pH	pH
CBR	California Bearing Ratio	pp	Pocket Penetrometer
Ch	Water Soluble Chlorides	PS	Particle Size
DS	Direct Shear	RV	R-Value
EI	Expansion Index	SE	Sand Equivalent
ER	Electrical Resistivity	SG	Specific Gravity
k	Permeability	SO_4	Water Soluble Sulfates
MD	Moisture	TX	Triaxial Compression
MP	Modified Proctor Compaction Test	TV	Torvane Shear
O	Organic Content	U	Unconfined Compression

F. STRATIFICATION LINES

The stratification lines indicated on the boring logs and profiles represent the ***approximate*** boundary between material types and the transition may be gradual.

SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

MAJOR DIVISION			GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
HIGHLY ORGANIC SOILS			Pt		Peat and other highly organic soils	Strong color or odor and often fibrous texture
COARSE-GRAINED SOILS (More than half by weight larger than No. 200 sieve size)	GRAVELS (More than half coarse fraction larger than No. 4 sieve size)	CLEAN GRAVELS	GW		Well-graded Gravels, Gravel-Sand mixtures (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			GP		Poorly-graded Gravels and Gravel-Sand mixtures (<5% fines)	Not meeting all above requirements
		DIRTY GRAVELS	GM		Silty Gravels, Gravel-Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or I_p<4
			GC		Clayey Gravels, Gravel-Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or I_p>7
	SANDS (More than half coarse fraction smaller than No. 4 sieve size)	CLEAN SANDS	SW		Well-graded Sands, Gravelly Sands (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			SP		Poorly-graded Sands or Gravelly Sands (<5% fines)	Not meeting all above requirements
		DIRTY SANDS	SM		Silty Sands, Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or I_p<4
			SC		Clayey Sands, Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or I_p>7

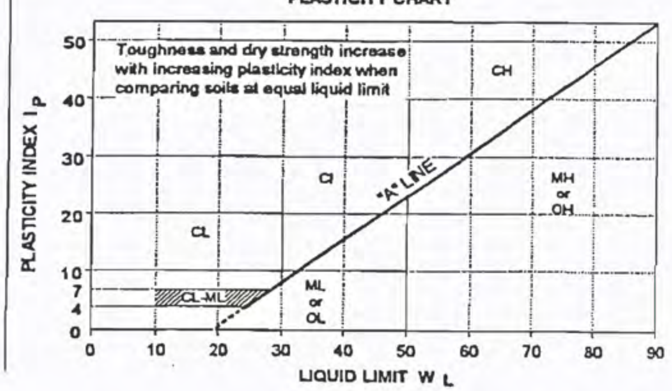
MAJOR DIVISION		GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
FINE-GRAINED SOILS (More than half by weight passes No. 200 sieve size)	SILTS Below "A" line on plasticity chart: negligible organic content	ML		Inorganic Silts and very fine Sands, Rock Flour, Silty Sands of slight plasticity	W_L< 50	See chart below
		MH		Inorganic Silts micaceous or diatomaceous, fine Sandy or Silty soils	W_L> 50	
	CLAYS Above "A" line on plasticity chart: negligible organic content	CL		Inorganic Clays of low plasticity, Gravelly, Sandy, or Silty Clays, lean Clays	W_L< 30	
		CI		Inorganic Clays of medium plasticity, Silty Clays	W_L> 30, <50	
		CH		Inorganic Clays of high plasticity, fat Clays	W_L> 50	
	ORGANIC SILTS & ORGANIC CLAYS Below "A" line on plasticity chart	OL		Organic Silts and organic Silty Clays of low plasticity	W_L< 50	
		OH		Organic Clays of high plasticity	W_L> 50	

The soil of each stratum is described using ASTM D2487 and D2488 modified slightly so that an inorganic clay of "medium plasticity" is recognized.

ADDITIONAL SOIL CLASSIFICATION

- Fill Soil
- Ss Sandstone
- Cs Claystone
- Ms Siltstone

PLASTICITY CHART



GEOBASE

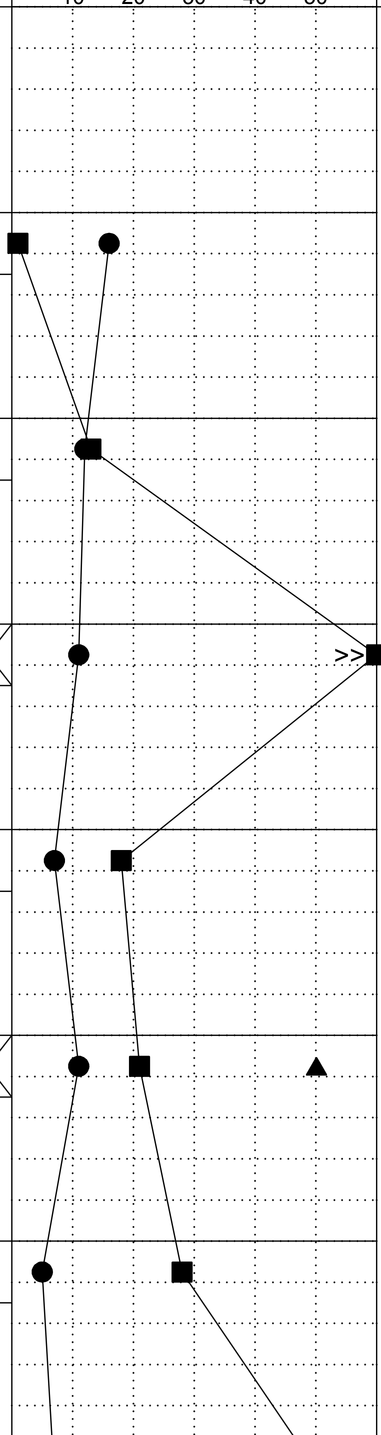
EXPLANATION OF TERMS AND SYMBOLS USED

Figure B-1

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%); Plastic Limit (W_P) – Liquid Limit (W_L); Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
0	GRASS AND ROOTS, GRASS			
	SAND (FILL), brown, clayey, very loose.			
5	SILT, brown, fined-sands, little clay, very soft.	ML		N = PUSH 200 Wash
10	...stiff			
	SAND, brown, fine- to medium-grained, some silt.	SM		
15	...coarse-grained, little gravels, very dense		>>	Blowcount = 94/12 in.
20	...medium- to coarse-grained, silty, medium dense			
25	...trace of gravels, medium dense			Blowcount = 32/12 in.
30	...poorly graded, trace to little silt, little gravels, medium dense			
35				



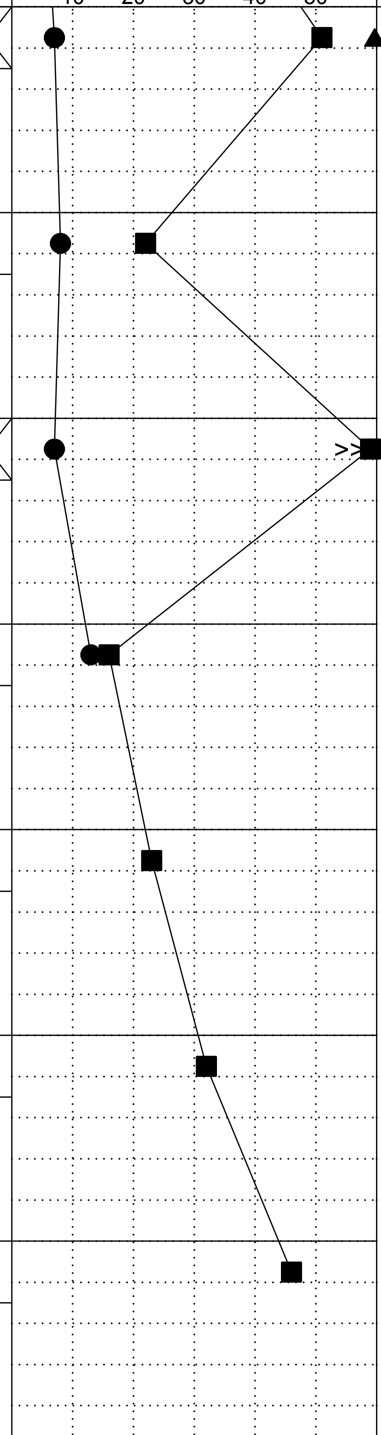
GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA	BORING NO. **B-1**
	DEPTH TO WATER feet	SURFACE ELEV. **1526 feet** — LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** DRILLER **Martini Drilling** — DATE LOGGED **06/07/2017**	FIGURE NO. B- **2**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%); Plastic Limit (W_P) – Liquid Limit (W_L); Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, light brown, medium- to coarse-grained, trace to little silt, little gravels, very dense.	SM	California Modified Sampler		Blowcount = 78/12 in. 200 Wash
40		...silty, medium dense		SPT		200 Wash
45		...coarse-grained, some silt and fine-gravels, very dense		California Modified Sampler	>>	Blowcount = 90/12 in. 200 Wash
50		SAND, brown, little silt, trace of gravel, medium dense.	SM	SPT		
		SAND, light brown, medium- to coarse-grained, little pea gravels, trace of silt, medium dense.	SP			
55		...minor seepage at 56.5 ft		SPT		
		SAND, light brown, medium- to coarse grained, some gravels, dense	SM			
60				SPT		PS
65		...little silt, trace of gravels, dense		SPT		PS
70		End of Boring at 66.5 feet. Boring dry at completion of drilling.. Backfilled with soil cuttings.				



GEOBASE, INC.	PROJECT	**KP Moreno Valley Medical Center** **27300 Iris Avenue, Moreno Valley, CA**	**BORING NO. B-1**
	DEPTH TO WATER feet	SURFACE ELEV. **1526 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** DRILLER **Martini Drilling** / DATE LOGGED **06/07/2017**	FIGURE NO. B- **2**

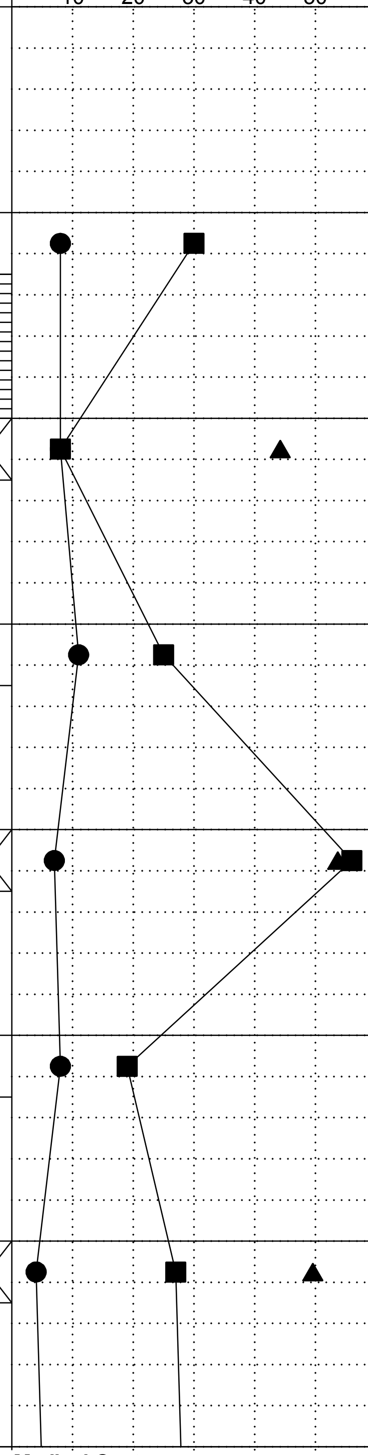
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DRY DENSITY (PCF): 80, 90, 100, 110, 120
Water Content (%): ●
Plastic Limit (W_P) — Liquid Limit (W_L)
Penetration, blows/foot: ■ 10, 20, 30, 40, 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
0	GRASS AND ROOTS, GRASS		
	SILT (FILL), brown, little sand and clay, soft.		
5	...some sands, very stiff		
	SAND, brown, fine- to medium-grained, sandy SILT with little clay.	SM-ML	Bulk Sample 5-10 ft. EI, MP, 95 RC, Ch, ER, pH, SO_4
10	...firm, some clay		Blowcount = 11/12 in. C, DS
15	...coarse-grained, little gravels, medium dense.		
	SAND, brown,.medium- to coarse-grained, silty, cementation, very dense	SM	
20			Blowcount = 86/12 in. C, DS
25	...trace of gravels, medium dense		
30	...poorly graded, trace to little silt, little gravels, medium dense		Blowcount = 41/12 in. C, DS
35			



GEOBASE, INC.

PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-2
DEPTH TO WATER feet	SURFACE ELEV. 1535 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/07/2017	FIGURE NO. B-3

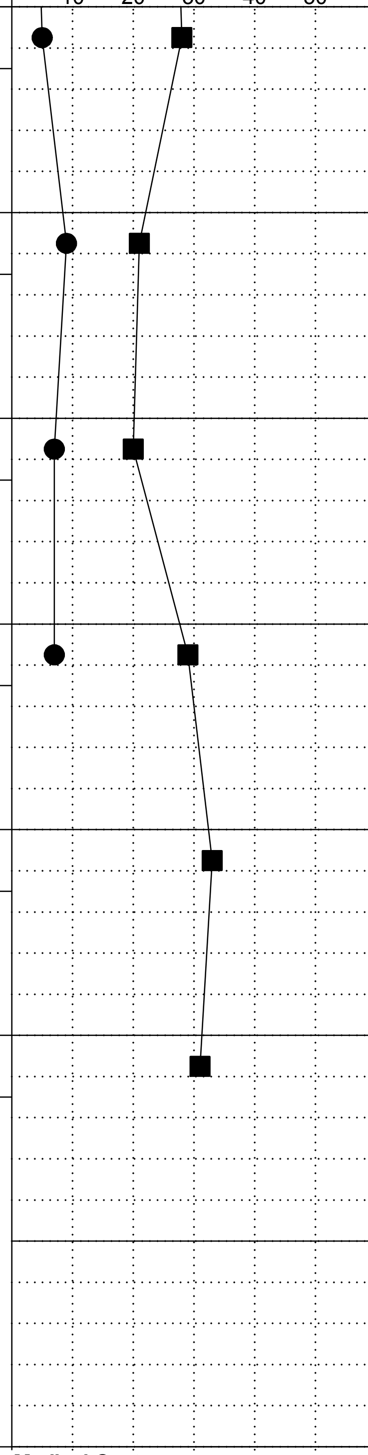
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

Legend: DRY DENSITY (PCF) ▲ 80, 90, 100, 110, 120; Water Content (%): ●; Plastic Limit (W_P) ├──┤ Liquid Limit (W_L); Penetration, blows/foot: ■ 10, 20, 30, 40, 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	SAND, light brown, medium- to coarse-grained, trace to little silt, little gravels, medium dense.	SP-SM	
40	...silty, white streak rock		
45	...silty		200 Wash
50	...fine-grained, silty		
55	...dense, minor seepage at 56 ft		
60	SAND, light brown, coasrse-grained, trace of silt, little gravel, dense.	SP	
65	End of Boring at 66.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.		
70			



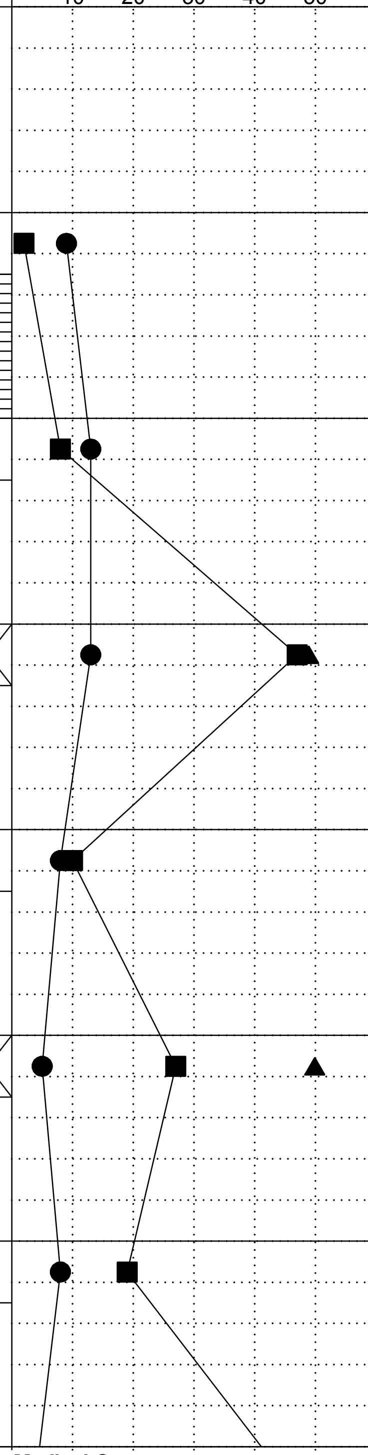
GEOBASE, INC.	PROJECT: KP Moreno Valley Medical Center, 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-2
	DEPTH TO WATER feet	SURFACE ELEV. 1535 feet / LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT / DRILLER Martini Drilling / DATE LOGGED 06/07/2017	FIGURE NO. B- 3

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
0	GRASS AND ROOTS, GRASS		
	SAND (FILL), brown, silty, very loose.		
5	SILT, brown, some fine-grained sand, soft.	ML	Bulk Sample 5-10 ft. EI, MP, 95 RC Ch, ER, pH, SO_4
10	SAND, brown, fine-grained, silty, loose.	SM	
	SILT, brown, little fine-grained sand, cementation, hard.	ML	
15			Blowcount = 72/12 in. DS, 200 Wash
	SAND, ...medium- to coarse-grained, silty.	SM	
20	...some silt, loose		
25	...trace of gravels, medium dense		Blowcount = 42/12 in.
30	...micaceous, fine- to medium-grained, silty and little gravels		
35			



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA	BORING NO. **B-3**
	DEPTH TO WATER feet	SURFACE ELEV. **1525 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** DRILLER **Martini Drilling** / DATE LOGGED **06/07/2017**	FIGURE NO. B- **4**

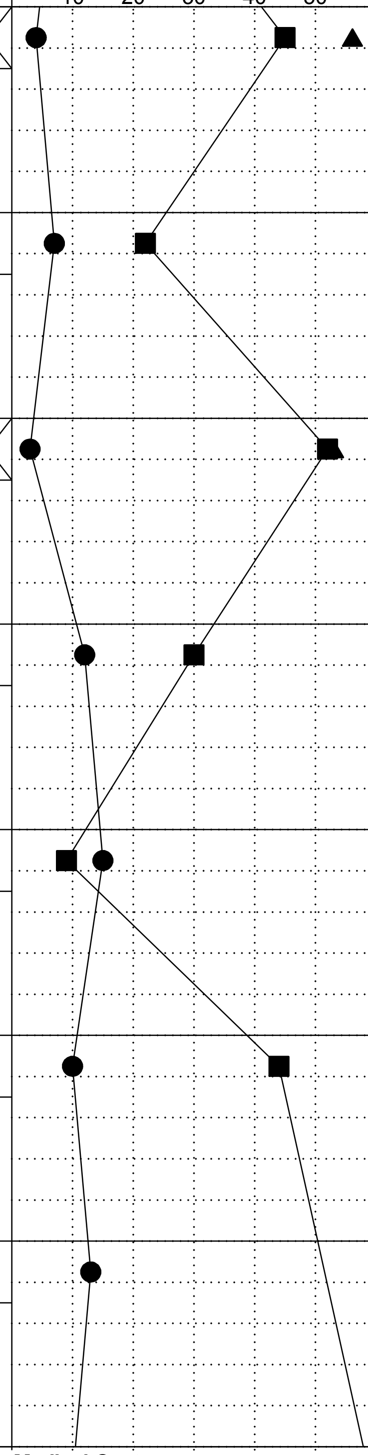
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
	SAND, light brown, medium- to coarse-grained, siltyt, trace of gravels, dense.	SM	California Modified Sampler	Blowcount = 69/12 in. DS, 200 Wash
40	...medium dense		SPT	
45	SAND, light brown, trace of silt and gravel, dense to very dense.	SP	California Modified Sampler	Blowcount = 80/12 in.
50	...coasrse-grained, little gravels, dense ...minor seepage at 51 ft		SPT	
55	SILT, brown, little sands, stiff.	ML	SPT	
60	SAND, brown, medium-grained, little silt and trace of gravel, dense.	SM	SPT	
65	SAND, brown, coarse-grained, little gravel, trace of silt, medium dense.	SP	SPT	
70				

DRY DENSITY (PCF): 80, 90, 100, 110, 120
Water Content (%): ●
Plastic Limit (W_P) — Liquid Limit (W_L)
Penetration, blows/foot: ■ 10, 20, 30, 40, 50



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA	BORING NO. B-3
	DEPTH TO WATER feet	SURFACE ELEV. 1525 feet | LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling | DATE LOGGED 06/07/2017	FIGURE NO. B- 4

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE; ◫ SPT SPLIT SPOON; ☒ CALIFORNIA MODIFIED SAMPLER; ☰ DISTURBED; ◪ NO RECOVERY; ▦ CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) ▲ 80 90 100 110 120; Water Content (%): ●; Plastic Limit (W_P) ├─┤ Liquid Limit (W_L); Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, brown, trace of silt, some gravels, very dense.	SP			
75		End of Boring at 71.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.				
80						
85						
90						
95						
100						
105						

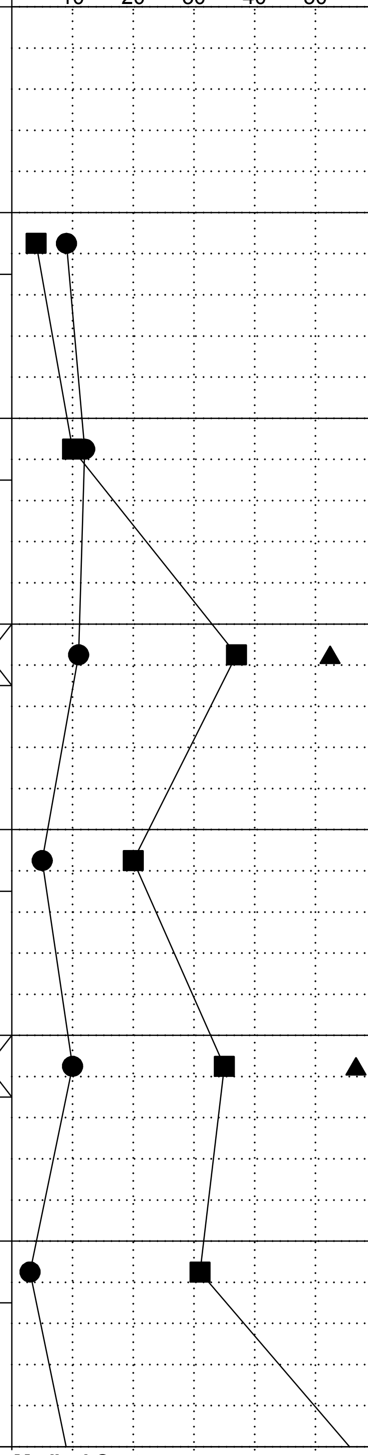
GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-3
	DEPTH TO WATER feet	SURFACE ELEV. 1525 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/07/2017	FIGURE NO. B-4

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) ▲ 80 90 100 110 120; Water Content (%): ●; Plastic Limit (W_P) ├─┤ Liquid Limit (W_L); Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
0		SAND (FILL), brown, silty, very loose.	SM			
5		SILT, brown, little fine- to coarse-grained sand, soft.	ML	SPT		
10		...stiff		SPT		
12		SAND, brown, silty, medium- to coarse grained, micaceous.	SM			
15		...dense		California		Blowcount = 37/12 in. C, DS, 200 Wash
20		...fine- to medium grained, little silt, medium dense		SPT		200 Wash
25		...silty, dense		California		Blowcount = 54/12 in.
30		SAND, light brown, fine- to medium grained, little silt, micaceous, dense.	SP	SPT		
35						



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. **B-4**
	DEPTH TO WATER feet	SURFACE ELEV. **1526 feet**	LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** DRILLER **Martini Drilling**	DATE LOGGED **06/08/2017**	FIGURE NO. B- **5**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

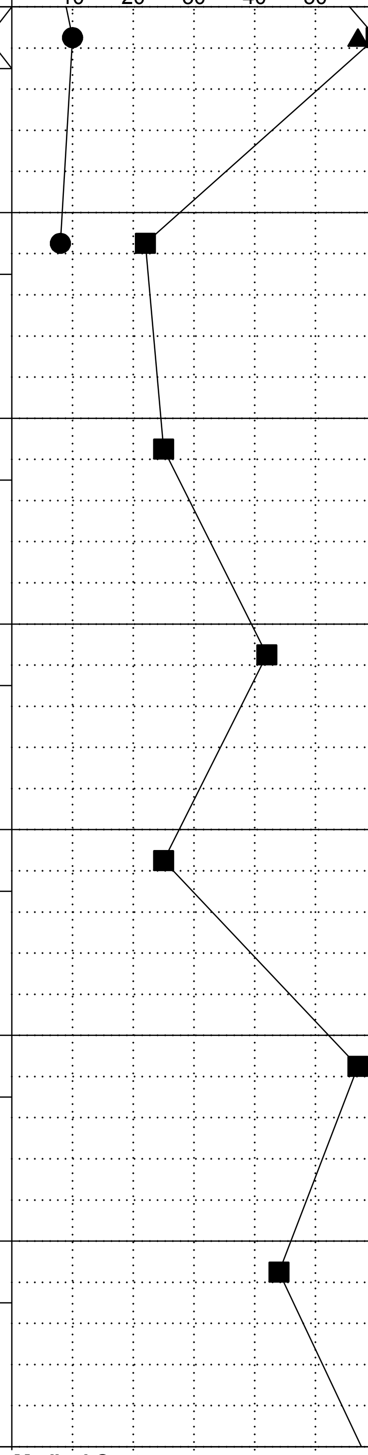
DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
	SAND, brown, medium- to coarse-grained, little silt, very dense.	SP-SM	California Modified Sampler	Blowcount = 91/10 in.
40	...silty medium dense		SPT	
45	...fine-grained, silty, medium dense		SPT	
50	...little silt, trace of gravel, dense		SPT	PS
55	SAND, brown, silty, fine-grained, medium dense. ...minor seepage at 56 ft	SM	SPT	
60	SAND, light brown, coarse-grained, trace of silt, very dense.	SP	SPT	
65	...fine- to medium-grained, dense		SPT	
70				

DRY DENSITY (PCF): 80, 90, 100, 110, 120

Water Content (%)

Plastic Limit (W_P) — Liquid Limit (W_L)

Penetration, blows/foot: 10, 20, 30, 40, 50



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-4
	DEPTH TO WATER feet	SURFACE ELEV. 1526 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/08/2017	FIGURE NO. B- 5

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%); Plastic Limit (W_P) / Liquid Limit (W_L); Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, brown, coarse grained, little silt, trace of fined-gravels, very dense.	SM			N = 79, PS
75		End of Boring at 71.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.				
80						
85						
90						
95						
100						
105						

GEOBASE, INC.	PROJECT: **KP Moreno Valley Medical Center, 27300 Iris Avenue, Moreno Valley, CA**		BORING NO. **B-4**
	DEPTH TO WATER feet	SURFACE ELEV. **1526 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** / DRILLER **Martini Drilling** / DATE LOGGED **06/08/2017**	FIGURE NO. B- **5**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

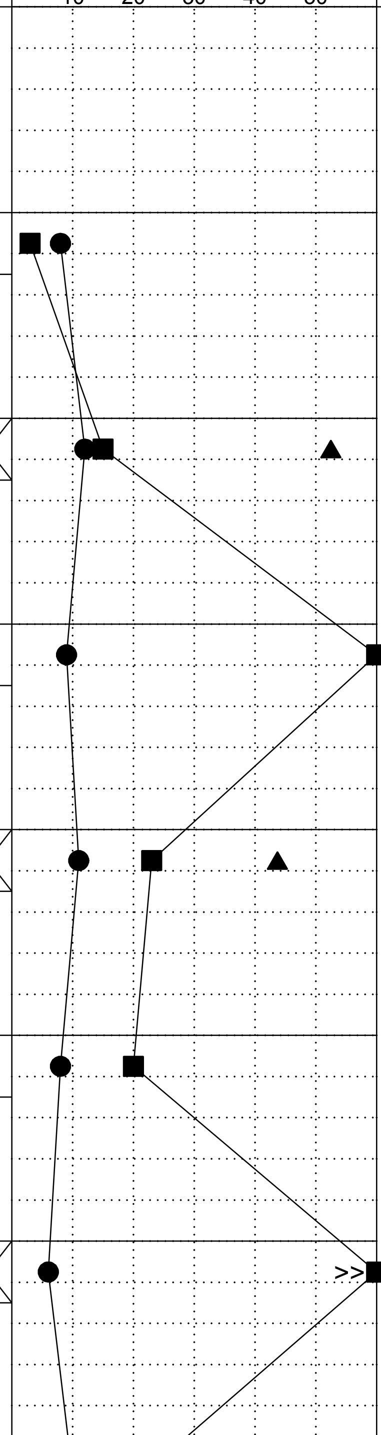
DRY DENSITY (PCF): 80 90 100 110 120

Water Content (%)

Plastic Limit (W_P) — Liquid Limit (W_L)

Penetration, blows/foot: 10 20 30 40 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
0	SAND (FILL), brown, silty, very loose.	SM	
5	SILT, brown, fine-grained sands, soff	ML	
10	...sandy, stiff		Blowcount = 23/12 in.
12	SAND, brown, silty, cementation, micaceous, trace of gravels.	SM	
15	...white streak, very dense		N = 83
18	SILT, ...some sands, very stiff.	ML	
20			Blowcount = 35/12 in.
21.5	SAND, brown, silty, fine-grained, medium dense.	SM	
25			200 Wash
30	...light brown, silty, very dense.		Blowcount = 98/11 in. 200 Wash
35			



GEOBASE, INC.	PROJECT: KP Moreno Valley Medical Center, 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-5
	DEPTH TO WATER feet	SURFACE ELEV. 1527 feet / LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT / DRILLER Martini Drilling / DATE LOGGED 06/08/2017	FIGURE NO. B- 6

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

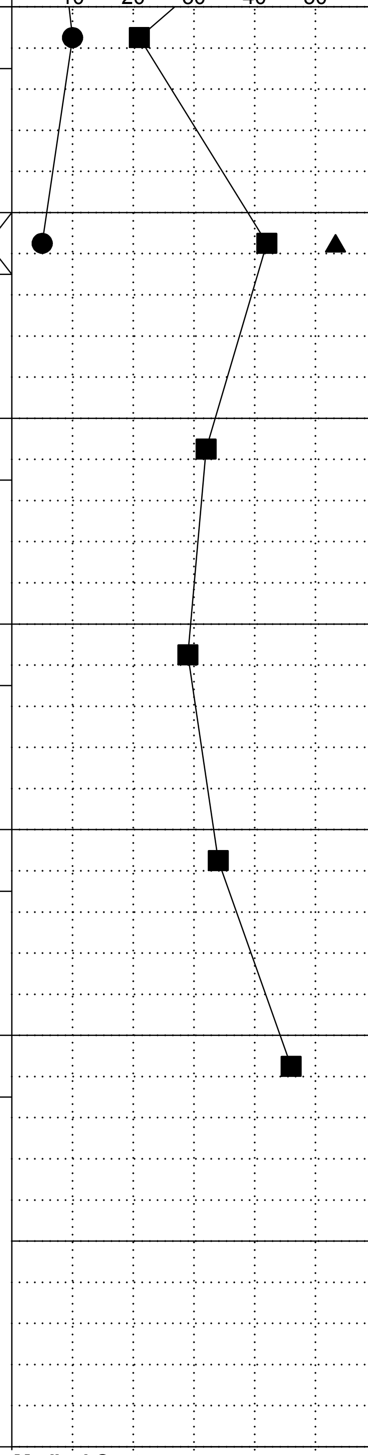
DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
	SAND, brown, medium- to coarse-grained, silty, medium dense.	SM	SPT	
40	...dense		CALIFORNIA MODIFIED SAMPLER	Blowcount = 65/12 in.
45	...fine-grained, little gravels, dense		SPT	
50	SAND, light brown, fine- to medium-grained, little fine-gravels, medium dense.	SP	SPT	
55	...silty, dense ...minor seepage at 56 ft		SPT	
60	SAND, light brown, coarse-grained, little fine-gravel, dense.	SP	SPT	
65	End of Boring at 61.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.			
70				

DRY DENSITY (PCF): 80 90 100 110 120

Water Content (%): ●

Plastic Limit (W_P) — Liquid Limit (W_L)

Penetration, blows/foot: ■ 10 20 30 40 50



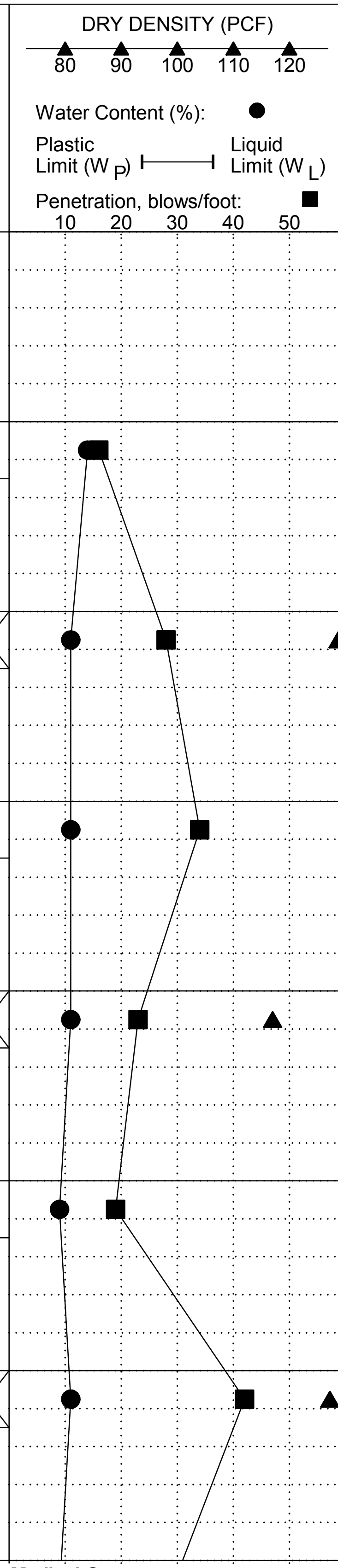
GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-5
	DEPTH TO WATER feet	SURFACE ELEV. 1527 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/08/2017	FIGURE NO. B- 6

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) / Water Content (%) / Plastic Limit (W_P) – Liquid Limit (W_L) / Penetration, blows/foot	REMARKS/ OTHER TESTS
0		SAND (FILL), brown, silty, very loose.	SM			
5		SILT, brown, fine-grained sands, stiff.	ML	SPT		
10		...some sands, very stiff		California		Blowcount = 43/12 in.
15		...little micaceous sands, white streak, cementation,, hard.		SPT		
20		...some sands, very stiff.		California		Blowcount = 35/12 in.
		SAND, brown, silty, fine-grained, medium dense.	SM			
25				SPT		
30		...little fine-gravel, dense		California		Blowcount = 64/12 in.
35						



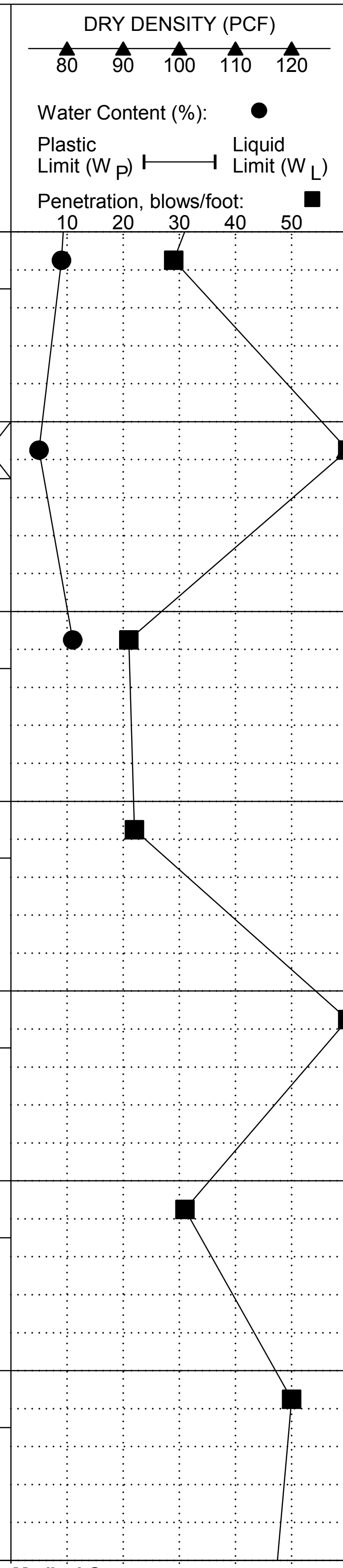
GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. **B-6**
	DEPTH TO WATER feet	SURFACE ELEV. **1520 feet**	LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** DRILLER **Martini Drilling**	DATE LOGGED **06/08/2017**	FIGURE NO. B- **7**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	SAND, brown, medium- to coarse-grained, little silt and gravels, medium dense.	SM	
40	...well-graded, trace of silt, some fine-gravels, very dense		Blowcount = 85/11 in.
45	...fine-grained, little gravels, medium dense		
50	SILT, brown, some sands, very stiff. ...minor seepage at 51.5 ft	ML	
55	SAND, light brown, coarse-grained, micaceous, fined-gravels, very dense.	SW	N = 66 PS
60	SAND, brown, coarse-grained, little fine-gravel and silt, dense.	SM	
65	SAND, light brown, coarse-grained, trace of silt and gravels, very dense.	SP	PS
70			



GEOBASE, INC.	PROJECT: **KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA**		BORING NO. **B-6**
	DEPTH TO WATER feet	SURFACE ELEV. **1520 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** DRILLER **Martini Drilling** / DATE LOGGED **06/08/2017**	FIGURE NO. B- **7**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 / Water Content (%) / Plastic Limit (W_P) – Liquid Limit (W_L) / Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, brown, silty, trace of gravels, dense.	SM			
75		End of Boring at 71.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.				
80						
85						
90						
95						
100						
105						

GEOBASE, INC.	PROJECT **KP Moreno Valley Medical Center, 27300 Iris Avenue, Moreno Valley, CA**		BORING NO. **B-6**
	DEPTH TO WATER feet	SURFACE ELEV. **1520 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** / DRILLER **Martini Drilling** / DATE LOGGED **06/08/2017**	FIGURE NO. B- **7**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

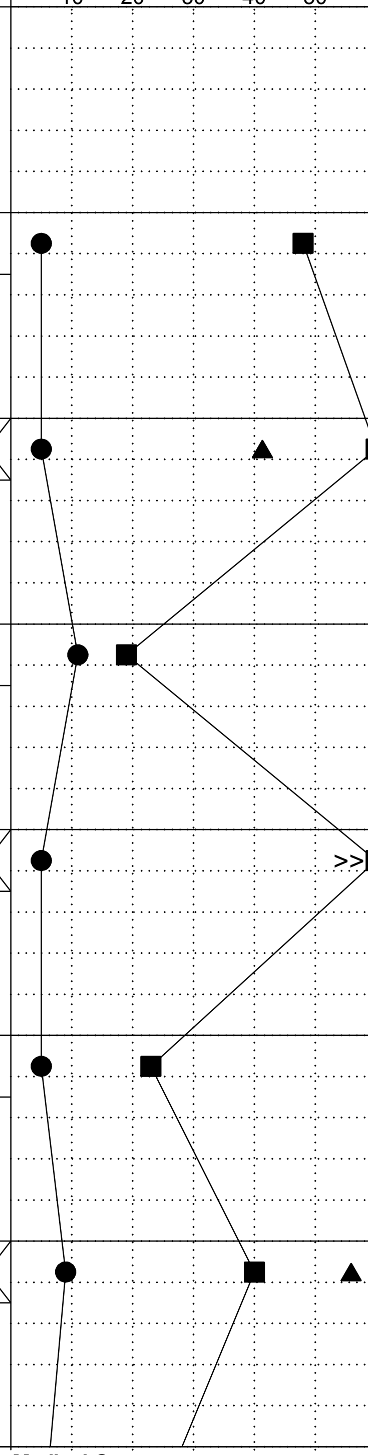
DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
		GRASS AND ROOTS,			
		SAND, brown, silty, very loose.	SM		
5		SILT, brown, little fine-grained sands, hard.	ML	SPT	
10		...cementation, sandy, hard		California	Blowcount = 50/5 in. C, DS
15		...trace of sands, stiff		SPT	
20		SAND, brown, silty, fine-grained, very dense.	SM	California	Blowcount = 85/11 in.
25		...micaceous, some silt, medium dense		SPT	
30		...silty, dense		California	Blowcount = 60/12 in.
35					

DRY DENSITY (PCF): 80 90 100 110 120

Water Content (%): ●

Plastic Limit (W_P) — Liquid Limit (W_L)

Penetration, blows/foot: ■ 10 20 30 40 50



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-7
	DEPTH TO WATER feet	SURFACE ELEV. 1517 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/08/2017	FIGURE NO. B- 8

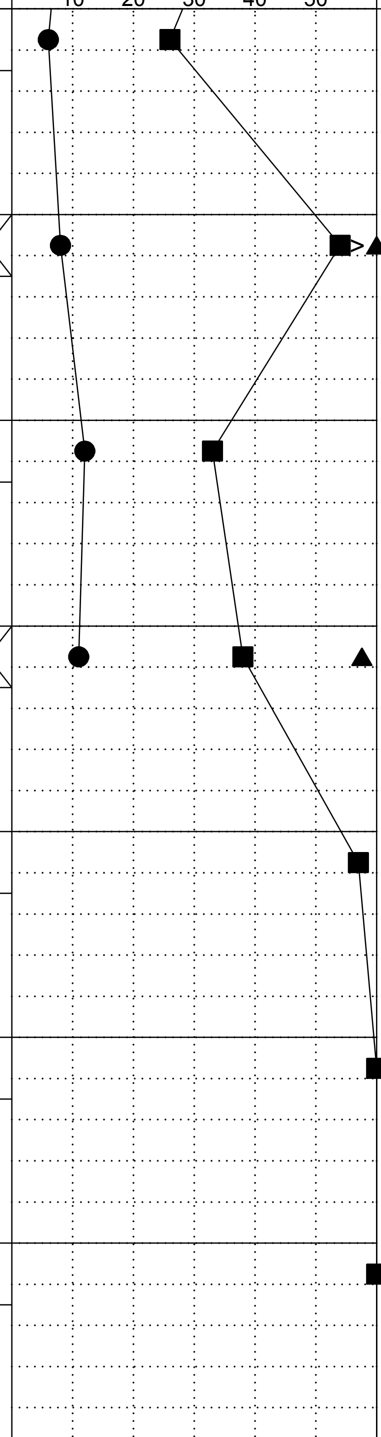
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DRY DENSITY (PCF): 80 90 100 110 120
Water Content (%): ●
Plastic Limit (W_P) — Liquid Limit (W_L)
Penetration, blows/foot: ■ 10 20 30 40 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
35	SAND, brown, medium- to coarse-grained, little silt and gravels, medium dense.	SM	PS
40	...little silt, trace of fine-gravels, very dense		Blowcount = 83/12 in.
45	SAND, light, brown, little fine-gravels, trace of silt, dense.	SP	
50	SAND, brown, some silt, little gravels, dense.	SM	Blowcount = 54/12 in.
	...minor seepage at 51.5 ft		
55	SAND, light brown, micaceous, litte silt and fine-gravels, very dense.	SP-SM	
60	...silty, little gravels, very dense		
65	...coarse-grained to fine-gravels		N = 79
70			



GEOBASE, INC.

PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-7
DEPTH TO WATER feet	SURFACE ELEV. 1517 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/08/2017	FIGURE NO. B- 8

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%); Plastic Limit (W_P) – Liquid Limit (W_L); Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, brown, little silt, some fine-gravels, very dense.	SP			N = 74
75		End of Boring at 71.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.				
80						
85						
90						
95						
100						
105						

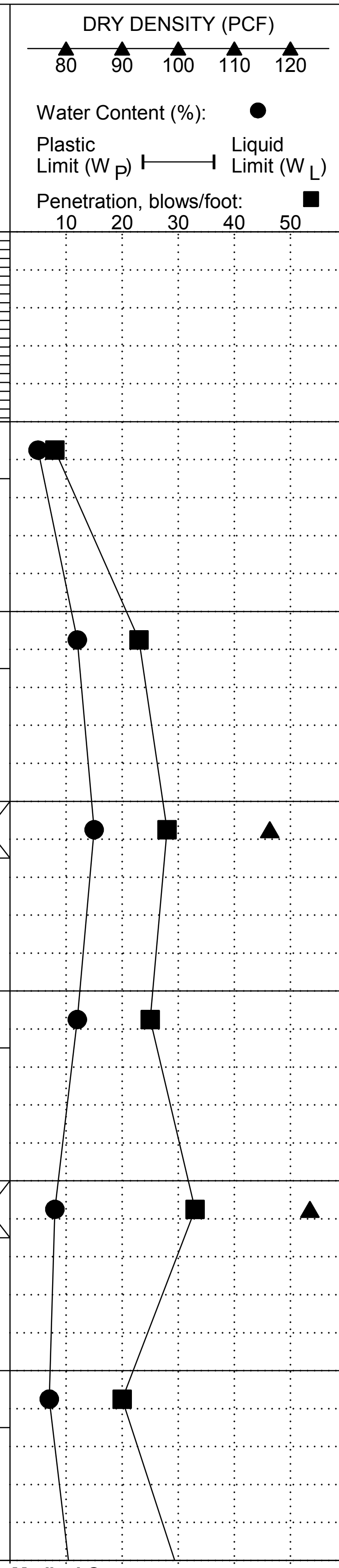
GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-7
	DEPTH TO WATER feet	SURFACE ELEV. 1517 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/08/2017	FIGURE NO. B- 8

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
0	GRASS AND ROOTS,		
	SAND, brown, silty, very loose.	SM	Bulk Sample 0-5 ft. EI, MP, RV, 200 Wash, 95 RC, Ch, ER, pH, SO_4
5	SILT, brown, some fine-grained sands, firm.	ML	
10	...little sands, very stiff		
15	...trace of sand		Blowcount = 43/12 in.
20	...little sands		
25	SAND, brown, fine-grained, silty, dense.	SM	Blowcount = 50/12 in.
30	...fine-grained, medium dense		
35			



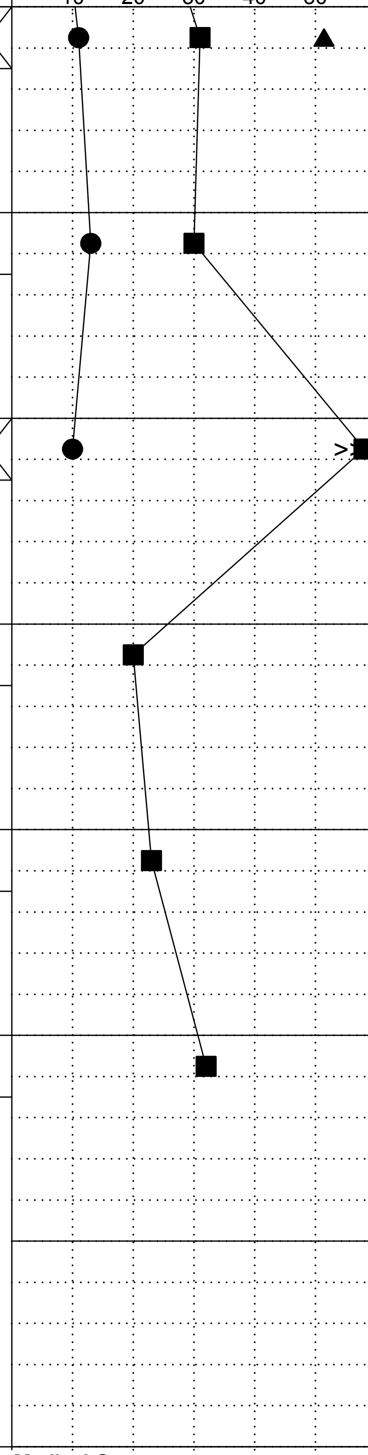
GEOBASE, INC.	PROJECT: **KP Moreno Valley Medical Center, 27300 Iris Avenue, Moreno Valley, CA**		BORING NO. **B-8**
	DEPTH TO WATER feet	SURFACE ELEV. **1514 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** / DRILLER **Martini Drilling** / DATE LOGGED **06/09/2017**	FIGURE NO. B- **9**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%); Plastic Limit (W_P) – Liquid Limit (W_L); Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, brown, medium- to coarse-grained, little silt and gravels, dense.	SM			Blowcount = 48/12 in.
40		...light gray, fine- to medium-grained				
45		... little fine-gravels and silt, very dense.			>	Blowcount = 89/12 in. PS
50		brown, some silt, trace of gravels, medium dense.				
55		...silty ...minor seepage at 56 ft				
60		...silty, fine- to medium-grained, dense				
65		End of Boring at 61.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.				
70						



GEOBASE, INC.			
PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-8
DEPTH TO WATER feet	SURFACE ELEV. 1514 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/09/2017	FIGURE NO. B- 9

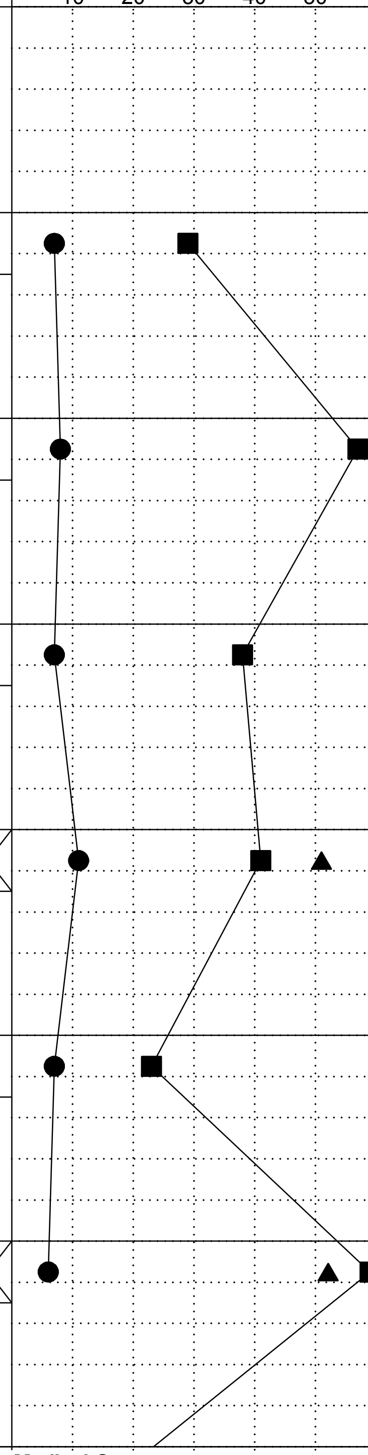
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE; SPT SPLIT SPOON; CALIFORNIA MODIFIED SAMPLER; DISTURBED; NO RECOVERY; CORE

DRY DENSITY (PCF): 80, 90, 100, 110, 120
Water Content (%)
Plastic Limit (W_P) — Liquid Limit (W_L)
Penetration, blows/foot: 10, 20, 30, 40, 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
0	GRASS AND ROOTS,			
	SAND (FILL), brown, silty, very loose.	SM		
	SILT, brown, little fine-grained sands.	ML		
5	...very stiff		SPT	
10	...some sands, cementation, hard		SPT	
15	...hard		SPT	
20	SAND, brown, silty, fine-grained, dense.	SM	California	Blowcount = 63/12 in. C, DS
25	...silty, trace of gravels, medium dense		SPT	200 Wash
30	...little fine-gravels, very dense.		California	Blowcount = 80/11 in.
35				



GEOBASE, INC.	PROJECT **KP Moreno Valley Medical Center, 27300 Iris Avenue, Moreno Valley, CA**		BORING NO. **B-9**
	DEPTH TO WATER feet	SURFACE ELEV. **1516 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** / DRILLER **Martini Drilling** / DATE LOGGED **06/09/2017**	FIGURE NO. B- **10**

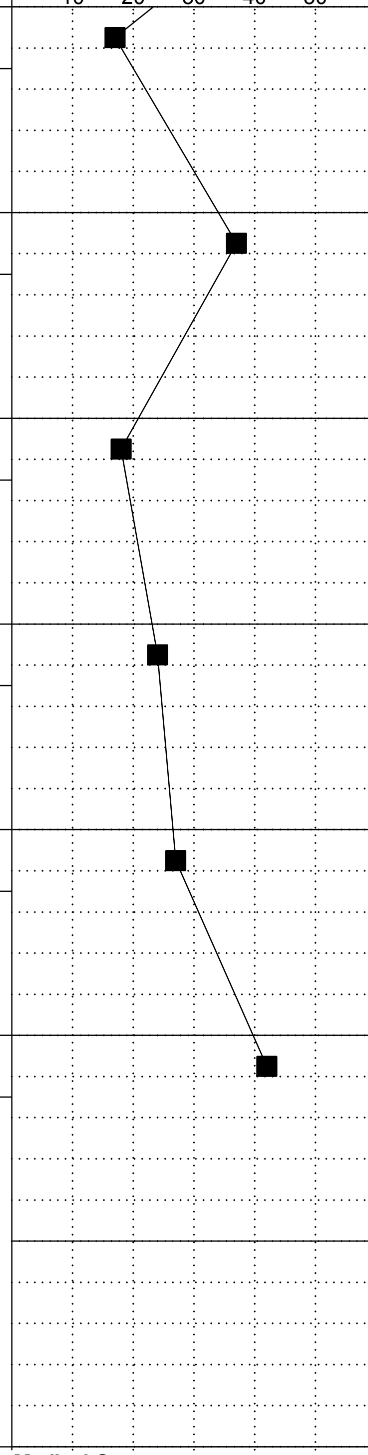
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DRY DENSITY (PCF): 80, 90, 100, 110, 120
Water Content (%): ●
Plastic Limit (W_P) — Liquid Limit (W_L)
Penetration, blows/foot: ■ 10 20 30 40 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	SAND, brown, some gravels, little silt, medium dense	SM	
40	SAND, light brown, fine-grained, trace of silt, dense.	SP	
45	SAND, brown, little to some silt, medium dense.	SM	
50	...silty,		
	...seepage at 51.5 ft		
55	...medium-grained		
60	SAND, light brown, coarse-grained, some gravels, trace of silt, dense.	SW	PS
65	End of Boring at 61.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.		
70			



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-9
	DEPTH TO WATER feet	SURFACE ELEV. 1516 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/09/2017	FIGURE NO. B-10

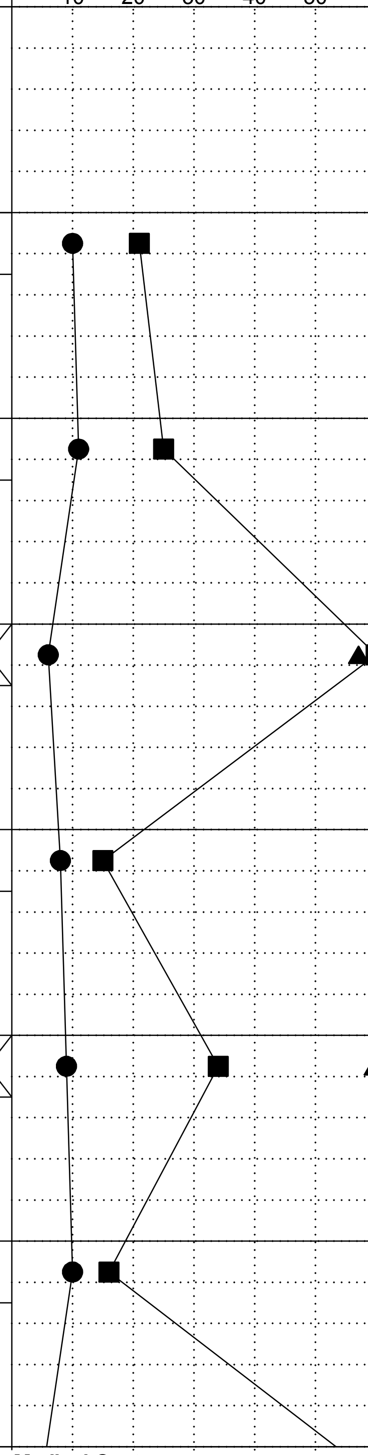
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DRY DENSITY (PCF): 80, 90, 100, 110, 120
Water Content (%): ●
Plastic Limit (W_P) — Liquid Limit (W_L)
Penetration, blows/foot: ■ 10, 20, 30, 40, 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
0	SAND (FILL), brown, silty, medium dense	SM	Bulk Sample 0-5 ft. EI, RV, 200 Wash, Ch, ER, pH, SO_4
5	SILT, brown, little fine-grained sands, very stiff.	ML	200 Wash
10	...very stiff		200 Wash
13	SAND, brown, silty interlayers, fine- to medium-grained, cementation.	SM	
15	...very dense layer.		Blowcount = 100/11 in. 200 Wash, C, DS
20	...fine-grained, medium dense		
25	...medium-grained, dense		Blowcount = 52/12 in. C, DS
30	...silty, little fine-gravels, medium dense.		200 Wash
35			



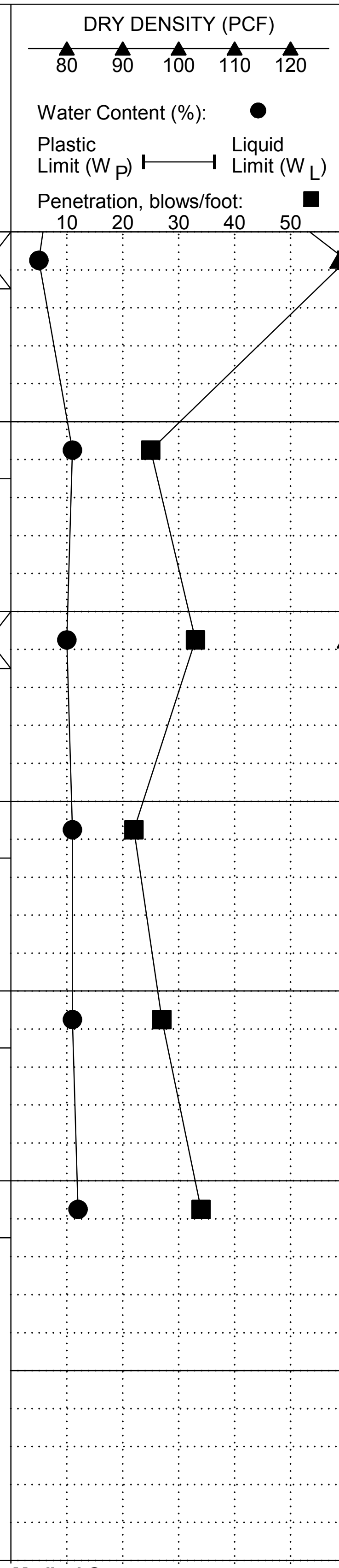
GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA	BORING NO. **B-10**
	DEPTH TO WATER feet	SURFACE ELEV. **1517 feet** / LOGGED BY **HDN**	PROJECT NO. **C.314.81.00**
	DEPTH TO SLOUGH	DRILL RIG **CME-75 HT** DRILLER **Martini Drilling** / DATE LOGGED **06/09/2017**	FIGURE NO. B- **11**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	<u>SAND</u>, light brown, silty interlayer, coarse-grained, trace of fine-gravels, very dense.	SM	Blowcount = 85/11 in. PS
40	...some silt, some fine-gravels up to 1/2 in fragments, medium dense.		PS
45	...silty, little gravels, dense		Blowcount = 51/12 in.
50	...coarse-grained, some fine-gravels , medium dense. ...seepage at 51.5 ft		PS
55	...fine-grained, little silt, medium dense.		PS
60	...silty, fine- to medium-grained, dense.		
65	End of Boring at 61.5 feet. Boring dry at completion of drilling.. Backfilled with soil cuttings.		
70			



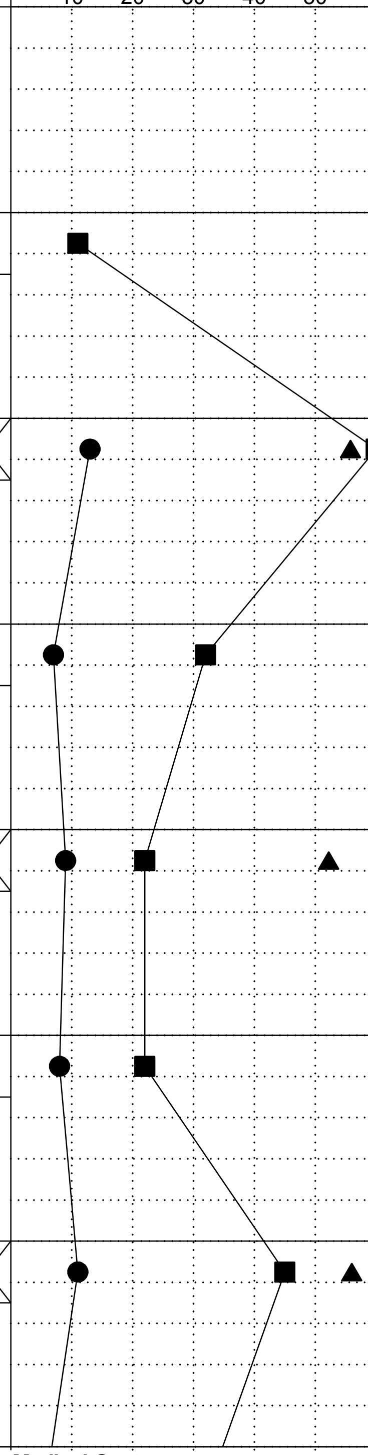
GEOBASE, INC.	PROJECT: KP Moreno Valley Medical Center, 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-10
	DEPTH TO WATER feet	SURFACE ELEV. 1517 feet / LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT / DRILLER Martini Drilling / DATE LOGGED 06/09/2017	FIGURE NO. B- 11

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 / Water Content (%) / Plastic Limit (W_P) Liquid Limit (W_L) / Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
0	SAND (FILL), brown, silty, medium dense	SM			
5	SILT, brown, little fine-grained sands, stiff.	ML	SPT		
10	...hard		California		Blowcount = 50/6 in.
15	SAND, brown, silt interlayer, coarse-grained, some gravels, dense.	SM	SPT		200 Wash
20	...fine-grained sands, medium dense		California		Blowcount = 33/12 in. 200 Wash
25	...silty, medium dense.		SPT		
30	...silty, dense		California		Blowcount = 69/12 in.200 Wash
35					



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA			BORING NO. B-11
	DEPTH TO WATER feet	SURFACE ELEV. 1517 feet	LOGGED BY HDN		PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/09/2017		FIGURE NO. B- 12

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

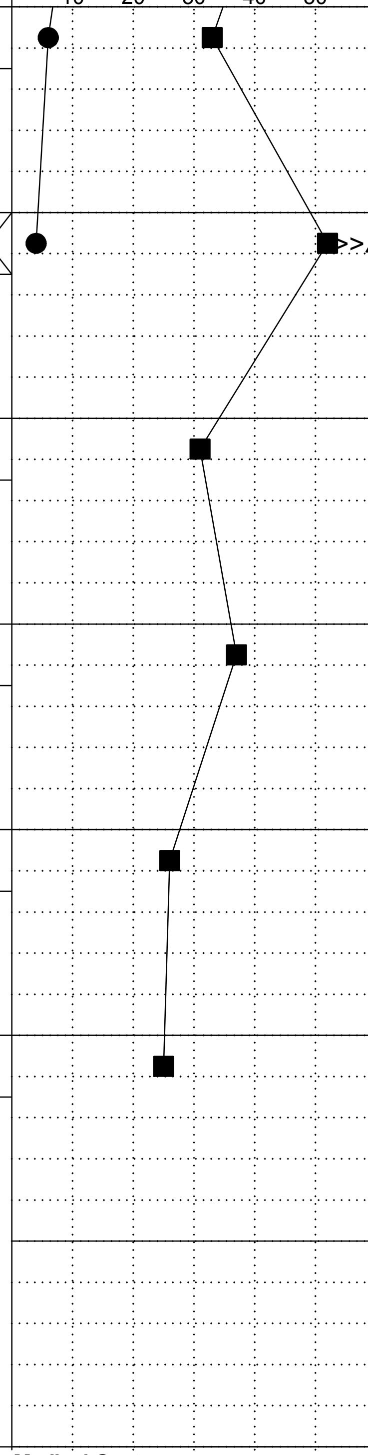
DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	SAND, brown, fine- to medium-grained, trace of silt, dense.	SM	200 Wash
40	SAND, light brown, coarse-grained, little silt, some fine-gravels, very dense.	SP	Blowcount = 80/12 in. PS
45	...fine-to medium-grained, silty, dense		
50	SAND, brown, silty, fine-to medium grained, trace of gravel, dense.	SM	PS
55	...medium dense ...seepage at 56 ft		
60	...trace of gravels, medium dense		
65	End of Boring at 61.5 feet. Boring dry at completion of drilling. Backfilled with soil cuttings.		
70			

DRY DENSITY (PCF) 80 90 100 110 120

Water Content (%):

Plastic Limit (W_P) Liquid Limit (W_L)

Penetration, blows/foot: 10 20 30 40 50



GEOBASE, INC.	PROJECT	KP Moreno Valley Medical Center 27300 Iris Avenue, Moreno Valley, CA		BORING NO. B-11
	DEPTH TO WATER feet	SURFACE ELEV. 1517 feet	LOGGED BY HDN	PROJECT NO. C.314.81.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER Martini Drilling	DATE LOGGED 06/09/2017	FIGURE NO. B- 12

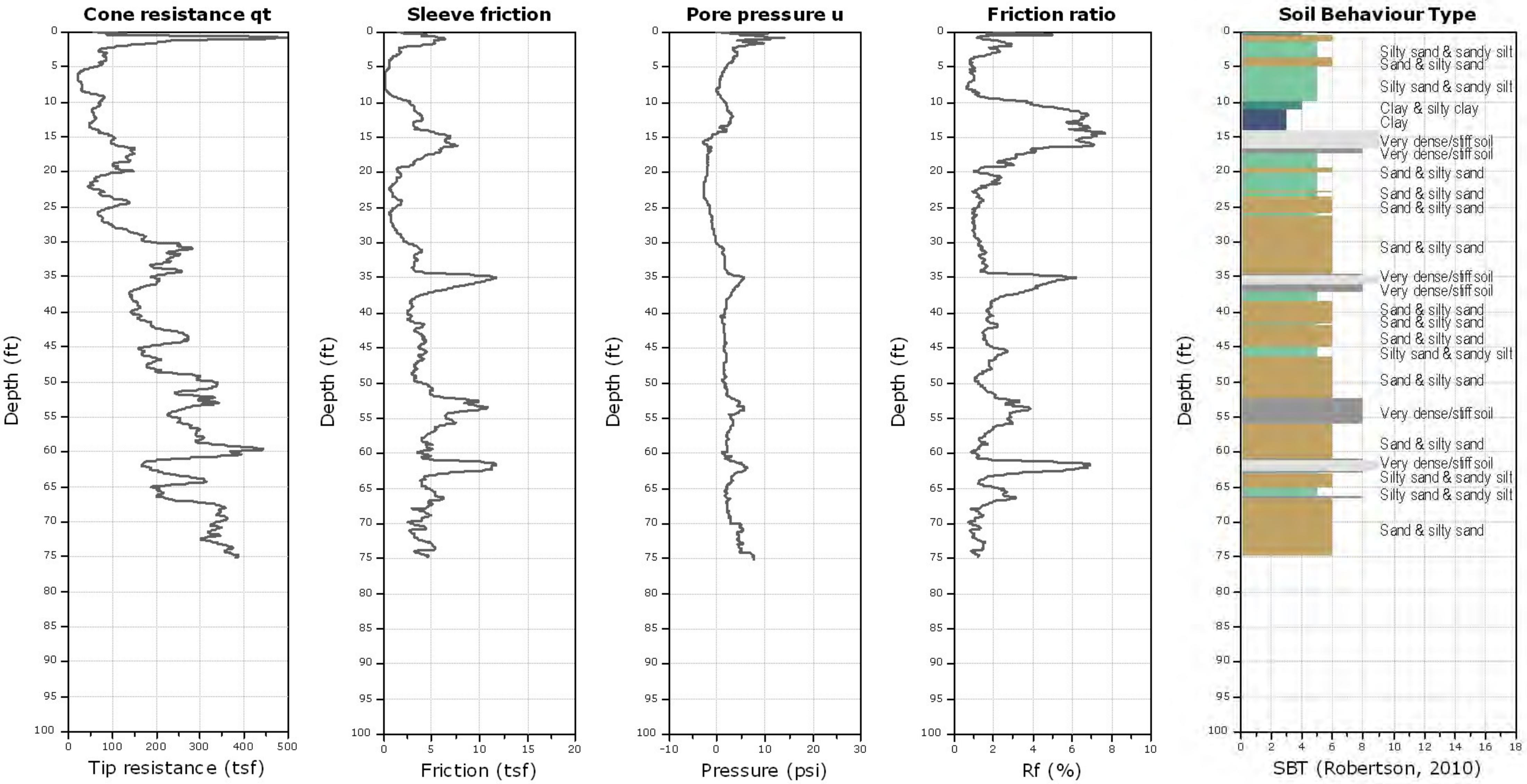
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-1

Project: **GEOBASE, Inc.**
Location: **27300 Iris Ave Moreno Valley, CA**

Total depth: 75.14 ft, Date: 6/8/2017
Cone Type: Vertek



CPeT-IT v.2.0.1.55 - CPTU data presentation & interpretation software - Report created on: 6/12/2017, 3:32:59 PM
Project file: C:\GeobaseMorenoValley6-17\Plot Data\Plots.cpt

C.314.81.00

Figure B-13

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714-901-7270
rich@kehoetesting.com
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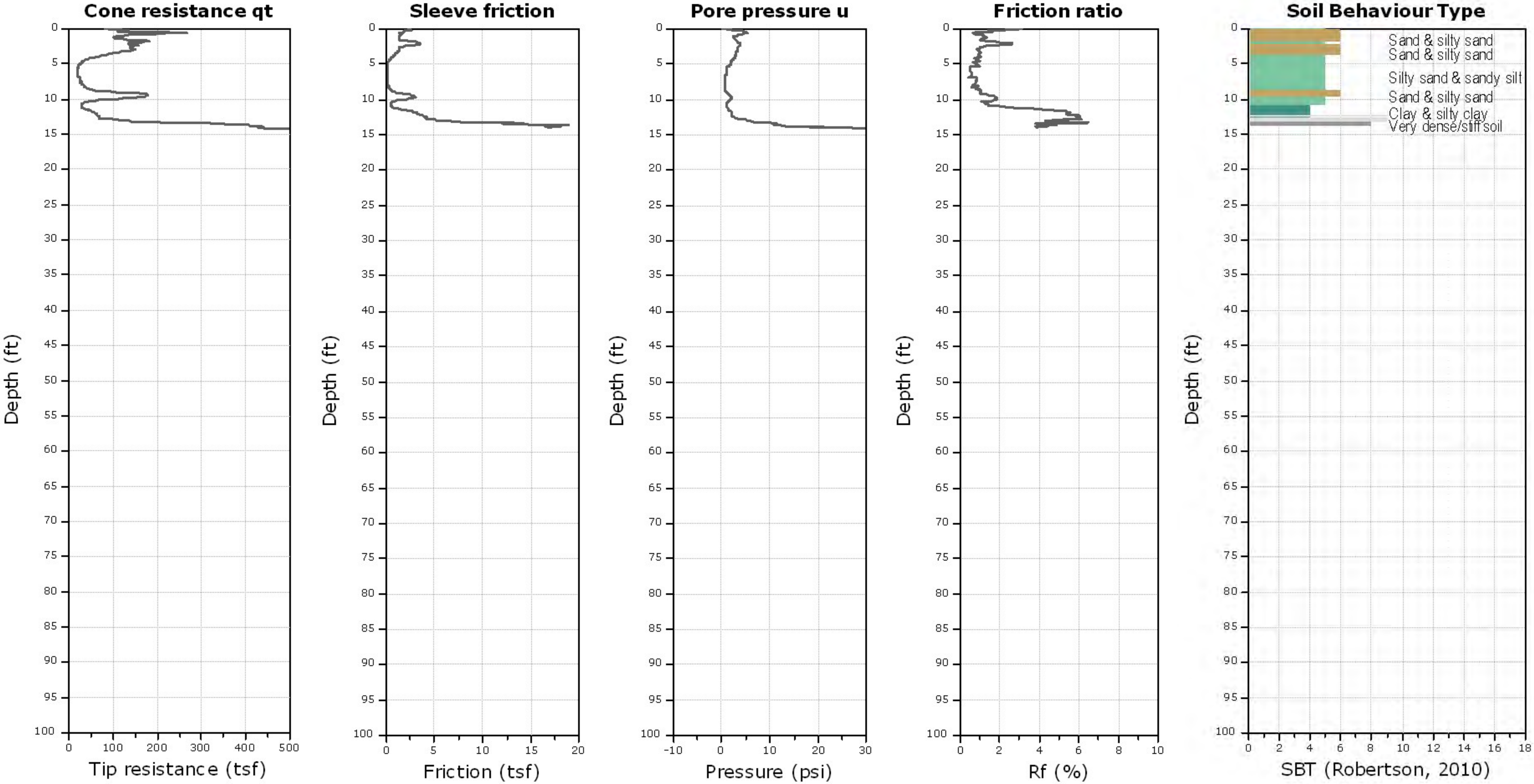
CPT-2

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 14.30 ft, Date: 6/8/2017
Cone Type: Vertek



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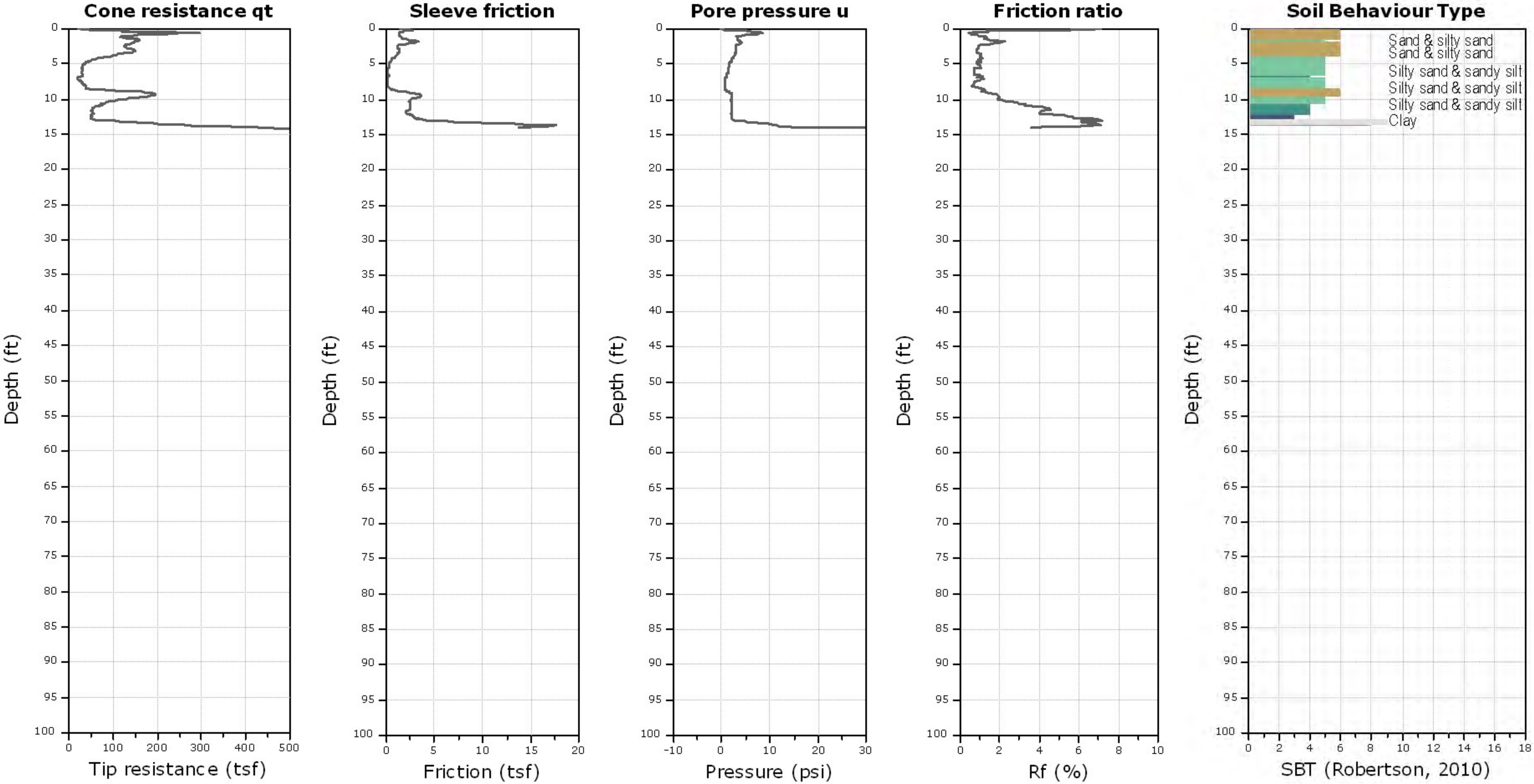
rich@kehoetesting.com
www.kehoetesting.com

CPT-2A

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 14.33 ft, Date: 6/8/2017
Cone Type: Vertek



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rich@kehoetesting.com
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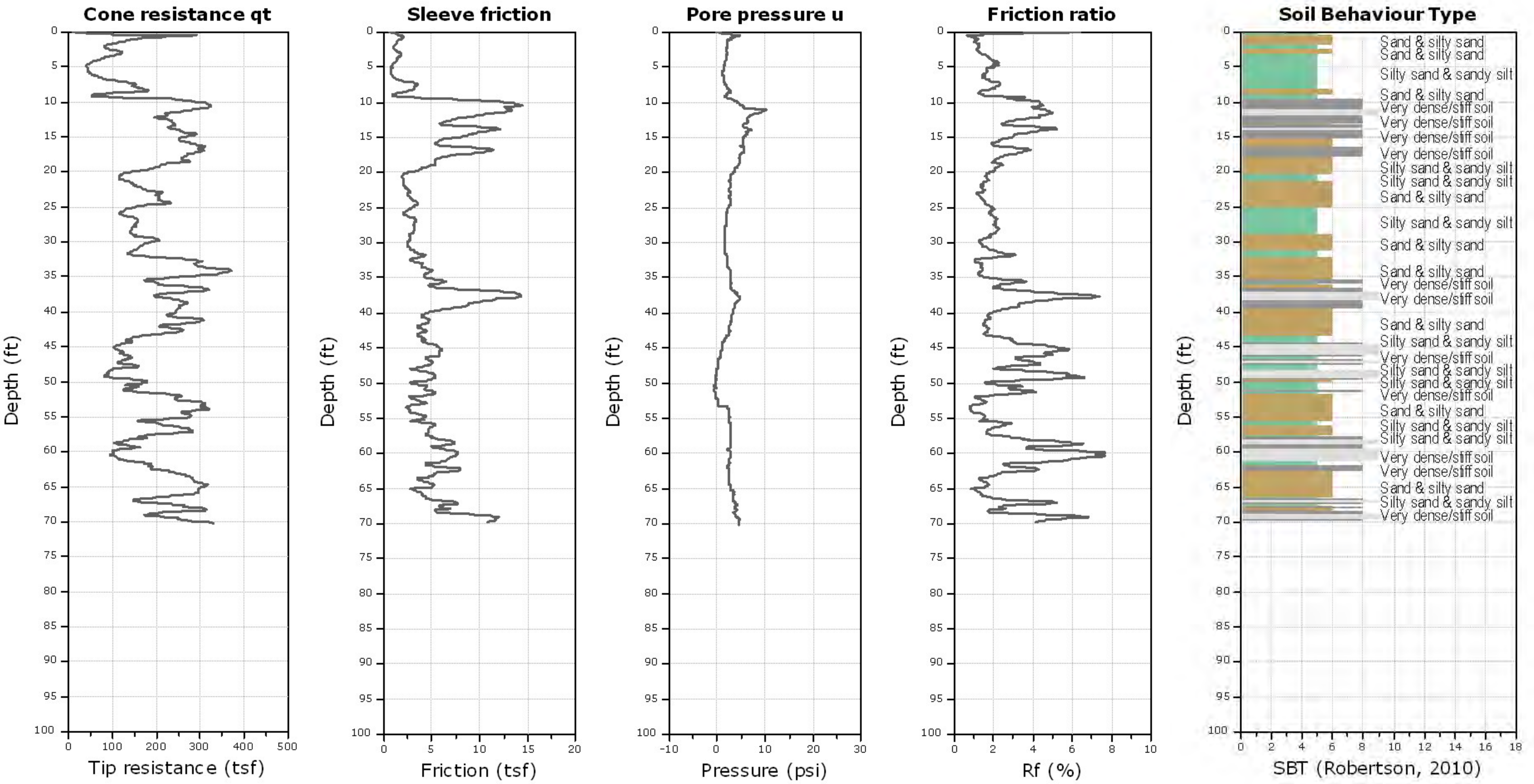
CPT-3

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 70.16 ft, Date: 6/8/2017
Cone Type: Vertek



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714-901-7270
rich@kehoetesting.com
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SCPT-4

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 100.15 ft, Date: 6/8/2017
Cone Type: Vertek

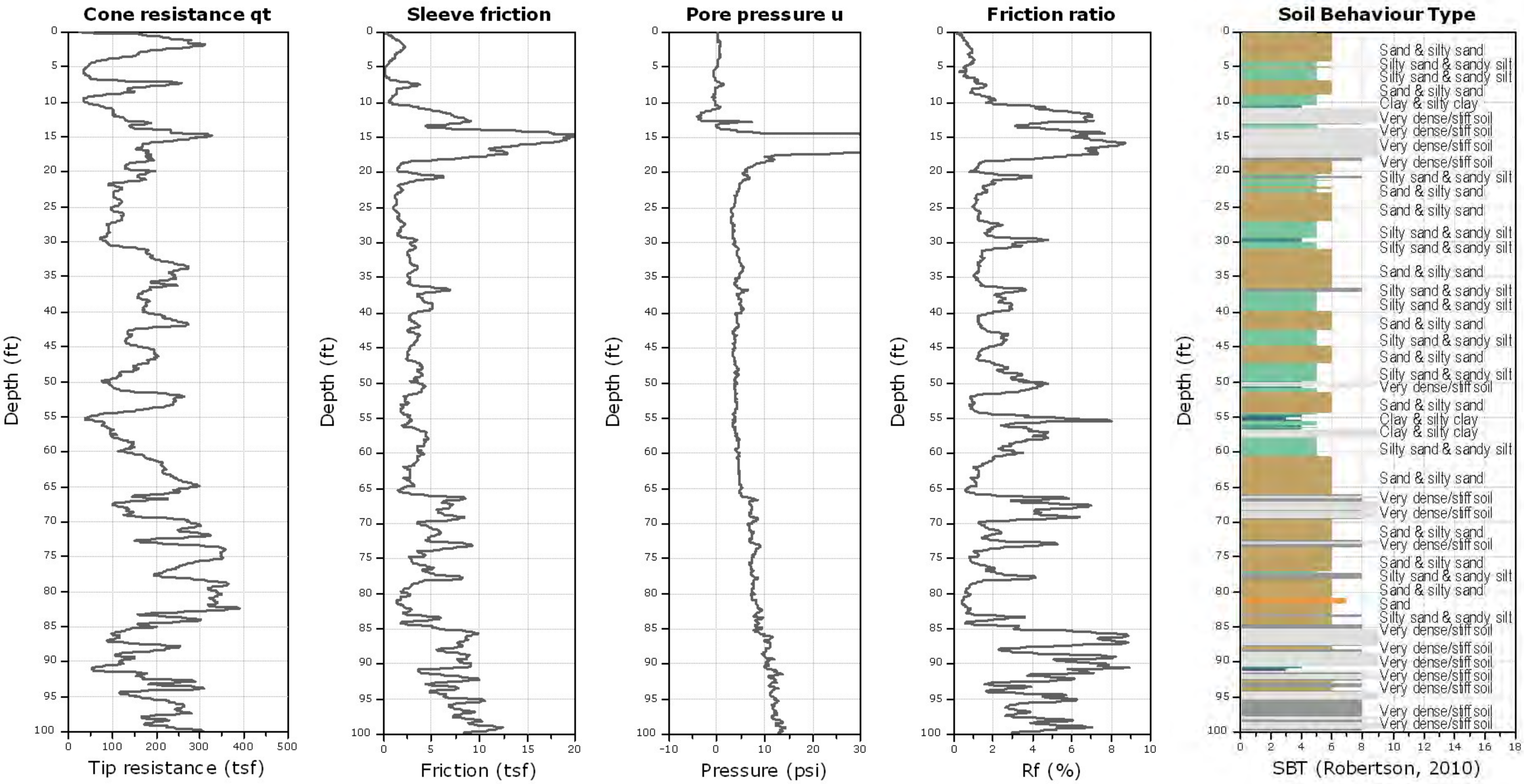

Figure B-16

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rich@kehoetesting.com
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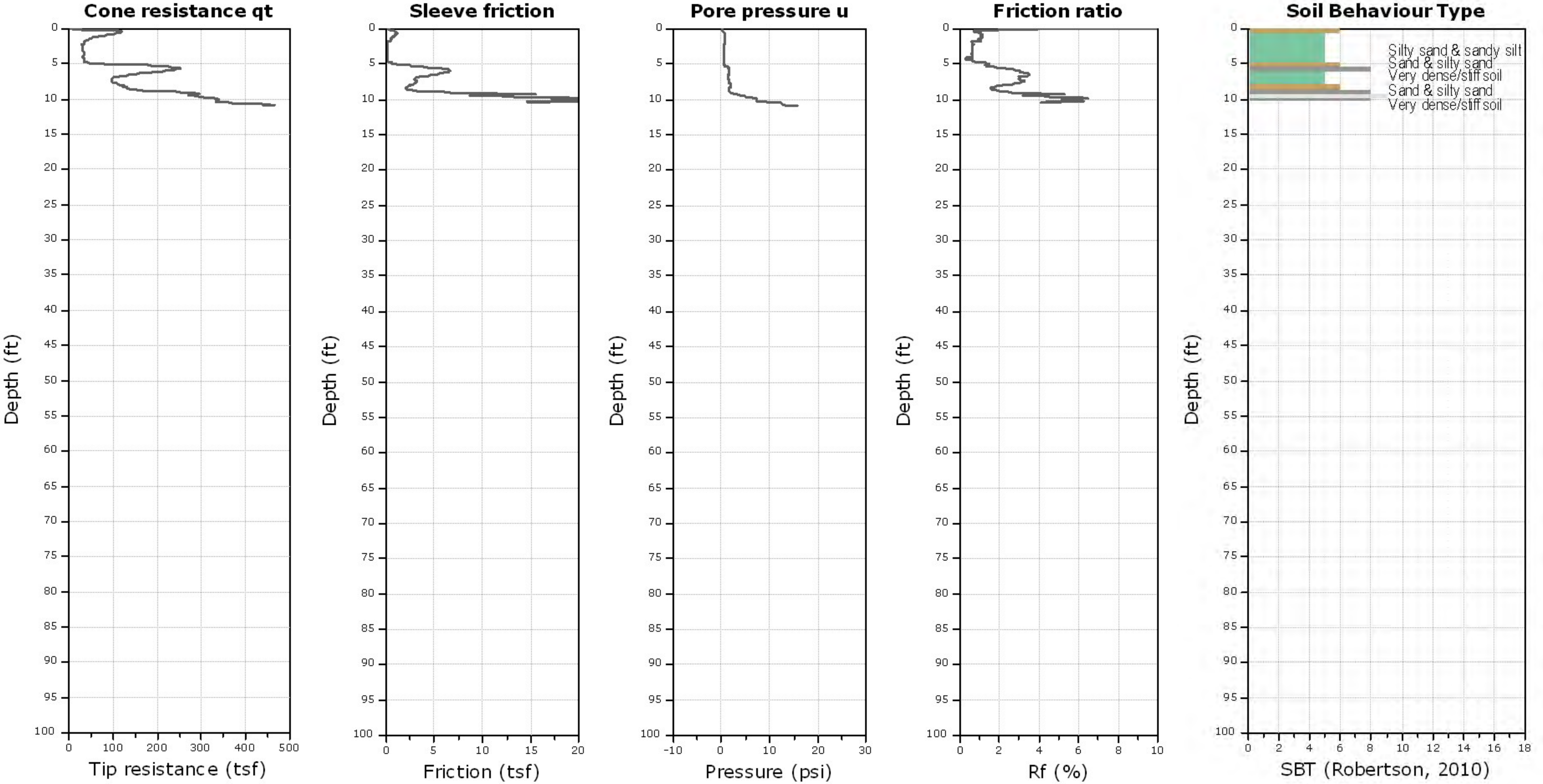
Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

CPT-5
Total depth: 10.90 ft, Date: 6/8/2017
Cone Type: Vertek



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rich@kehoetesting.com
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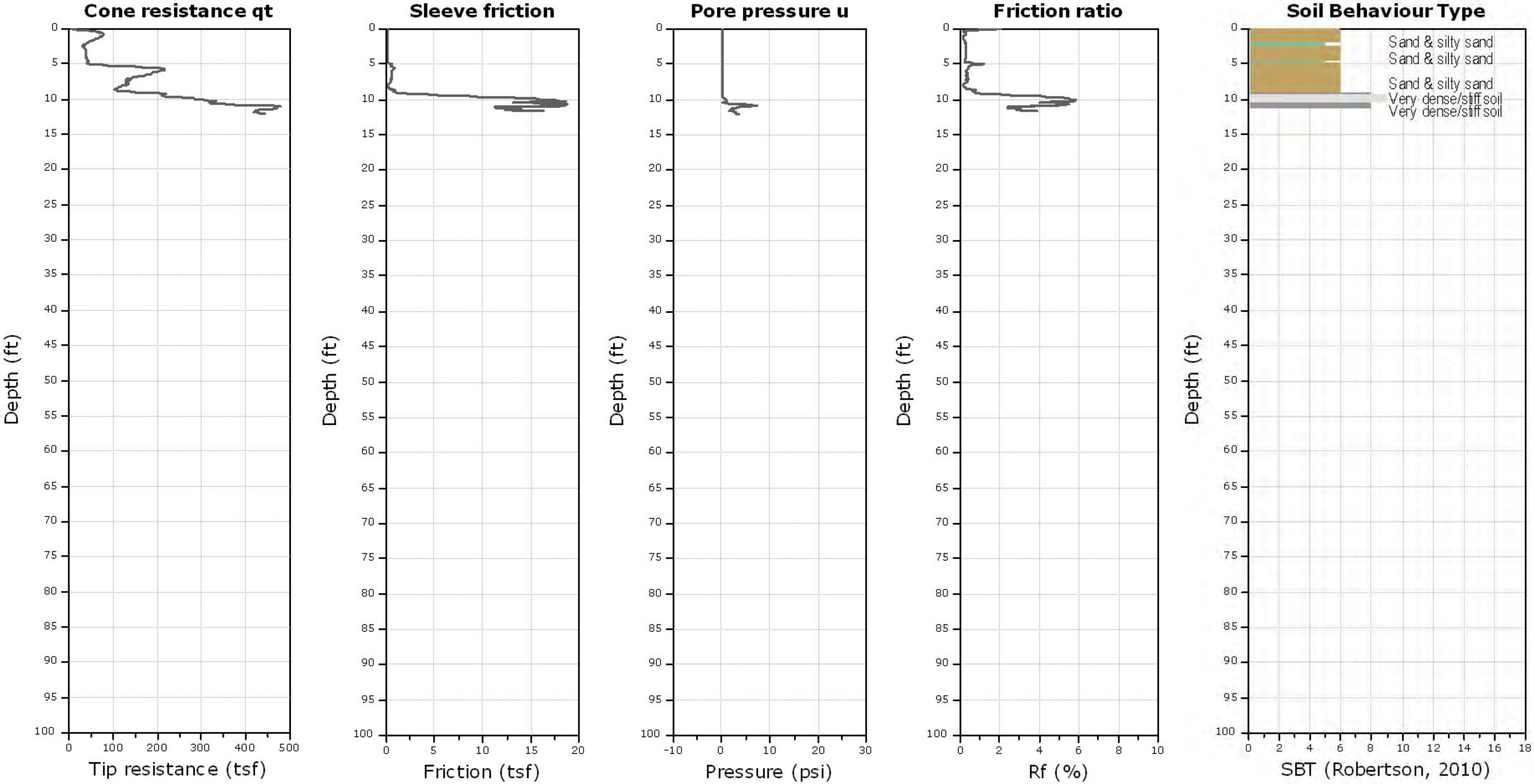
CPT-5A

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 12.01 ft, Date: 6/8/2017
Cone Type: Vertek



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
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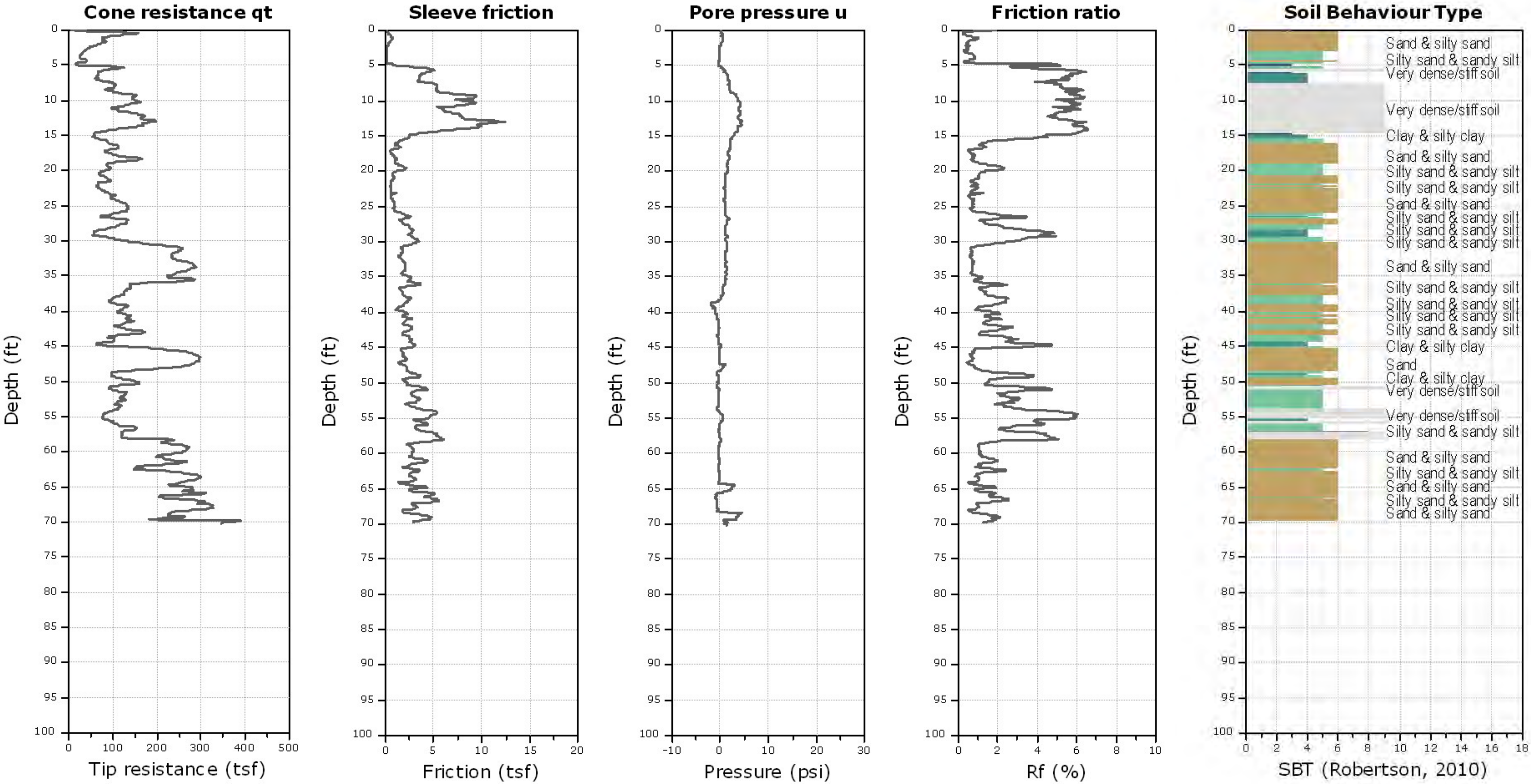
CPT-6

Project: **GEOBASE, Inc.**
Location: **27300 Iris Ave Moreno Valley, CA**

Total depth: 70.15 ft, Date: 6/8/2017
Cone Type: Vertek

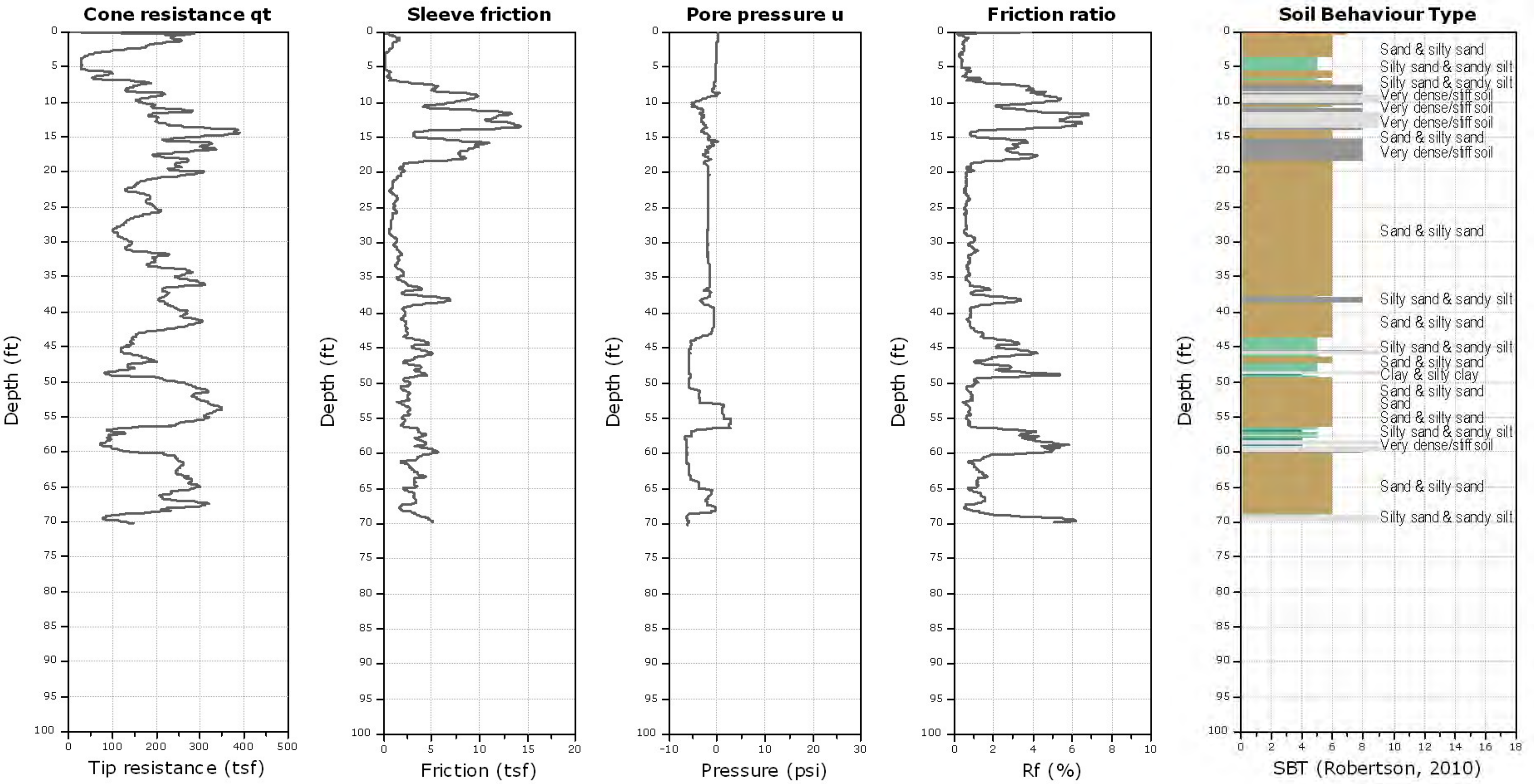


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-7

Project: **GEOBASE, Inc.**
Location: **27300 Iris Ave Moreno Valley, CA**

Total depth: 70.22 ft, Date: 6/8/2017
Cone Type: Vertek



Kehoe Testing and Engineering
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rich@kehoetesting.com
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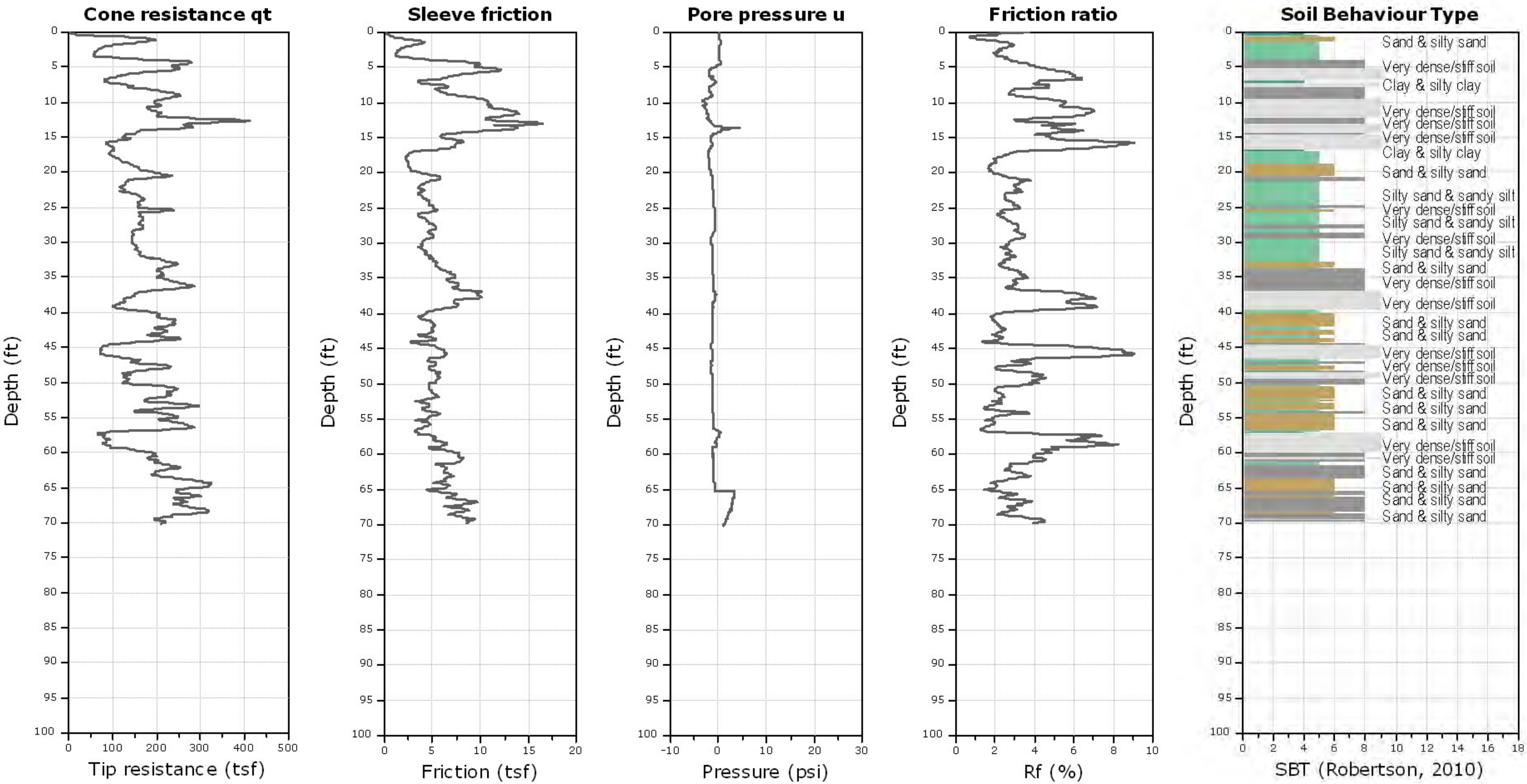
CPT-8

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 70.15 ft, Date: 6/9/2017
Cone Type: Vertek



CPeT-IT v.2.0.1.55 - CPTU data presentation & interpretation software - Report created on: 6/12/2017, 3:36:02 PM
Project file: C:\GeobaseMorenoValley6-17\Plot Data\Plots.cpt

C.314.81.00

Figure B-20

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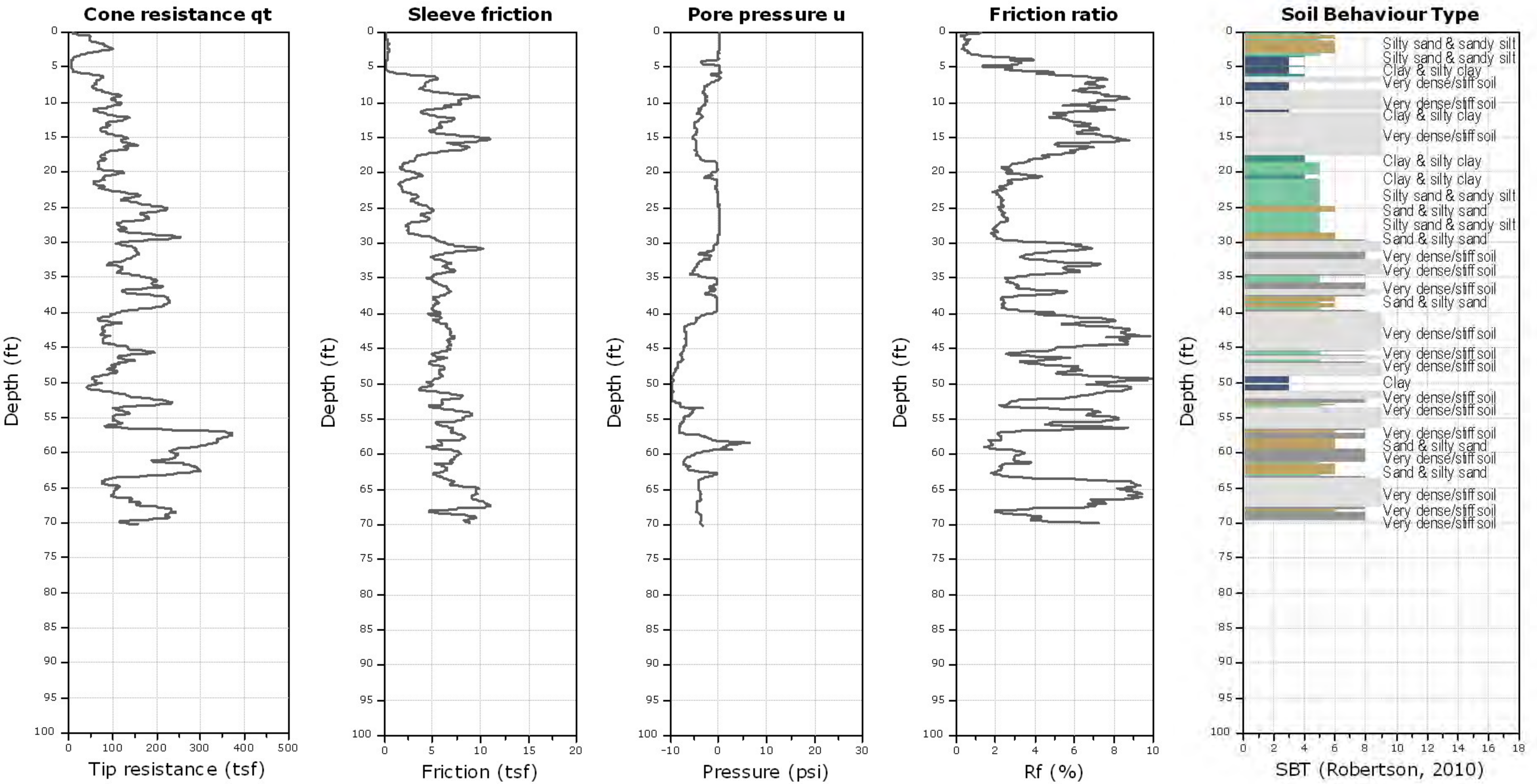
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-9

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 70.24 ft, Date: 6/9/2017
Cone Type: Vertek

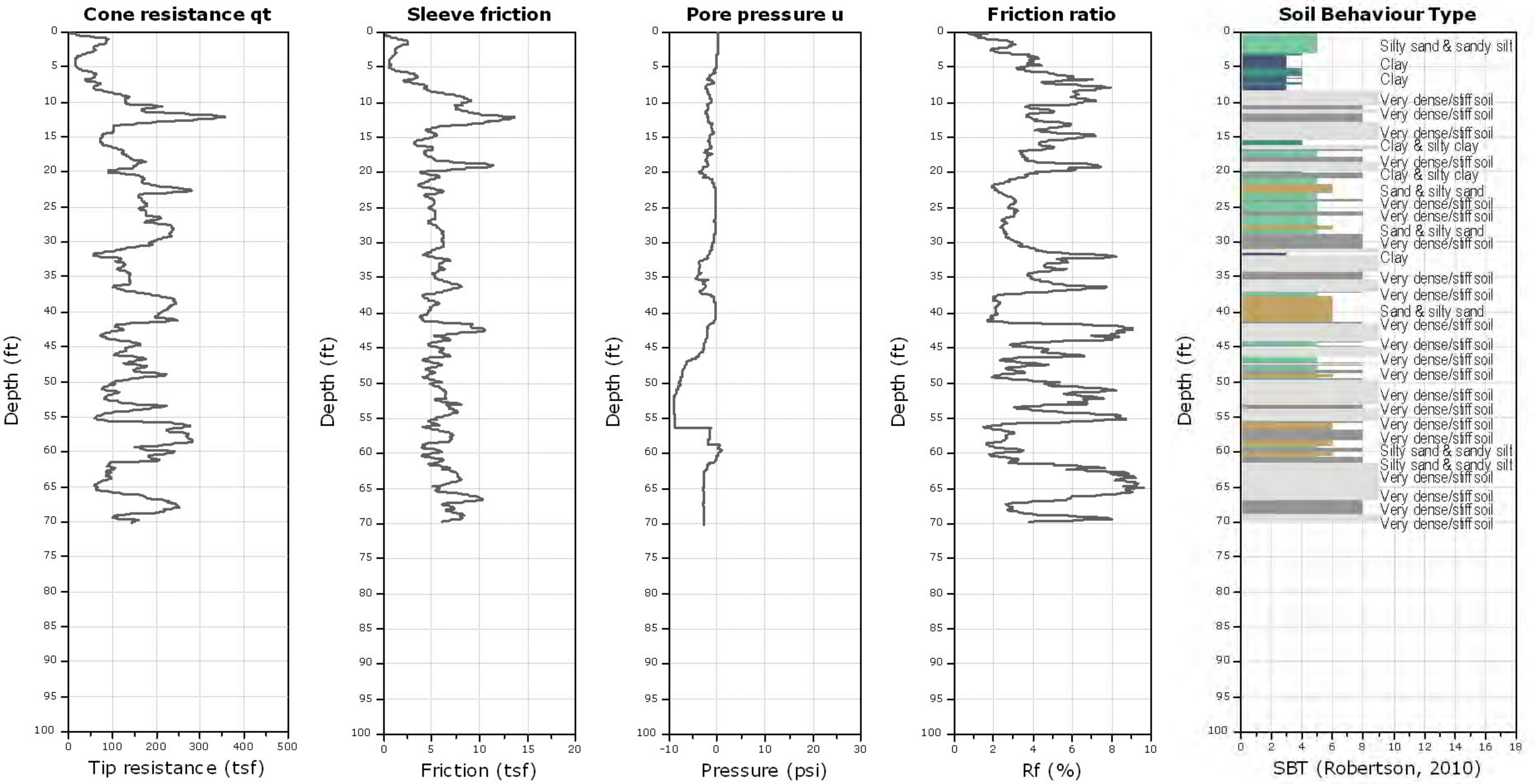


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-10

Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 70.15 ft, Date: 6/9/2017
Cone Type: Vertek



Kehoe Testing and Engineering
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rich@kehoetesting.com
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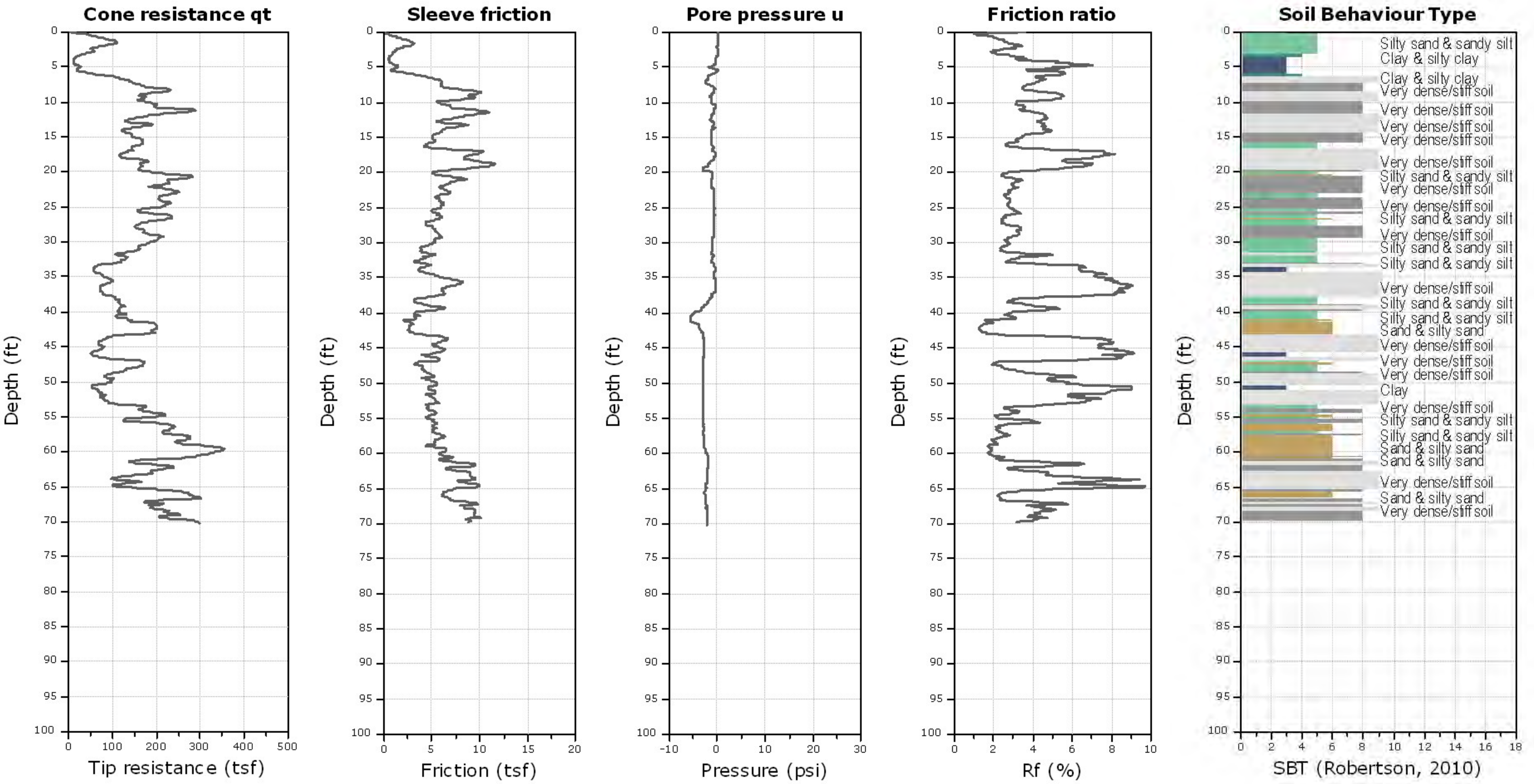
CPT-11

Total depth: 70.22 ft, Date: 6/9/2017

Cone Type: Vertek

Project: GEOBASE, Inc.

Location: 27300 Iris Ave Moreno Valley, CA



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rich@kehoetesting.com
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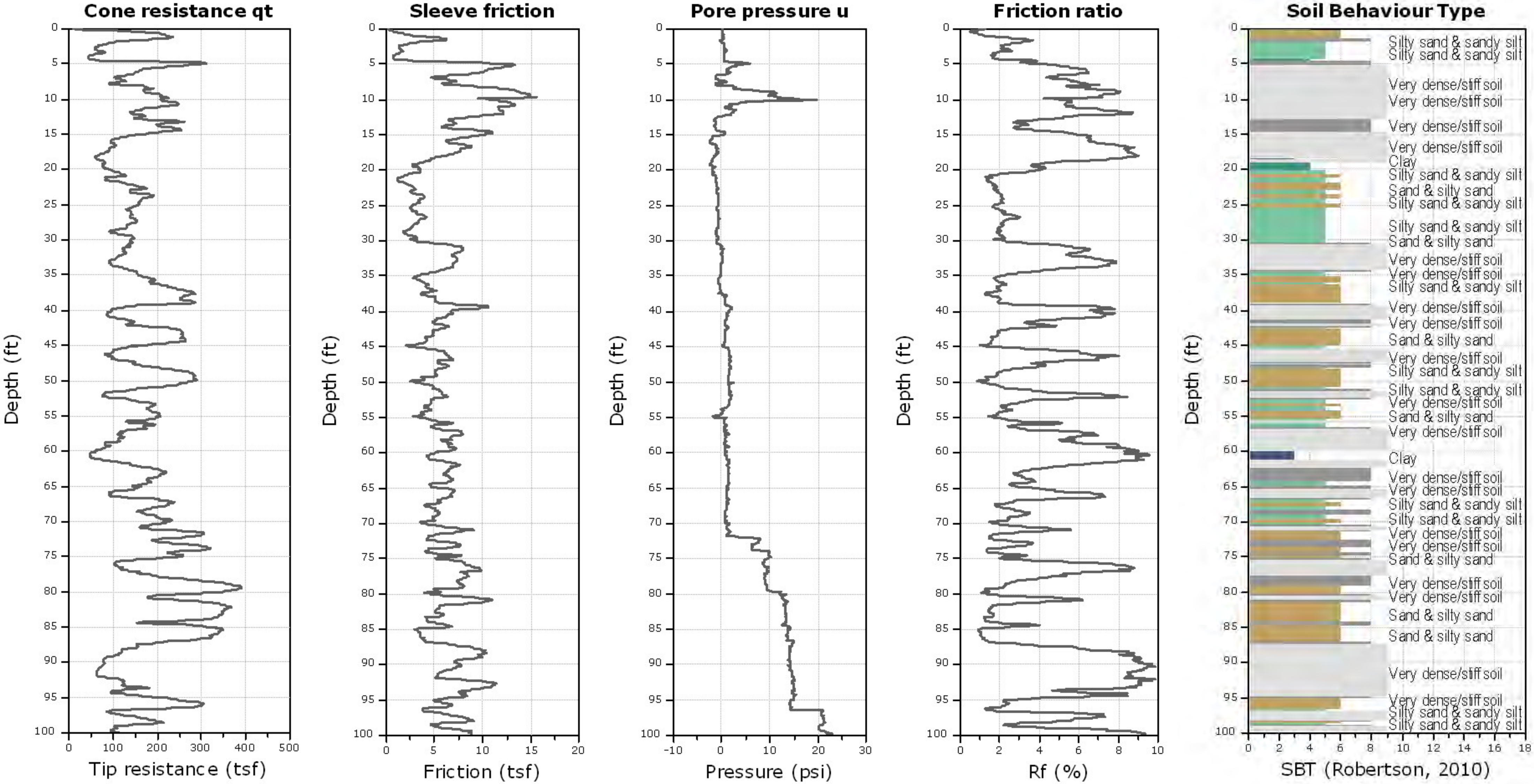
SCPT-12

Project: GEOBASE, Inc.

Location: 27300 Iris Ave Moreno Valley, CA

Total depth: 100.13 ft, Date: 6/9/2017
Cone Type: Vertek



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rich@kehoetesting.com
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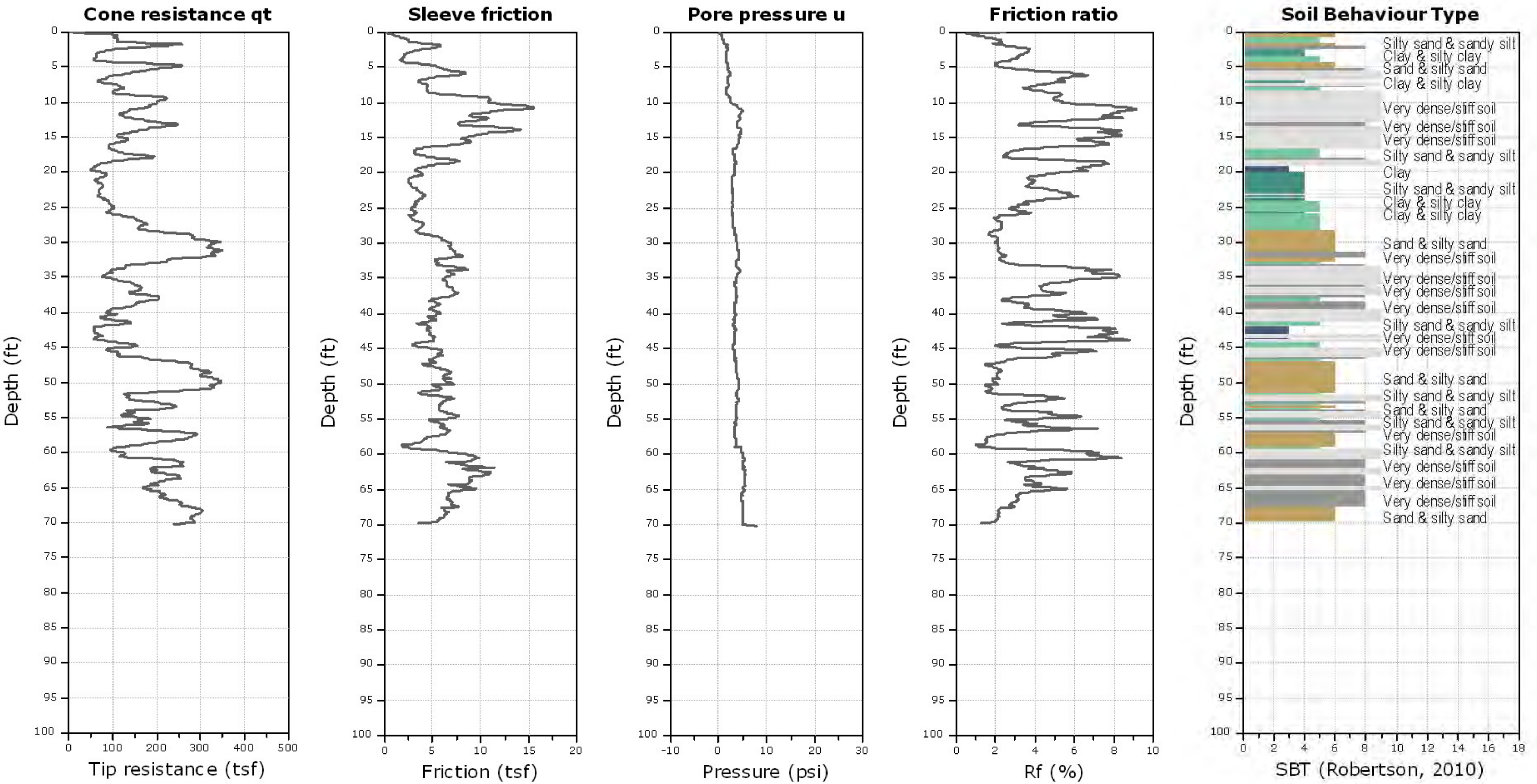
Project: GEOBASE, Inc.
Location: 27300 Iris Ave Moreno Valley, CA

CPT-13
Total depth: 70.22 ft, Date: 6/9/2017
Cone Type: Vertek

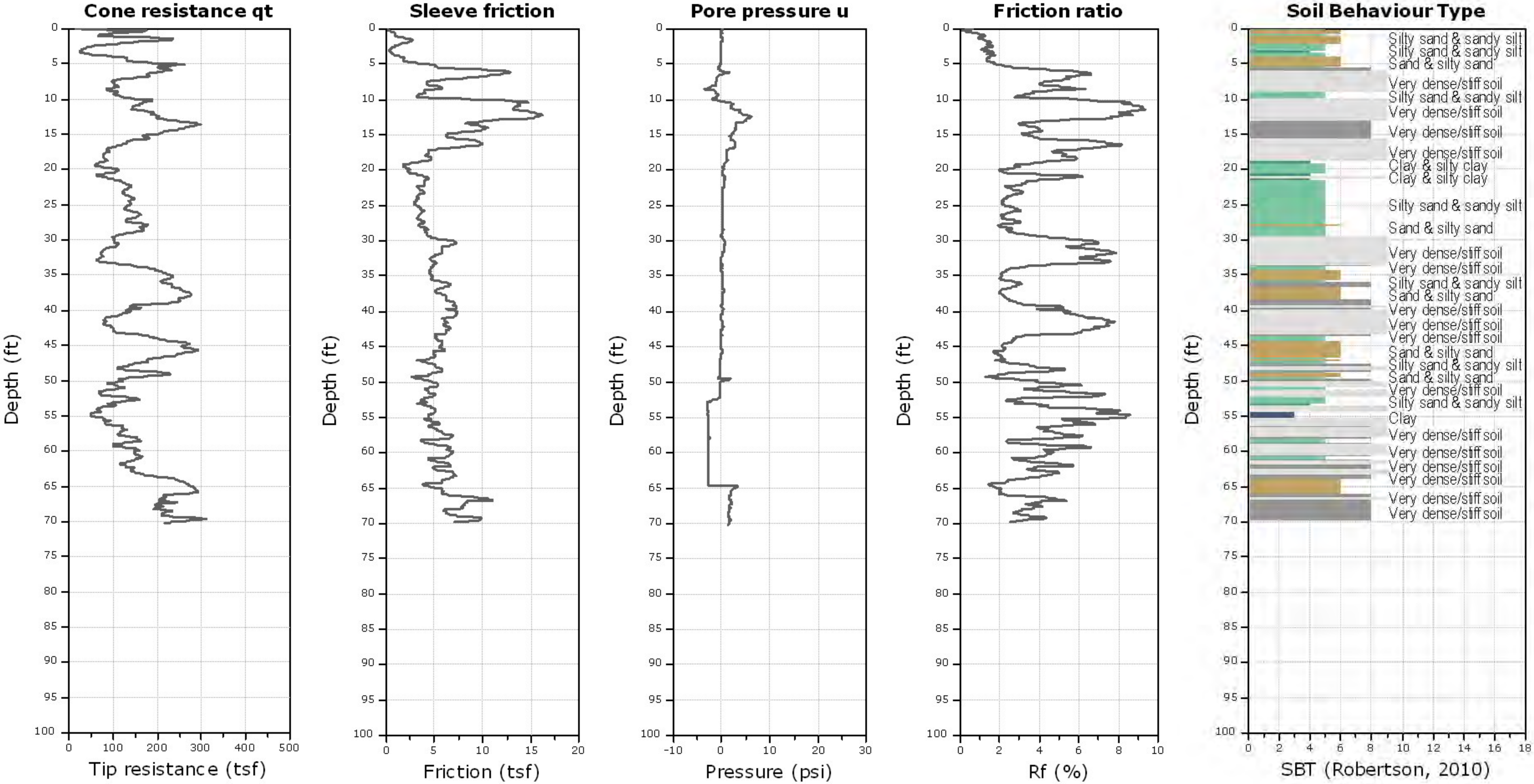


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-14

Project: **GEOBASE, Inc.**
Location: **27300 Iris Ave Moreno Valley, CA**

Total depth: 70.21 ft, Date: 6/9/2017
Cone Type: Vertek

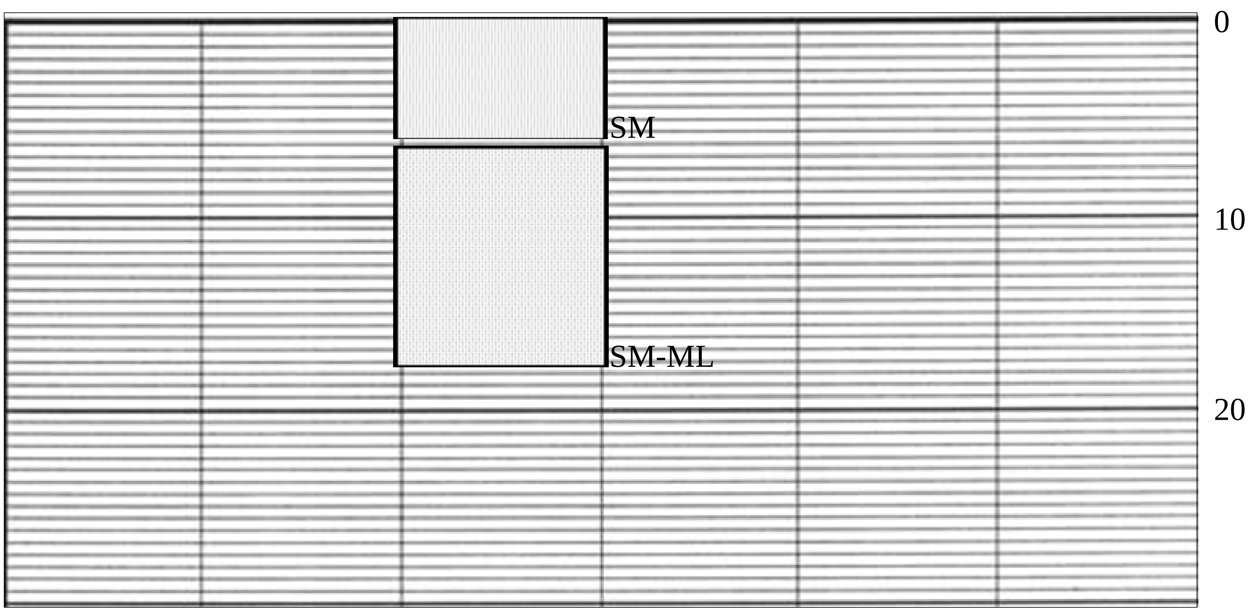


LOG OF TEST PIT: TP - 1

Soil Interval	Soil Interval Depth (Feet bgs)	Soil Sample Depth (Feet bgs)	SOIL DESCRIPTION
A	0.0 – 1.0		FILL- Aggregate Bases
B	1.0 - 5.0		SAND (SM), light brown, fine- to medium grained, little to some silt, trace of gravels, moist, loose to medium dense.
C	5.0-14.0		SAND TO SILT (SM-ML), brown, fine-grained, white streak and concretion, cementation, medium dense to very stiff.
D	14.0-18.0		SAND TO SILT (SM-ML), brown, very distinct white streak, concretion, cementation, stratified, very dense. Difficult to excavate.

GRAPHIC REPRESENTATION

SCALE: 1 inch = 10 Feet **BEARING**: N **WALL**: FRONT (North)



Project Number: **C.314.81.00**

Date: 5/7/2015

GEOBASE INC.

Location: Figures A-2 & A-3, Appendix A

Equipment: JD410

Project: KP Moreno Valley Medical Center

Approx. Elevation: 1525 feet AMSL (Top)

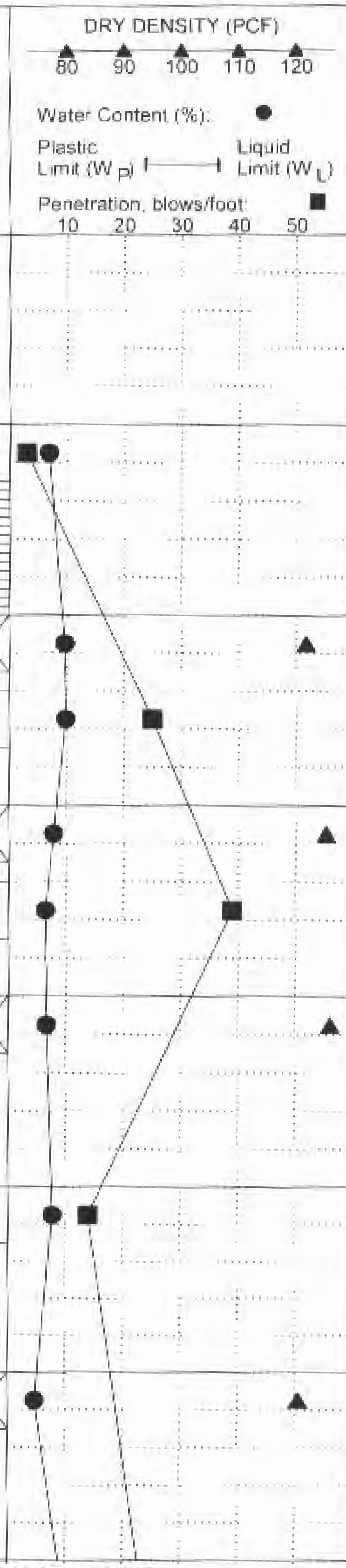
Logged By: HDN

FIGURE B-27

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
0		PAVEMENT, AC = 5 in. and AB = 5 in.			
		SAND (FILL), brown, silty, moist	SM		
5		SILT, brown, sandy, trace of gravels, very loose, damp.	ML	SPT	PS
				Disturbed	Bulk Sample 5-10 FT. EI Ch, ER, pH, SO_4
10		...dense, moist		California	C*, DS
		SAND, brown, silty, trace of gravels, medium dense, moist.	SM	SPT	
15		...orange to brown, damp		California	PS
		...dense		SPT	
20		...sandy, trace of gravels		California	PS
25		...orange to brown, silty, medium dense		SPT	
30		...some gravels, damp.		California	
35					



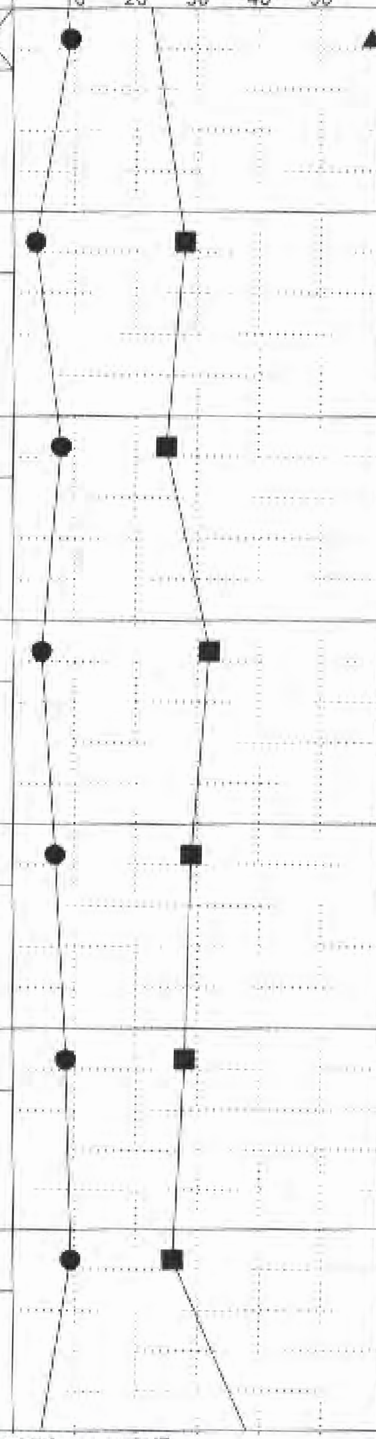
GEOBASE, INC.	PROJECT	MVCH – Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California		BORING NO. B-1
	DEPTH TO WATER feet	SURFACE ELEV. 1524.5 feet	LOGGED BY HDN	PROJECT NO. C.314.39.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/30/2010	FIGURE NO. B-2

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%) ●; Plastic Limit (W_P) — Liquid Limit (W_L); Penetration, blows/foot ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, brown, silty, little gravels, moist.	SM			
40		SAND, brown, white streak, trace of silt, medium dense, damp	SP			
45		SAND, tan brown, silty, little gravels, medium dense, damp.	SM			
50		SAND, brown, white streak, trace of silt, dense, moist	SP			
55		SAND, brown, some silt, medium dense, damp	SM			
60		...silty, greyish brown, slightly moist				
65		...brown, little silt, moist				
70						



GEOBASE, INC.	PROJECT	MVCH -- Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California		BORING NO. B-1
	DEPTH TO WATER feet	SURFACE ELEV 1524.5 feet	LOGGED BY HDN	PROJECT NO. C.314.39.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/30/2010	FIGURE NO. B-2

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 / Water Content (%) / Plastic Limit (W_P) – Liquid Limit (W_L) / Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, gray to brown, little of silt, dense, moist.	SM			
75		* End of Boring at 71.5 feet. * Boring dry at completion of drilling.				
80						
85						
90						
95						
100						
105						

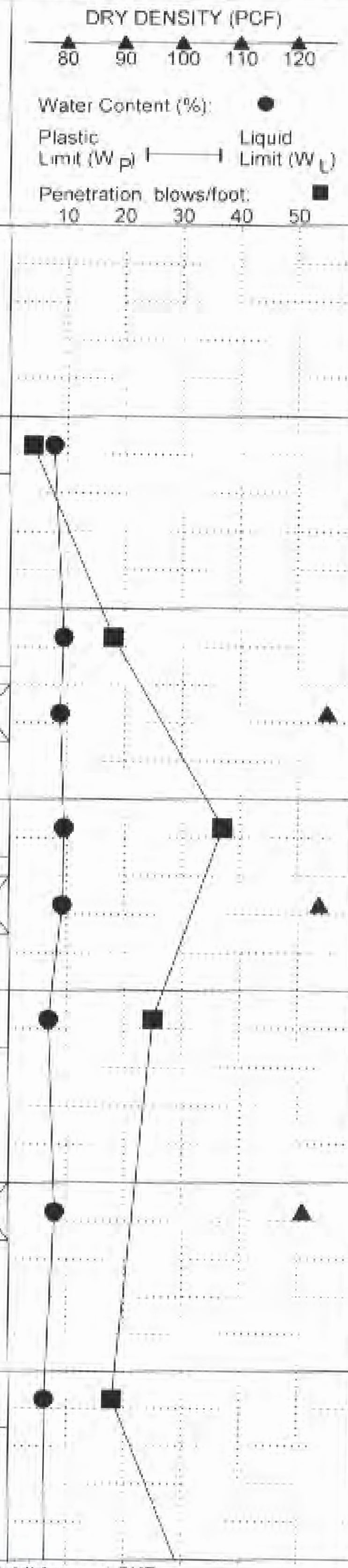
GEOBASE, INC.	PROJECT	MVCH -- Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California		BORING NO. B-1
	DEPTH TO WATER feet	SURFACE ELEV. 1524.5 feet	LOGGED BY HDN	PROJECT NO C.314.39.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/30/2010	FIGURE NO. B-2

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
0	PAVEMENT, AC = 4.5 in. and AB = 5 in		
	SAND (FILL), brown, silty, moist.	SM	
5	SILT, brown, sandy, trace of gravels, very soft, moist.	ML	200 Wash
10	..very stiff		PS
	...hard		PS, C*
15	SAND, olive to brown, white streak, silty, trace of gravels, dense	SM	PS
	brown, some silt, slightly moist.		PS
20	...orange brown, silty, medium dense.		PS
25			PS
30	...light brown, some silt		200 Wash
35			



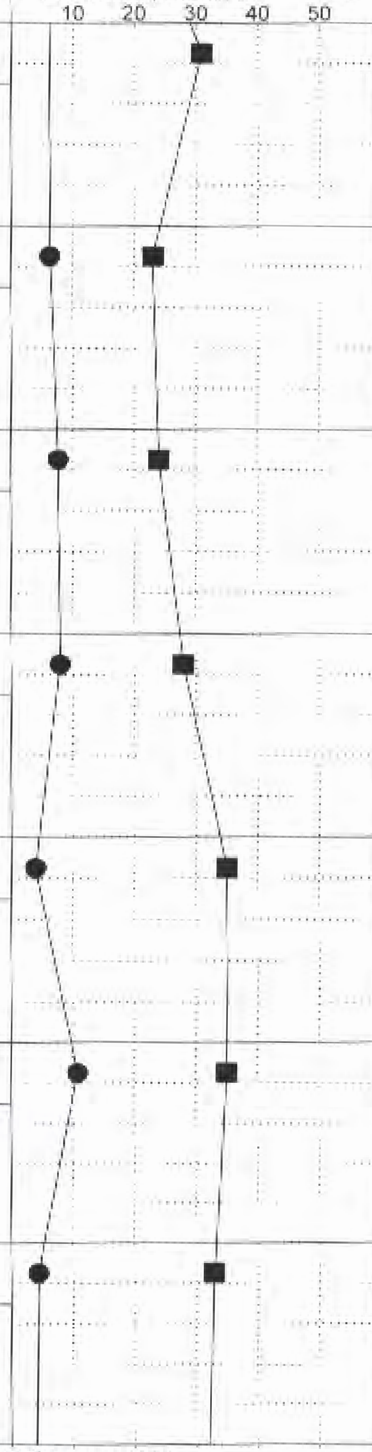
GEOBASE, INC.	PROJECT	MVCH -- Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California		BORING NO. B-2
	DEPTH TO WATER feet	SURFACE ELEV. 1523 feet	LOGGED BY HDN	PROJECT NO. C.314.39.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/31/2010	FIGURE NO. B- 3

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 / Water Content (%) ● / Plastic Limit (W_P) — Liquid Limit (W_L) / Penetration, blows/foot ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, light brown, little silt, dense, damp.	SM			
40		...medium dense				200 Wash
45		.. greyish-brown, silty, medium dense, moist.				200 Wash
50		...light brown				200 Wash
55		...medium-grained				200 Wash
60		.. tan brown, silty				200 Wash
65		...fined grained, little silt.				200 Wash
70						



GEOBASE, INC.				
PROJECT	MVCH -- Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California			BORING NO. B-2
DEPTH TO WATER feet	SURFACE ELEV. 1523 feet	LOGGED BY HDN		PROJECT NO. C.314.39.00
DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/31/2010		FIGURE NO. B-3

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%): ●; Plastic Limit (W_P) — Liquid Limit (W_L); Penetration, blows/foot ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, light brown, trace of silt, medium dense, moist.	SP			200 Wash
75		* End of Boring at 71.5 feet. * Boring dry at completion of drilling.				
80						
85						
90						
95						
100						
105						

GEOBASE, INC.	PROJECT	MVCH -- Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California		BORING NO. B-2
	DEPTH TO WATER feet	SURFACE ELEV. 1523 feet	LOGGED BY HDN	PROJECT NO. C.314.39.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/31/2010	FIGURE NO. B-3

Note. This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

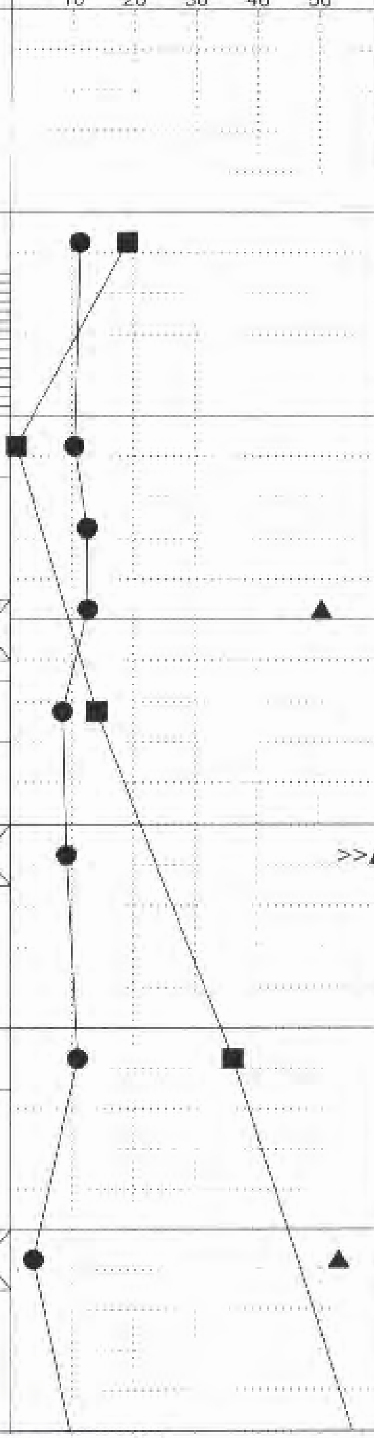
DRY DENSITY (PCF): 80 90 100 110 120

Water Content (%) ●

Plastic Limit (W_P) ├──┤ Liquid Limit (W_L)

Penetration, blows/foot ■ 10 20 30 40 50

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
0	PAVEMENT, AC = 6 in. and AB = 4 in.		
	SAND (FILL), brown, silty, moist.	SM	
5	SAND, light brown, little silt, medium dense	SM	200 Wash
	SILT, reddish brown, sandy, little gravels, very moist.	ML	Bulk Sample 5-10 FT 200 Wash, Ch, ER, pH, SO_4
10	...very loose		200 Wash
15	...little clay, sandy, moist.		200 Wash, C*
	SAND, light brown, some silt, little gravels, medium dense, moist	SM	200 Wash
20	...reddish brown, trace of gravels		>> 200 Wash, C*, DS
25	SILT, brown, sandy, hard, moist	ML	200 Wash
30	SAND, light brown, coarse grained, trace of silt and gravels.	SP	200 Wash
35			



GEOBASE, INC.	PROJECT: MVCH -- Hospital Addition and CUP, 2300 IRIS AVENUE, Moreno Valley, California		BORING NO. B-4
	DEPTH TO WATER feet	SURFACE ELEV. 1540 feet / LOGGED BY HDN	PROJECT NO. C.314.39.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT, DRILLER MARTINI / DATE LOGGED 03/30/2010	FIGURE NO. B-5

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

page 1 of 3

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	SAND, light brown, little silt and gravels, very dense moist.	SM	200 Wash
40	SAND, brown, coarsed grained, trace of silt damp.	SP	200 Wash
45	SAND, light brown, some silt, trace of gravels, medium dense, damp	SM	200 Wash
50			200 Wash
55	...tan, little silt, dense, moist		200 Wash
60	...little silt.		200 Wash
65	...moist.		200 Wash
70			

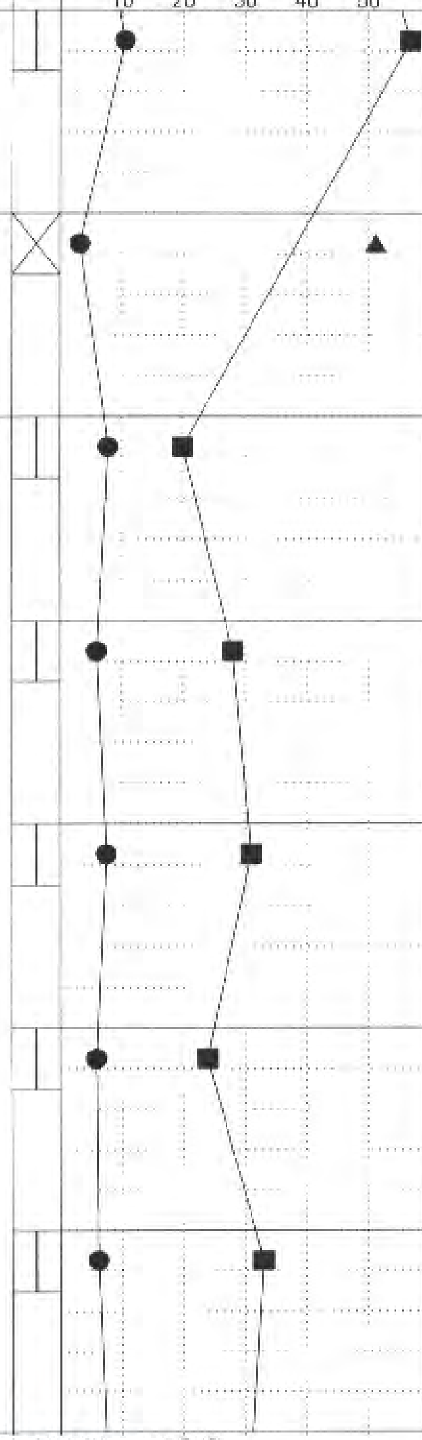
DRY DENSITY (PCF): 80 90 100 110 120

Water Content (%) ●

Plastic Limit (W_P) ├──┤ Liquid Limit (W_L)

Penetration, blows/foot ■

10 20 30 40 50



GEOBASE, INC.	PROJECT	MVCH – Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California		BORING NO. B-4
	DEPTH TO WATER feet	SURFACE ELEV. 1540 feet	LOGGED BY HDN	PROJECT NO C.314.39.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/30/2010	FIGURE NO B-5

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: THIN WALLED TUBE | SPT SPLIT SPOON | CALIFORNIA MODIFIED SAMPLER | DISTURBED | NO RECOVERY | CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120; Water Content (%) ●; Plastic Limit (W_P) ⊢—⊣ Liquid Limit (W_L); Penetration, blows/foot ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		SAND, brown, some silt, trace of gravels, dense, moist.	SM			200 Wash
75		* End of Boring at 71.5 feet. * Boring dry at completion of drilling				
80						
85						
90						
95						
100						
105						

GEOBASE, INC.				
PROJECT	MVCH -- Hospital Addition and CUP 2300 IRIS AVENUE, Moreno Valley, California			BORING NO. B-4
DEPTH TO WATER feet	SURFACE ELEV. 1540 feet	LOGGED BY HDN		PROJECT NO C.314.39.00
DEPTH TO SLOUGH	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 03/30/2010		FIGURE NO B- 5

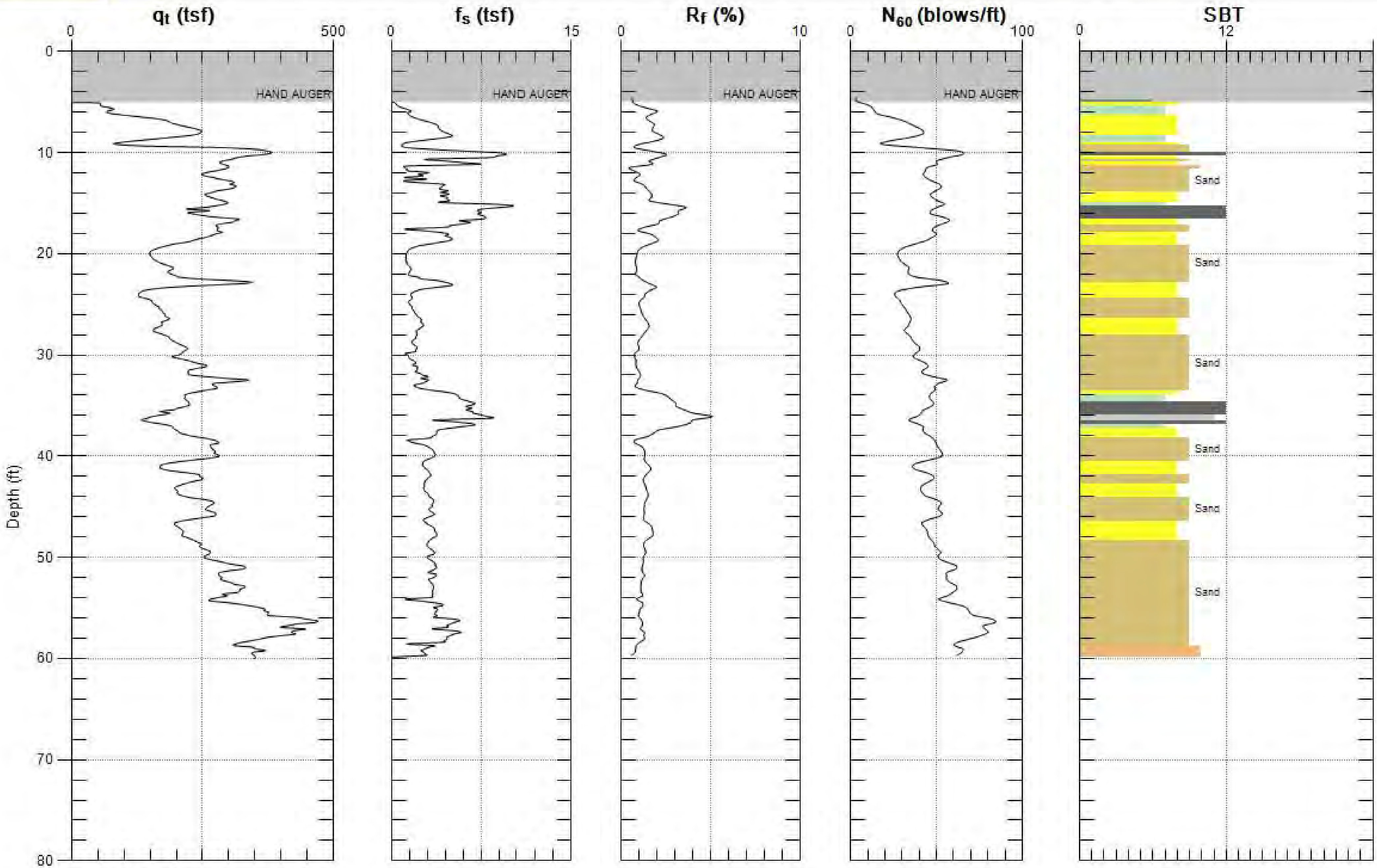
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

page 3 of 3

GEOBASE

Site: KAISER MORENO VALLEY Engineer: H.NGUYEN

Sounding: CPT-3 Date: 2010-03-31 10:36



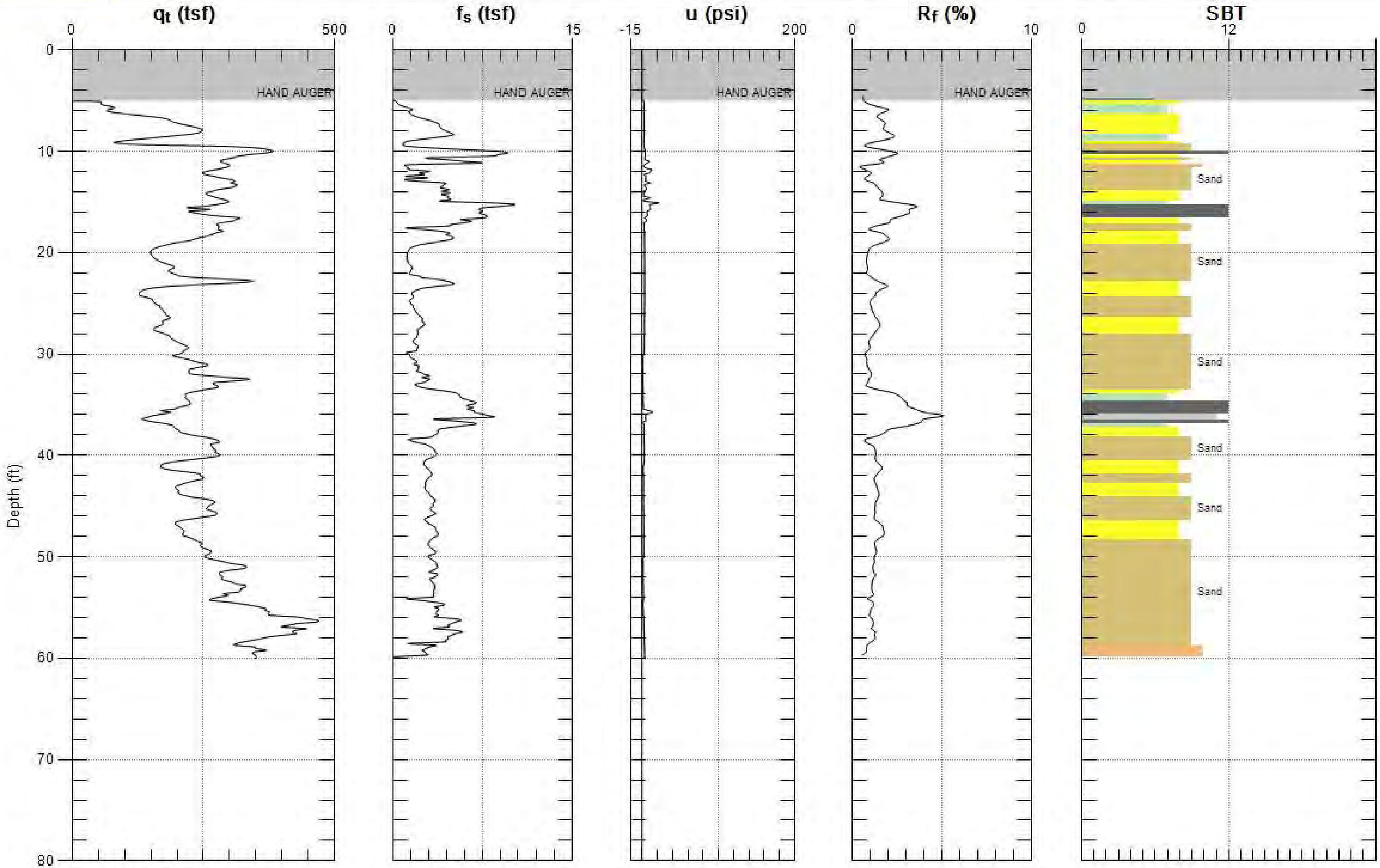
Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **GEOBASE**

Site: KAISER MORENO VALLEY Engineer: H.NGUYEN

Sounding: CPT-3 Date: 2010-03-31 10:36



Max. Depth: 60.039 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

REPORT

SURFACE WAVE MEASUREMENTS

27300 IRIS AVENUE
MORENO VALLEY, CALIFORNIA

GEO*Vision* Project No. 17242

Prepared for

GEOBASE, Inc.
23362 Peralta Dr., Unit 4
Laguna Hills, CA 92653
(949) 588-3744

Prepared by

GEO*Vision* Geophysical Services, Inc.
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

Report 17242-01

July 21, 2017

TABLE OF CONTENTS

1 INTRODUCTION....1

2 OVERVIEW OF THE SURFACE WAVE METHODS....2

3 FIELD PROCEDURES....6

4 DATA REDUCTION AND MODELING....7

5 INTERPRETATION AND RESULTS....10

6 CONCLUSIONS....12

7 REFERENCES....13

8 CERTIFICATION....15

APPENDIX A TECHNICAL NOTE – ACTIVE AND PASSIVE SURFACE WAVE TECHNIQUES
TECHNICAL NOTE – HVSR METHOD

LIST OF TABLES

Table 1 V_S Model – Array 1 (Metric Units)
Table 2 V_S Model – Array 1 (Imperial Units)
Table 3 V_S Model – Array 2 Model 1 (Metric Units)
Table 4 V_S Model – Array 2 Model 1 (Imperial Units)
Table 5 V_S Model – Array 2 Model 2 (Metric Units)
Table 6 V_S Model – Array 2 Model 2 (Imperial Units)

LIST OF FIGURES

Figure 1 Site Map
Figure 2 Observed H/V Spectral Ratio
Figure 3 Surface Wave Model – Array 1
Figure 4 Surface Wave Model – Array 2
Figure 5 Observed & Calculated H/V Spectral Ratio

1 INTRODUCTION

In-situ seismic measurements using active and passive surface wave techniques were performed in a lot north of the Kaiser Permanente hospital located at 27300 Iris Avenue in Moreno Valley, California on July 10^{th}, 2017. The purpose of this investigation was to provide a shear (S) wave velocity profile to a depth of greater than 30 m and estimate the average S-wave velocity of the upper 30 m (V_{S30}). The active surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The passive surface wave technique consisted of the array microtremor method. Because bedrock was expected to be greater than 30 m deep at the site, horizontal over vertical spectral ratio (HVSR) measurement were also made at the site. The locations of the active and passive surface wave arrays and HVSR measurements are shown on Figure 1.

V_{S30} is used in the NEHRP provisions and the Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design (BSSC, 1994). The average shear wave velocity of the upper 100 ft (V_{S100ft}) is used in the International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – V_{S30} > 1500 m/s (UBC) or V_{S100ft} > 5,000 ft/s (IBC)
Class B – rock – 760 < V_{S30} ≤ 1500 m/s (UBC) or 2,500 < V_{S100ft} ≤ 5,000 ft/s (IBC)
Class C – very dense soil and soft rock – 360 < V_{S30} ≤ 760 m/s (UBC)
or 1,200 < V_{S100ft} ≤ 2,500 ft/s (IBC)
Class D – stiff soil – 180 < V_{S30} ≤ 360 m/s (UBC) or 600 < V_{S100ft} ≤ 1,200 ft/s (IBC)
Class E – soft soil – V_{S30} < 180 m/s (UBC) or V_{S100ft} < 600 ft/s (IBC)
Class F – soils requiring site-specific evaluation

At many sites, active surface wave techniques (MASW) with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain a 30 m (100 ft) S-wave velocity sounding. At sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to 30 m and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive surface wave techniques, such as the array microtremor technique or the refraction microtremor method of Louie (2001), can be used to extend the depth of investigation at sites that have adequate ambient noise conditions. It should be noted that two-dimensional passive surface wave arrays (e.g. triangular, circular or L-shaped arrays) will perform better than linear arrays.

This report contains the results of the active and passive surface wave measurements conducted at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Interpretation and results are presented in Section 5 and Section 6 presents our conclusions. References and our professional certification are presented in Sections 7 and 8, respectively.

2 OVERVIEW OF THE SURFACE WAVE METHODS

A discussion of active and passive surface wave methods is provided in the technical note included as Appendix A. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. The Rayleigh wave 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. The Love wave phase velocity, V_L, depends primarily on V_S and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor (Q).

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 (dispersion curve) is the variation of V_R or V_L with λ or f.

The SASW and MASW methods are in-situ seismic method for determining shear wave velocity (V_S) profiles (Stokoe et al., 1994; Stokoe et al., 1989; Park et al., 1999a and 1999b, Foti, 2000). Surface wave techniques are 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%). SASW testing consists of collecting surface wave phase data in the field, generating the dispersion curve, and then using iterative forward or inverse modeling to calculate the shear stiffness profile. MASW testing consists of collecting multi-channel seismic data in the field, applying a wavefield transform to obtain the dispersion curve, and data modeling.

A detailed description of the SASW field procedure is given in Joh, 1996. A vertical dynamic load is used to generate horizontally-propagating Rayleigh waves and a horizontal force is used to generate Love waves. The ground motions are monitored by two, or more, vertical (Rayleigh wave) or horizontal (Love wave) 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, necessitate 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. To develop a V_S model to a 30 meter depth using Rayleigh wave methods, energy sources typically include: small hammers (rock hammer or 3 lb hammer) for short receiver intervals; 10 to 20 lb sledgehammers for intermediate separations, and accelerated weight drops (AWD) or an electromechanical shaker for larger spacings. More energetic sources, such as bulldozers or seismic vibrators (Vibroseis™), can be used to conduct characterize velocity structure to depths of 100 m or more. Energy sources for shallow imaging using Love waves include a hammer and horizontal traction plank, portable hammer impact aluminum source, and inclined or horizontal accelerated weight drop systems. Energy sources for deeper imaging using Love waves include horizontal seismic vibrators. Generally, high frequency (short wavelength) surface waves are recorded across receiver pairs spaced at short intervals, whereas low frequency (long wavelength) surface waves require greater spacing between receivers. Dispersion data averaged across greater distances are often smoother because effects of localized heterogeneities are averaged.

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 wave train 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 two 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:

$$V_{R/L} = f * d_2/(\Delta\phi/360°)$$

where V_R = Rayleigh wave phase velocity
V_L = Love wave phase velocity
f = frequency
d_2 = distance between receivers
$\Delta\phi$ = the phase difference in degrees

A detailed description of the MASW method is given by Park, 1999a and 1999b. Ground motions are recorded by 24 or more geophones spaced 1 to 3 m apart and aligned in a linear array and connected to a seismograph. Energy sources are the same as those outlined above for SASW testing. When applying the MASW technique to develop a one-dimensional (1-D) V_S model, the surface-wave data preferably is acquired using multiple-source offsets at both ends of the array. Rayleigh and Love wave MASW acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively. A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to phase velocity-frequency space in which the surface-wave dispersion curve can be easily identified. Common wave-field transforms include the frequency-wavenumber (f-k) transform, slant-stack transform (τ-p), frequency domain beamformer, and phase-shift transform.

A detailed discussion of the array microtremor method can be found in Okada, 2003. This technique uses 4, or more receivers aligned in a 2-dimensional array. Triangle, circle, semi-circle, and "L" shaped arrays are commonly used, although any 2-dimensional arrangement of receivers can be used. For investigation of the upper 100 m, receivers typically consist of 1 to 4.5 Hz geophones. The triangle array, which consists of several embedded equilateral triangles, is often used as it provides good results with a relatively small number of geophones. With this array, the outer side of the triangle should be at least equal to the desired depth of investigation. The "L" array is useful at sites located at the corner of perpendicular intersecting streets. Typically 20, or more, 30-second noise records are acquired for analysis. The surface wave dispersion curve is typically estimated from array microtremor data using various f-k methods such as beam-forming (Lacoss, *et al*., 1969) and maximum-likelihood (Capon, 1969); and the spatial-autocorrelation (SPAC) method, which was originally based on work by Aki, 1957. The SPAC method has since been extended and modified (Ling and Okada, 1993 and Ohori *et al*., 2002) to permit the use of noncircular arrays, and is now collectively referred to as extended spatial autocorrelation (ESPAC or ESAC).

The refraction microtremor technique (ReMi™), a detailed description of which can be found in Louie, 2001, differs from the more established array microtremor technique in that it uses a linear receiver array rather than a two dimensional array. Unlike the SASW method, which uses an active energy source (i.e. hammer), the microtremor technique records background noise emanating from ocean wave activity, wind noise, traffic, industrial activity, construction, etc. Refraction microtremor field procedures typically consist of laying out a linear array of 24, or more, 4.5 Hz geophones and recording 20, or more, 30 second noise records. These noise records are reduced using the software package SeisOpt® ReMi™ v2.0 by Optim™ Software and Data Services. This package is used to generate and combine the slowness (p) – frequency (f) transform of the noise records. The surface wave dispersion curve is picked at the lower envelope of the surface wave energy identified in the p-f spectrum. It should be noted that other data reduction techniques such as seismic interferometry and extended spatial autocorrelation (ESAC) can also be used to extract surface wave dispersion curves from linear array, passive surface wave data.

The horizontal-to-vertical spectral ratio (H/V spectral ratio or HVSR) technique was first introduced by Nogoshi and Igarashi (1971) and popularized by Nakamura (1989). This technique utilizes single-station recordings of ambient vibrations (microtremor or noise) made with a three-component seismometer. In this method, the ratio of the Fourier amplitude spectra of the horizontal and vertical components is calculated to determine the frequency of the maximum HVSR response (HVSR peak frequency), commonly accepted as an approximation of the fundamental frequency (f_0) of the sediment column overlying bedrock. The HVSR peak frequency associated with bedrock is a function of the bedrock depth and S-wave velocity of the sediments overlying bedrock. The theoretical HVSR response can be calculated for an S-wave velocity model using modeling schemes based on surface wave ellipticity, vertically propagating body waves, or diffuse wavefields containing body and surface waves. The HVSR frequency peak can also be estimated using the quarter-wavelength approximation:

$$f_0 = \frac{\overline{V}_S}{4z}$$

where f_0 is the site fundamental frequency and $\overline{V}_S$ is the average shear-wave velocity of the soil column overlying bedrock at depth z.

The active and passive surface wave techniques complement one another as outlined below:

- SASW/MASW techniques image the shallow velocity structure which cannot be imaged by the microtremor technique and is needed for an accurate V_{S30}/V_{S100ft} estimate.
- Microtremor techniques work best in noisy environments where SASW/MASW depth investigation may be limited.
- In a noisy environment the microtremor technique will usually extend the depth of an SASW/MASW sounding.
- The degree of fit in the overlapping portion of the dispersion curves from the two techniques provides a level of confidence in the results.

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. Several options exist for the Rayleigh wave forward solution: a formulation that takes into account only fundamental-mode Rayleigh wave motion; one that includes all stress waves and incorporates receiver geometry in an SASW test named the 3-D solution (Roesset et al., 1991); one that computes an effective mode for an MASW test but assumes a plane Rayleigh wave and no body wave effects and a multi-mode solution that models different Rayleigh wave modes. Both fundamental mode and multi-mode forward solutions are available for modeling of Love wave data.

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 active and/or passive surface wave 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.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling from MASW or SASW data due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength (V_{R40}) in which case V_{S30} can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on statistical analysis of a large number of surface wave data sets from sites with control by velocities measured in nearby boreholes and has been further tested by Martin and Diehl, 2004, and Albarello and Gargani, 2010.

As with all surface geophysical methods, inversion of surface wave dispersion data does not yield a unique V_S model and there are multiple possible solutions that may equally well fit the experimental data. Based on our experience at other sites, the shear wave velocity models (V_S and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods [Brown, 1998]. The average velocity of the upper 30 m or 100 ft, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model. V_{S30} does not appear to suffer from the non-uniqueness inherent in V_S models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011). Therefore, V_{S30} is more accurately estimated from inversion of surface wave dispersion data than the resulting V_S models.

3 FIELD PROCEDURES

Active surface wave data were acquired along two linear arrays (Array 1M and Array 2M) using the MASW technique (Figure 1). Passive surface wave data were collected using an "L" shaped array (Array 1P) and a nested triangle array (Array 2P). Two HVSR measurements were made near the center of the each passive surface wave array.

A typical MASW field layout is shown in Appendix A. MASW equipment used during this investigation consisted of two Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, seismic cables, a 4 lb hammer, 10 lb sledgehammer, 240 lb accelerated weight drop (AWD), and an aluminum plate. MASW data were acquired along a linear array of 48 geophones spaced 1.5 m apart for line lengths of 70.5 m. Shot points were generally located 1.5 to 30 m from the end geophone locations and at 18 m intervals in the interior of the array. The 4 lb hammer and 10 lb sledgehammer were used for the 1.5 m offset and interior source locations. The AWD was used for all off-end source locations. Data from the transient impacts (hammers) were averaged 10 times, or more, to improve the signal-to-noise ratio. Photographs of typical MASW equipment are presented in Appendix A. All field data were saved to hard disk and documented on field data acquisition forms.

The passive surface wave equipment consisted of two Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, and seismic cables. Ambient noise measurements were made for 25 minutes at a 2 ms sample rate (50, 30 second records) along a 48 channel "L" shaped array (Array 1P). Ambient noise measurements were also made for over 30 minutes along a 43 channel nested triangle array (Array 2P) where geophones were distributed along 2 equilateral triangles, sharing a common center point, with side lengths of 30 and 60 m. All passive surface wave data were stored on a laptop computer for later processing. The field geometry and associated files names were documented in field data acquisition forms.

HVSR data were acquired near the center of Array 1P (Figure 1) using a Moho Science and Technology Tromino ENGR seismometer. Additional HVSR data were acquired the near the center of Array 2P (Figure 1) using a Nanometrics Trillium Compact broadband seismometer. HVSR measurements were made for the duration of array microtremor acquisition (> 30 minutes) with ambient noise data recorded at 128 samples per second. Microtremor data were stored in the instruments internal memory, downloaded, and converted to ASCII format files at the end of data acquisition.

4 DATA REDUCTION AND MODELING

HVSR data were reduced using the Geopsy Version 2.9.1 software package (http://www.geopsy.org) developed by Marc Wathelet, ISTerre, Grenoble, France with the help of many other researchers.

Microtremor data recorded by the Tromino and Trillium were exported to ASCII format. The data file was then loaded into the Geopsy software package, where data file columns containing the vertical and horizontal (north and east) components and the sample rate were specified. HVSR was typically calculated over a frequency range dependent upon the observed site response and using a time window length of 60 s. Time windows were automatically picked. Fourier amplitude spectra were calculated after applying a 5% cosine taper and smoothed by the Konno and Ohmachi filter with a smoothing coefficient value of 30. The vertical amplitude spectra were divided by the root-mean-square (RMS) of the horizontal amplitude spectra to calculate the HVSR for each time window and the average HVSR. Time windows containing clear transients (nearby foot or vehicular traffic) or yielding poor quality results were then deleted and the computations repeated. The average HVSR peak frequency and standard deviation from all time windows used for analysis were computed and presented along with the standard deviation of the HVSR amplitudes for all time windows.

The MASW data were reduced using the software Seismic Pro Surface V8.0 developed by Geogiga using the following steps:

- Input seismic record into software.
- Enter receiver spacing, geometry, offset range used for analysis, etc.
- Apply wavefield transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify and pick Rayleigh wave dispersion curve.
- Repeat for all seismic records.
- Apply near-field criteria (maximum wavelength equal 1 to 1.3 times the source to midpoint of receiver array distance for Rayleigh wave data and 1.5 times the source to midpoint of receiver array distance for Love wave data).
- Merge multiple dispersion curves extracted from the MASW data collected along each seismic spread (different source types, source locations, different receiver offset ranges, etc.).
- Convert dispersion curves to required format for modeling.
- Calculate a representative dispersion curve for the combined MASW dispersion data using a moving average polynomial curve fitting routine.

A unique data acquisition and data reduction procedure used by **GEO***Vision* for 1-D MASW soundings is the use of multiple source types and source locations during data acquisition and the extraction of multiple dispersion curves from the different source locations, and limited offset range receiver gathers associated with each source location. The use of such a data acquisition and processing strategy ensures that the modeled dispersion curve covers as wide a frequency/wavelength range as possible and is representative of average conditions beneath the array.

The array microtremor data were reduced using the software Seisimager SW developed by Oyo Corporation/Geometrics, Inc. and the following steps:

- Input all seismic records for a dataset into software.
- Load geometry (x and y positions) for each channel in seismic records.
- Calculate the SPAC coefficients for each seismic record and average.
- For each frequency calculate the RMS error between the SPAC coefficients and a Bessel function of the first kind and order zero over a user defined phase velocity range and velocity step.
- Plot an image of RMS error as a function for frequency (f) and phase velocity (v).
- Identify and pick the dispersion curve as the continuous trend on the f-v image with the lowest RMS error.
- Convert dispersion curves to appropriate format for modeling.
- Combine multiple passive dispersion curves, as appropriate.
- Calculate a representative dispersion curve for the passive dispersion data using a moving average polynomial curve fitting routine.

The representative dispersion curves from the active and passive surface wave data at each sounding location were combined and the moving average polynomial curve fitting routine in WinSASW V3 was used to generate a composite representative dispersion curve for modeling. During this process the active surface wave data were given equal weight to the combined passive surface wave data in the overlapping wavelength range. An equal logarithm wavelength sample rate was used for the representative dispersion curve to reflect the gradual loss in model resolution with depth.

The final composite representative dispersion curve was loaded into a forward or inverse modeling software package to develop a V_S model. Rayleigh wave dispersion data were modeled using the fundamental mode solution in the WinSASW V3. During this process an initial velocity model was generated based on general characteristics of the dispersion curve and the forward or inverse modeling routine utilized to adjust the layer V_S until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted and the inversion process repeated until a V_S model was developed with low RMS error between the observed and calculated dispersion curves. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e. less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.9 to 2.4 gm/cm^3 were used in the profile for subsurface soils/rock depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible (±2%) affect on the estimated V_S from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity, V_P, for unsaturated sediments was estimated using a Poisson's ratio, ν, of 0.3 and the relationship:

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

Poisson's ratio has a larger affect than density on the estimated V_S from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity (V_R), V_S and *v:*

$$V_R = V_S\,[(0.862 + 1.14\,v)/(1 + v)]$$

Using this relationship, it can be shown that V_S derived from V_R only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$V_S = 1.16V_R \text{ for } v = 0$$
$$V_S = 1.05V_R \text{ for } v = 0.5$$

The realistic range of the Poisson's ratio for typical unsaturated sediments is about 0.25 to 0.35. Over this range, V_S derived from modeling of Rayleigh wave dispersion data will vary by about 5%. An intermediate Poisson's ratio of 0.3 was selected for modeling to minimize any error associated with the assumed Poisson's ratio.

To reduce errors associated with expected high Poisson's ratio of saturated sediments, seismic refraction first arrival data were reviewed in the MASW seismic records to determine if there was any evidence of a refractor associated with the top of the saturated zone in the upper 20 to 30 m. If a saturated zone refractor was identified, interactive layer based modeling was conducted to estimate the depth to and V_P (>1,500 m/s) of the saturated sediments, which was then constrained when modeling the dispersion data. Poisson's ratio of saturated, soft sediments can be slightly less than 0.5, and gradually decrease with depth as the sediments become stiffer.

The predicted HVSR response based on the diffuse field assumption was computed for all V_S models where HVSR data were available using the software package *HV-Inv* Release 1.0 Beta, which is summarized in García-Jerez, et al., 2016, and compared to the observed HVSR peaks.

5 INTERPRETATION AND RESULTS

The observed HVSR data collected near the center of Array 1P and Array 2P are presented as Figure 2. The fit of the theoretical dispersion curve to the experimental data collected along Array 1M and the modeled V_S profile for the surface wave sounding is presented as Figure 3. The fit of the theoretical dispersion curve to the experimental data collected along Array 2M and Array 2P and the combined modeled V_S profile for the surface wave sounding is presented as Figure 4. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The V_S profile used to match the field data is provided in tabular form in metric and imperial units as Tables 1 to 6, respectively.

The observed HVSR data collected near the center of Array 1P and 2P are presented as Figure 2. The HVSR peak is approximately the fundamental site frequency. There is a peak in the HVSR data at a frequency of about 2.7 Hz for Array 1 and about 2.5 Hz for Array 2, which is expected to be associated with the top of the bedrock. The frequency HVSR peaks at different frequencies indicates that bedrock is dipping/undulating in the vicinity of the measurement location. Bedrock is deeper beneath the lower frequency peak at the measurement location of Array 2.

The V_S model for Array 1 (Figure 3 and Tables 1 and 2) was developed from the surface wave dispersion data derived from MASW data acquired along Array 1M. The passive surface wave dispersion data from the "L" shaped array (Array 1P) were not used for modeling purposes. Inspection of the seismic refraction first arrival data and the HVSR data indicates that rock is likely getting deeper in the northern portion of the site. The subsurface beneath the south to north leg of the "L" shaped array will then contain a high degree of lateral velocity variation. Since the passive surface wave data were not need to extend the depth of investigation to 30 m or more, it is not presented. The estimated depth of investigation for Array 1 is about 60 m; the model is most reliable in the upper 45 m.

The V_S model for Array 1 has a 3 m thick surficial layer of sediments with modeled V_S of about 332 m/s, underlain by a layer that extends to a depth of about 18 m and has V_S of about 394 to 398 m/s. V_S increases slightly to about 460 m/s below this layer at a depth of about 18 m and continues to increases to about 496 m/s at a depth of approximately 30 m. There is an abrupt increase in modeled V_S to 1,529 m/s at a depth of 44 m, which is likely related to the top of bedrock. V_S models from the surface wave dispersion data are non-unique models and the depth of bedrock may vary by at least 10% of the depth.

The V_S models for Array 2 (Figure 4 and Tables 3 to 6) were developed from the surface wave dispersion data derived from MASW (Array 2M) and passive surface wave data acquired along the nested triangle array (Array 2P). The Rayleigh wave phase velocities from the passive surface wave array are generally in excellent agreement with those from the MASW data in the regions of overlapping wavelength. The estimated depth of investigation for the combined active and passive surface wave sounding is about 80 m.

The V_S models for Array 2 have a 2.5 m thick surficial layer of sediments with modeled V_S of about 338, underlain by a layer that extends to a depth of about 8 m and has V_S of about 374 m/s. V_S increases slightly to about 390 m/s below this layer at a depth of about 8 m and continues to increases to about 441 m/s at a depth of approximately 17 m. V_S increases to about 532 m/s at a depth of about 30 m. There is an abrupt increase in modeled V_S to 1,153 m/s (Model 1) and 1,122 m/s (Model 2) at a depth of 49 m, which is likely related to the top of bedrock. V_S models from the surface wave dispersion data are non-unique models and the depth of bedrock may vary by at least 10% of the depth.

The computed HVSR is presented, along with the observed HVSR data for Arrays 1 and 2, as Figure 5. In both cases, the width of the calculated HVSR peak fit better assuming that the ambient noise field consisted of Rayleigh waves only rather than the Rayleigh and Love wave assumption. There is decent agreement in the observed and calculated HVSR response for Array 1 demonstrating that the V_S model satisfies both surface wave dispersion and observed HVSR data. There is decent agreement in the observed and calculated HVSR response width for model 1 of Array 2. However, a much higher half space velocity is needed to better fit the HVSR peak amplitude. Model 2 of Array 2 displays increase of V_S to about 1,739 m/s at a depth of about 69 m possibly related to weathered rock becoming more competent at depth. Model 2 from Array 2 better demonstrates a V_S model satisfying both surface wave dispersion and observed HVSR data.

The average shear wave velocity to a depth of 30 m (V_{S30}) is 411 m/s beneath Array 1. The average shear wave velocity to a depth of 100 ft, V_{S100ft}, is 1,352 ft/s beneath Array 1. The average shear wave velocity to a depth of 30 m (V_{S30}) is 402 m/s beneath Array 2. The average shear wave velocity to a depth of 100 ft, V_{S100ft}, is 1,324 ft/s beneath Array 2. Therefore, according to the NEHRP provisions of the Uniform Building Code, the site is classified as Site Class C, very dense soil and soft rock.

6 CONCLUSIONS

Active and passive surface wave measurements were made at a lot north of the Kaiser Permanente hospital located at 27300 Iris Avenue in Moreno Valley, California to develop two S-wave velocity profiles to a depth of greater than 30 m and estimate V_{S30}. The locations of the geophysical testing arrays are presented in Figure 1.

The observed HVSR data collected near the center of Arrays 1P and 2P are presented as Figure 2. The surface wave dispersion data and V_S model for Array 1 are presented as Figure 2 and in Tables 1 and 2. The surface wave dispersion data and V_S models for Array 2 are presented as Figure 4 and in Tables 3 to 6. Depth of investigation of the two V_S models is about 60 and 80 m, respectively. Calculated HVSR from the V_S model for Arrays 1 and 2 are in decent agreement with the observed HVSR at the center of the arrays (Figure 5).

V_{S30} and V_{S100ft} are 411 m/s and 1,352 ft/s, respectively, for Array 1. V_{S30} and V_{S100ft} are 402 m/s and 1,324 ft/s, respectively, for Array 2. Therefore, according to the Uniform and International Building Codes, the area in the vicinity of the surface wave arrays is classified as Class C, very dense soil and soft rock.

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8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Prepared by

7/21/2017

David Carpenter — Date

California Professional Geophysicist, P. Gp. 1088
GEO*Vision* Geophysical Services

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

TABLES

Table 1 V_S Model – Array 1 (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	3	332	622	0.300	1.90
3	6	394	737	0.300	1.95
9	9	398	745	0.300	1.95
18	12	460	860	0.300	2.00
30	14	496	1,750	0.300	2.05
44	>16	1,529	2,860	0.300	2.40

Table 2 V_S Model – Array 1 (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	9.8	1,090	2,040	0.300	119
9.8	19.7	1,293	2,418	0.300	122
29.5	29.5	1,306	2,443	0.300	122
59.1	39.4	1,508	2,821	0.300	125
98.4	45.9	1,627	5,741	0.300	128
144.4	>52.5	5,016	9,385	0.300	150

Table 3 V_S Model – Array 2 Model 1 (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	2.5	338	633	0.300	1.90
2.5	5.5	374	700	0.300	1.95
8	9	390	730	0.300	1.95
17	13	441	825	0.300	2.00
30	19	532	1,750	0.300	2.05
49	>11	1,153	2,157	0.300	2.20

Table 4 V_S Model – Array 2 Model 1 (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	8.2	1,110	2,077	0.300	119
8.2	18.0	1,227	2,296	0.300	122
26.2	29.5	1,280	2,394	0.300	122
55.8	42.7	1,448	2,708	0.300	125
98.4	62.3	1,745	5,741	0.300	128
160.8	>36.1	3,782	7,075	0.300	137

Table 5 V_S Model – Array 2 Model 2 (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	2.5	339	633	0.300	1.90
2.5	5.5	374	700	0.300	1.95
8	9	390	730	0.300	1.95
17	13	442	827	0.300	2.00
30	19	522	1,750	0.300	2.05
49	20	1,122	2,099	0.300	2.20
69	>11	1,739	3,252	0.300	2.40

Table 6 V_S Model – Array 2 Model 2 (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	8.2	1,111	2,078	0.300	119
8.2	18.0	1,227	2,296	0.300	122
26.2	29.5	1,280	2,394	0.300	122
55.8	42.7	1,451	2,714	0.300	125
98.4	62.3	1,714	5,741	0.451	128
160.8	65.6	3,681	6,886	0.300	137
226.4	>36.1	5,704	10,671	0.300	150

FIGURES

Legend

- Active Surface Wave Array (MASW)
- Passive Surface Wave Array
- H/V Spectral Ratio Measurement Location

NOTES:
1. California State Plane Coordinate System, NAD 83, Zone VI (0406), US Survey Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

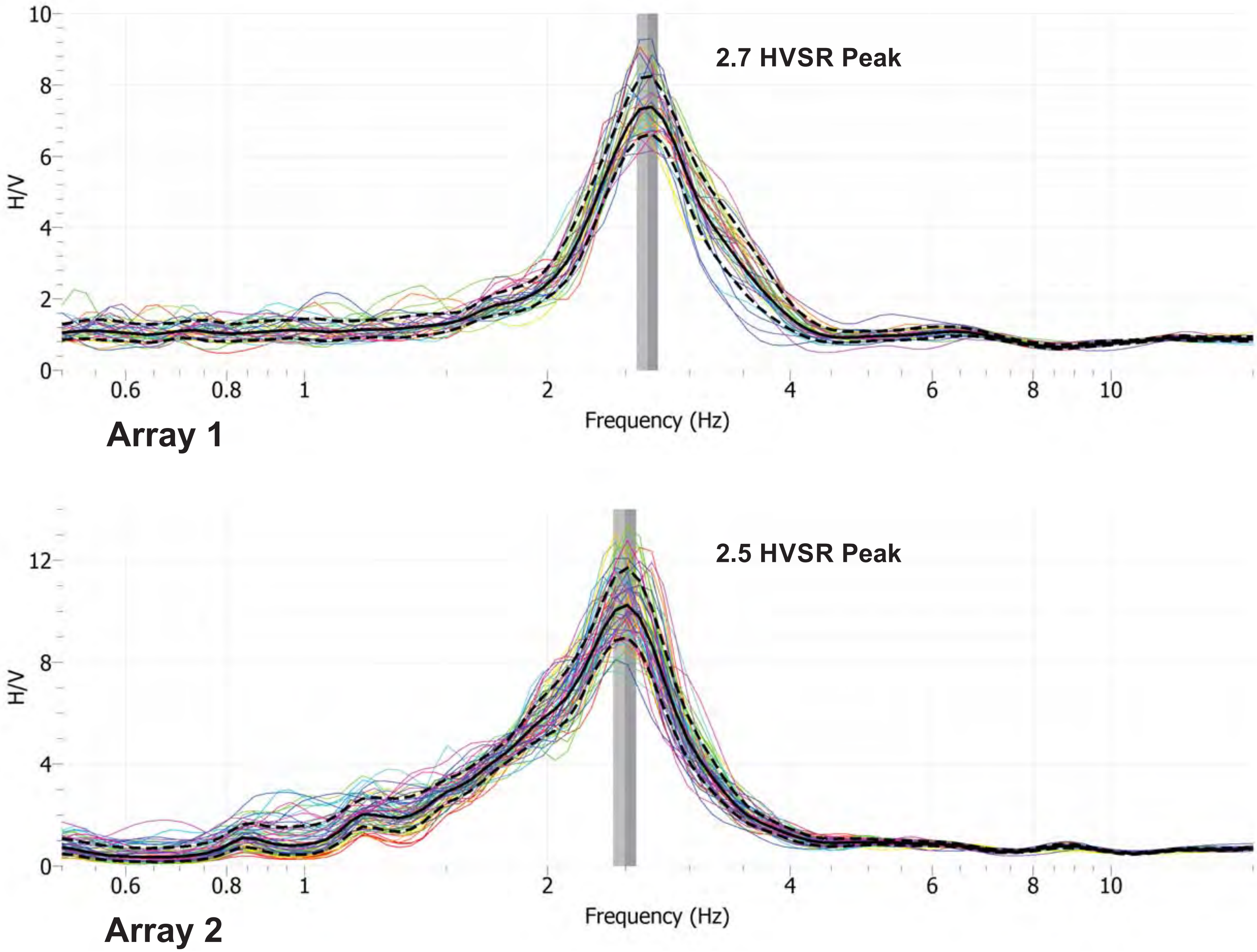
GEOVision geophysical services

Date:	7/20/2017
GV Project:	17242
Developed by:	D Carpenter
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	17242_1.MXD

FIGURE 1
SITE MAP

SITE LOCATED AT
27300 IRIS AVENUE
MORENO VALLEY, CALIFORNIA

PREPARED FOR
GEOBASE, INC.



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Project No.: 17242

Date: JULY 20, 2017

Drawn By: CARPENTER

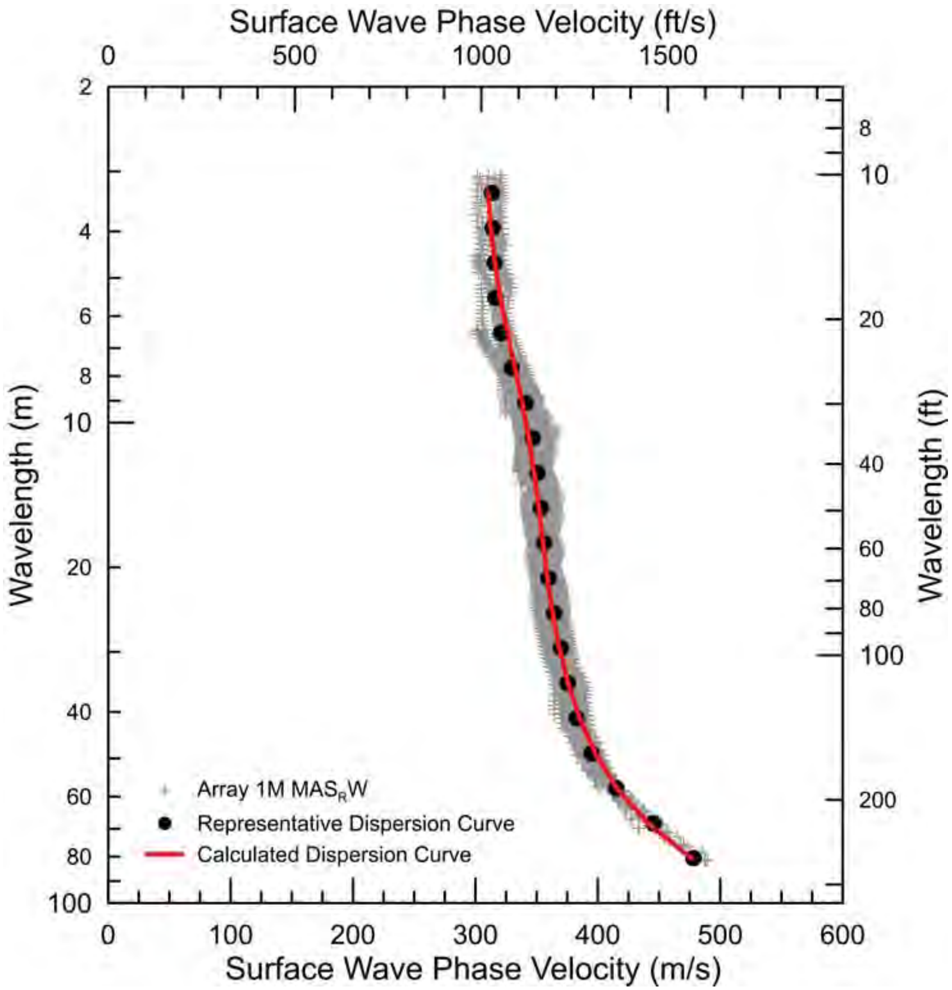
Approved By: CARPENTER

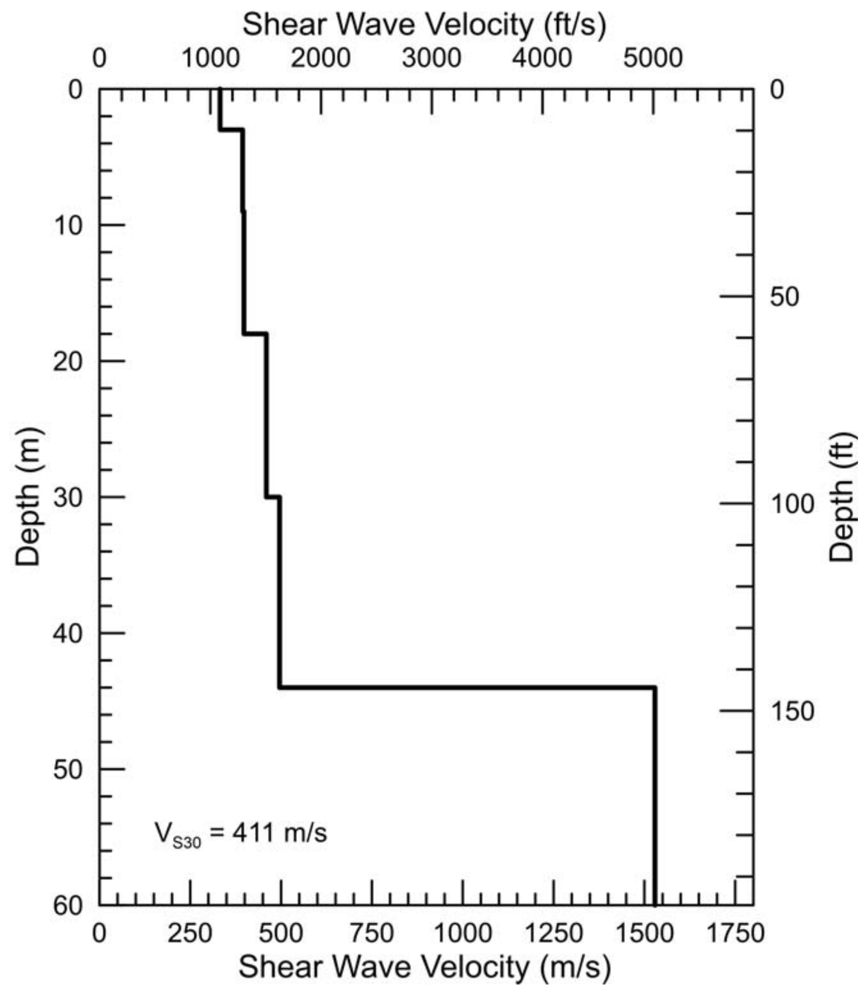
File P:\Project Files\2017\17242 - GEOBASE...SW\Report\Figure2.cdr

FIGURE 2
OBSERVED H/V SPECTRAL RATIO

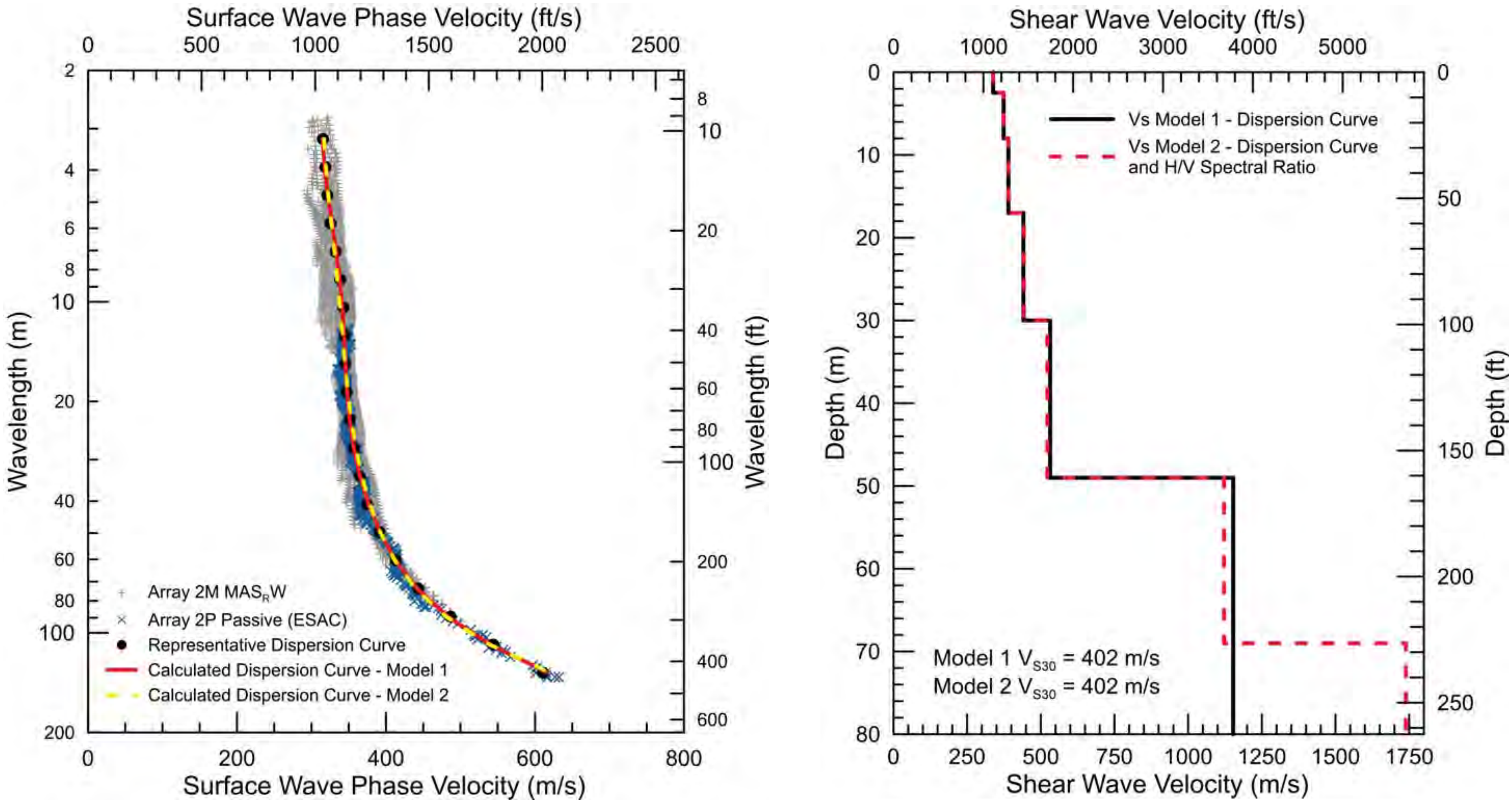
27300 IRIS AVENUE
MORENO VALLEY, CALIFORNIA

PREPARED FOR
GEOBASE, INC.

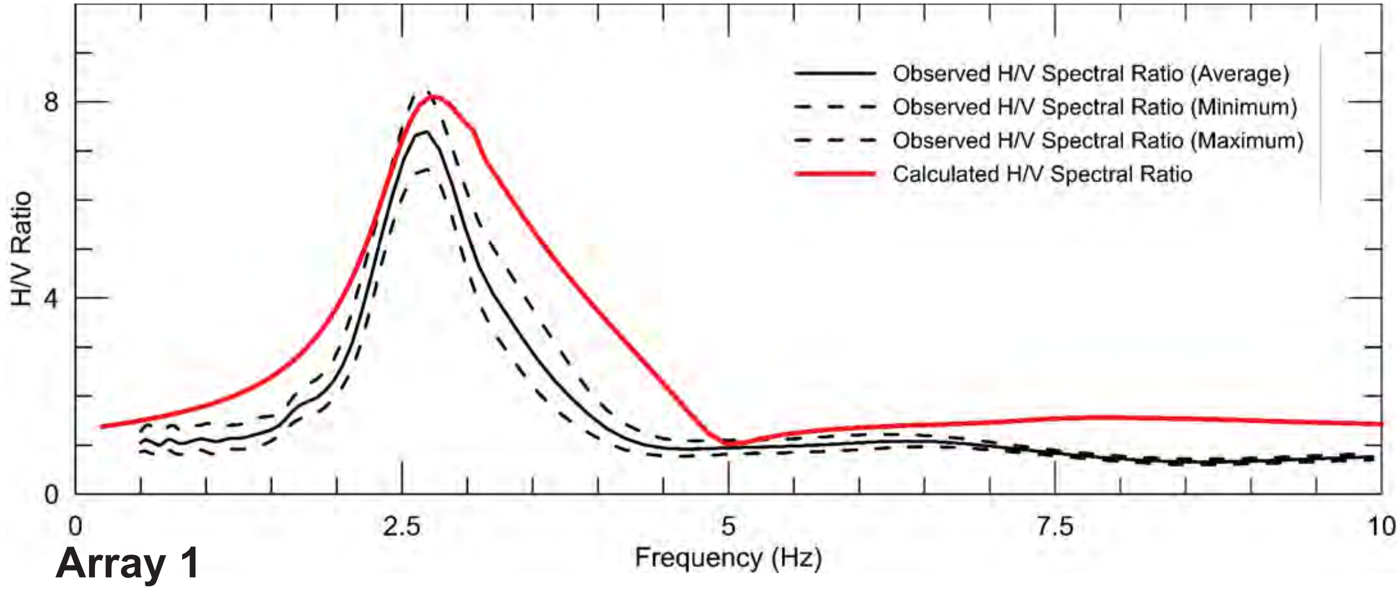




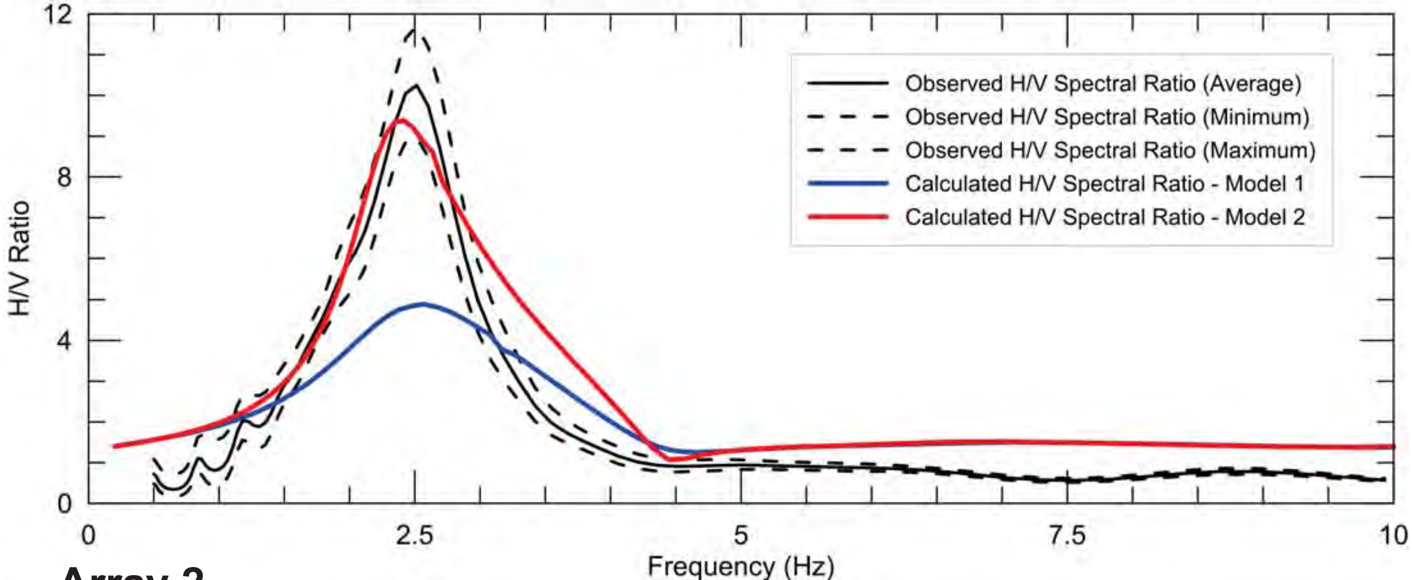
GEOVision geophysical services	FIGURE 3 SURFACE WAVE MODEL - ARRAY 1
Project No.: 17242 Date: JULY 20, 2017	27300 IRIS AVENUE MORENO VALLEY, CALIFORNIA
Drawn By: CARPENTER Approved By: CARPENTER File P:\Project Files\2017\17242 - GEOBASE...SW\Report\Figure3.cdr	PREPARED FOR GEOBASE, INC.



GEOVision geophysical services	FIGURE 4 SURFACE WAVE MODEL - ARRAY 2
Project No.: 17242	27300 IRIS AVENUE MORENO VALLEY, CALIFORNIA
Date: JULY 20, 2017	
Drawn By: CARPENTER	PREPARED FOR GEOBASE, INC.
Approved By: CARPENTER	
File P:\Project Files\2017\17242 - GEOBASE...SW\Report\Figure4.cdr	



Array 1



Array 2

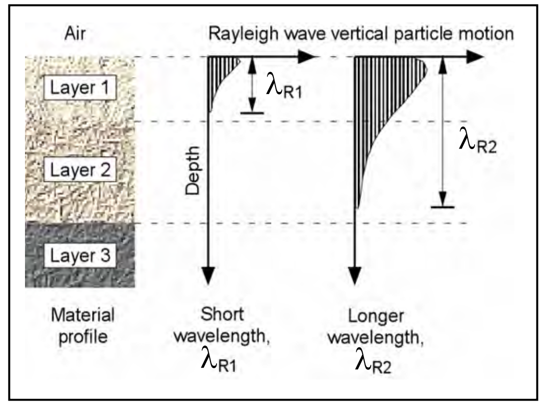
GEOVision geophysical services	FIGURE 5 OBSERVED & CALCULATED H/V SPECTRAL RATIO
Project No.: 17242 Date: JULY 20, 2017	27300 IRIS AVENUE MORENO VALLEY, CALIFORNIA
Drawn By: CARPENTER Approved By: CARPENTER File P:\Project Files\2017\17242 - GEOBASE...SW\Report\Figure5.cdr	PREPARED FOR GEOBASE, INC.

APPENDIX A

ACTIVE AND PASSIVE SURFACE WAVE TECHNIQUES

Overview

Active and passive surface wave techniques are relatively new in-situ seismic methods for determining shear wave velocity (V_S) profiles. Testing is performed on the ground surface, allowing for less costly measurements than with traditional borehole methods. The basis of surface wave techniques is the dispersive characteristic of Rayleigh waves when traveling through a layered medium. Rayleigh wave velocity is determined by the material properties (primarily shear wave velocity, but also to a lesser degree compression wave velocity and material density) of the subsurface to a depth of approximately 1 to 2 wavelengths. As shown in the adjacent diagram, longer wavelengths penetrate deeper and their velocity is affected by the material properties at greater depth. Surface wave testing consists of measuring the surface wave dispersion curve at a site and modeling it to obtain the corresponding shear wave velocity profile.



Active Surface Wave Techniques

Active surface wave techniques measure surface waves generated by dynamic sources such as hammers, weight drops, electromechanical shakers, vibroseis and bulldozers. These techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods.

Hammer Energy Sources

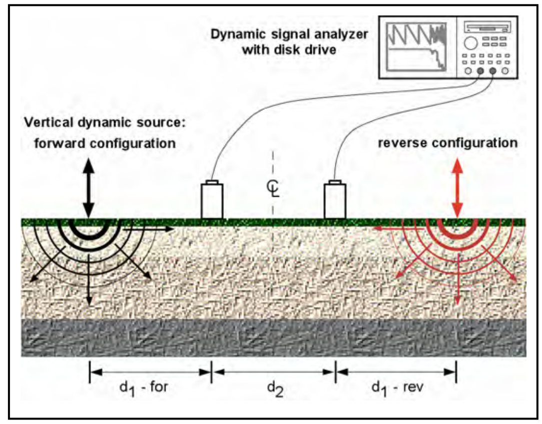
Accelerated Weight Drop

Electromechanical Shaker

Bulldozer Energy Source

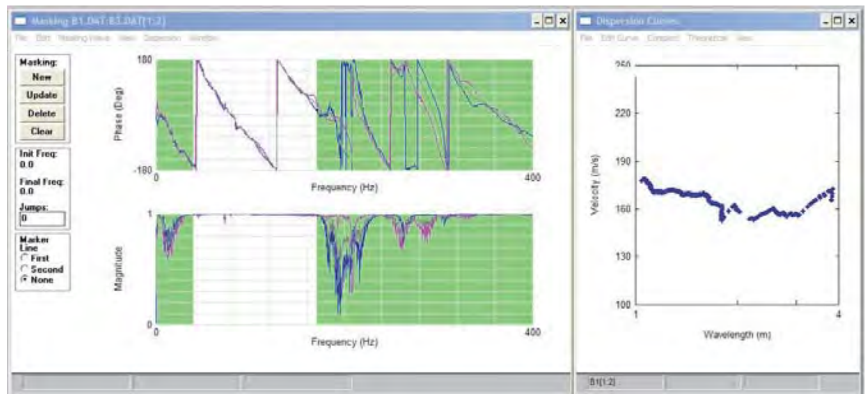
1124 Olympic Drive, Corona, California 92881 ph. 951-549-1234 fx 951-549-1236 www.geovision.com

The SASW method is optimized for conducting V_S depth soundings. A dynamic source is used to generate surface waves of different wavelengths (or frequencies) which are monitored by two or more receivers at known offsets. An expanding receiver spread and optimized source-receiver geometry are used to minimize near field effects, body wave signal and attenuation. A dynamic signal analyzer is typically used to calculate the phase and coherence of the cross spectrum of the time history data collected at a pair of receivers. During data analysis, an interactive masking process is used to discard low quality data and to unwrap the phase spectrum, as shown in the figure below. The dispersion curve (Rayleigh wave phase velocity versus frequency or alternatively wavelength) is calculated from the unwrapped phase spectrum.



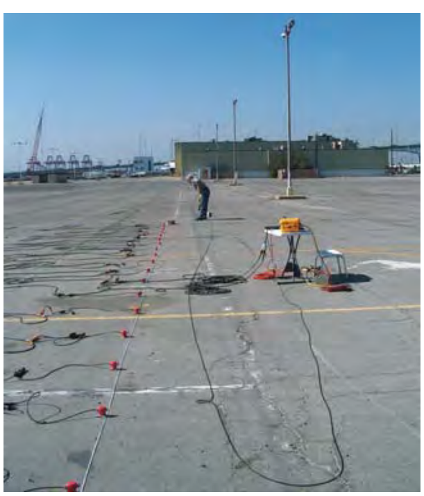
SASW Setup

HP Dynamic Signal Analyzer

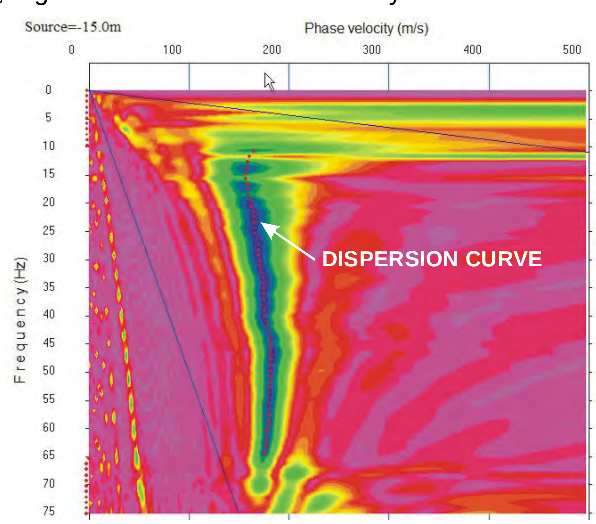


Masking of Wrapped Phase Spectrum and Resulting Dispersion Curve

The MASW field layout is similar to that of the seismic refraction technique. Twenty four, or more, geophones are laid out in a linear array with 1 to 2m spacing and connected to a multi-channel seismograph as shown below. This technique is ideally suited to 2D V_S imaging, with data collected in a roll-along manner similar to that of the seismic reflection technique. The source is offset at a predetermined distance from the near geophone usually determined by field testing. The Rayleigh wave dispersion curve is obtained by a wavefield transformation of the seismic record such as the f-k or τ-p transforms. These transforms are very effective at isolating surface wave energy from that of body waves. The dispersion curve is picked as the peak of the surface wave energy in slowness (or velocity) – frequency space as shown. One advantage of the MASW technique is that the wavefield transformation may not only identify the fundamental mode but also higher modes of surface waves. At some sites, particularly those with large velocity inversions, higher surface wave modes may contain more energy than the fundamental mode.



MASW Field Setup

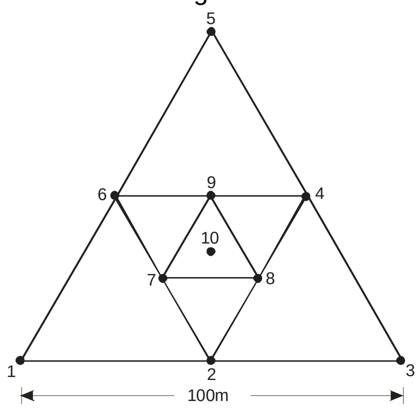


Wavefield Transform of MASW data

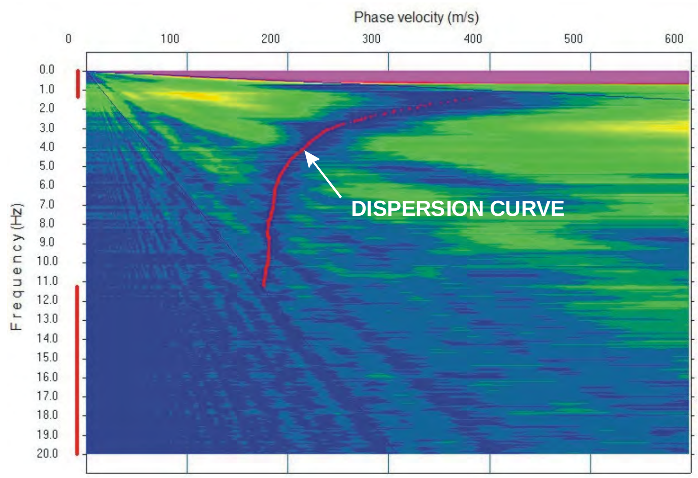
Passive Surface Wave Techniques

Passive surface wave techniques measure noise; surface waves from ocean wave activity, traffic, factories, etc. These techniques include the array microtremor and refraction microtremor (REMI) techniques.

The array microtremor technique typically uses 7 or more 4.5- or 1-Hz geophones arranged in a two-dimensional array. The most common arrays are the triangle, circle, semi-circle and "L" arrays. The triangle array, which consists of several embedded equilateral triangles, is often used as it provides good results with a relatively small number of geophones. With this array the outer side of the triangle should be at least as long as the desired depth of investigation. Typically, fifteen to twenty 30-second noise records are acquired for analysis. The spatial autocorrelation (SPAC) technique is one of several methods that can be used to estimate the Rayleigh wave dispersion curve. A first order Bessel function is fit to the SPAC function to determine the phase velocity for particular frequency. The image shown below shows the degree of fitness of the Bessel function to the SPAC function for a wide range of phase velocity and frequency. The dispersion curve, is the peak (best fit), as shown in the figure below.



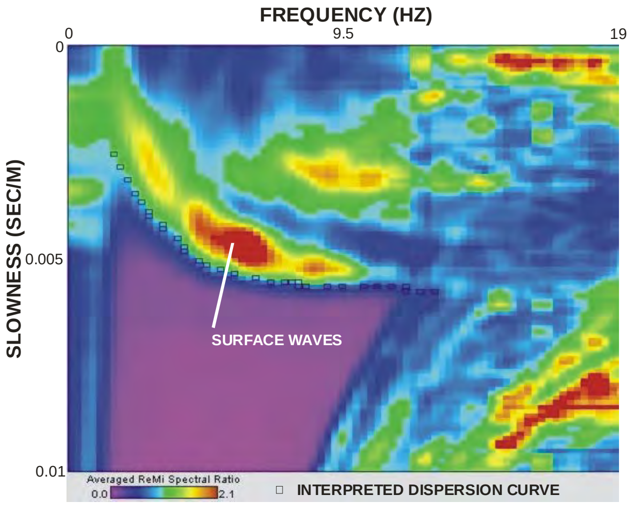
Triangle Array Geometry



Dispersion Curve from Array Microtremor Measurements

The refraction microtremor (REMI) technique uses a field layout similar to the seismic refraction method (hence its name). Twenty-four, 4.5 Hz geophones are laid out in a linear array with a spacing of 6 to 8m and fifteen to twenty 30-second noise records are acquired. A slowness-frequency (p-f) wavefield transform is used to separate Rayleigh wave energy from that of other waves. Because the noise field can originate from any direction, the wavefield transform is conducted for multiple vectors through the geophone array, all of which are summed. The dispersion curve is defined as the lower envelope of the Rayleigh wave energy in p-f space. Because the lower envelope is picked rather than the energy peak (energy traveling along the profile is slower than that approaching from an angle), this technique may be somewhat more subjective than the others, particularly at low frequencies. The SPAC technique can also be used to extract the surface wave dispersion curve from linear array microtremor data providing there are omni-directional noise sources.

Refraction Microtremor Array Layout



Wavefield Transform of REMI Data

Depth of Investigation

Active surface wave investigations typically use various sized sledge hammers to image the shear wave velocity structure to depths of up to 15m. Weight drops and electromechanical shakers can often be used to image to depths of 30m. Bulldozers and vibroseis trucks can be used to image to depths as great as 100m. Passive surface wave techniques can often image shear wave velocity structure to depths of over 100m, given sufficient noise sources and space for the receiver array. Large passive arrays, utilizing long-period seismometers with GPS clocks have been used to image shear wave velocity structure to depths of several kilometers.

Combined Active and Passive Surface Wave Testing

The combined use of active and passive techniques may offer significant advantages on many investigations. It can be very costly to mobilize large energy sources for 30m/100ft active surface wave soundings. In urban environments, the combined use of active and passive surface wave techniques can image to these depths without the need for large energy sources. We have found that dispersion curves from active and passive surface wave techniques are generally in good agreement, making the combined use of the two techniques viable. It is not recommended that passive surface wave techniques be applied alone for UBC/IBC site classification investigations. Microtremor techniques do not generally characterize near surface velocity, which may have a significant impact of the average shear wave velocity of the upper 30m or 100ft and so should always be used in conjunction with SASW or MASW. An SASW sounding to a depth of 30m requires at least a 60m linear array. If sufficient space is not available for this, it may be possible to use a 45m triangle array on the site or place a 100-200m long REMI array along an adjacent sidewalk or an "L" array at an adjacent street intersection.

Microtremor Measurements along Sidewalk

Modeling

There are several options for interpreting surface wave dispersion curves, depending on the accuracy required in the shear wave velocity profile. A simple empirical analysis can be done to estimate the average shear wave velocity profile. For greater accuracy, forward modeling of fundamental-mode Rayleigh wave dispersion as well as full stress wave propagation can be performed using several software packages. A formal inversion scheme may also be used. With many of the analytical approaches, background information on the site can be incorporated into the model and the resolution of the final profile may be quantified.

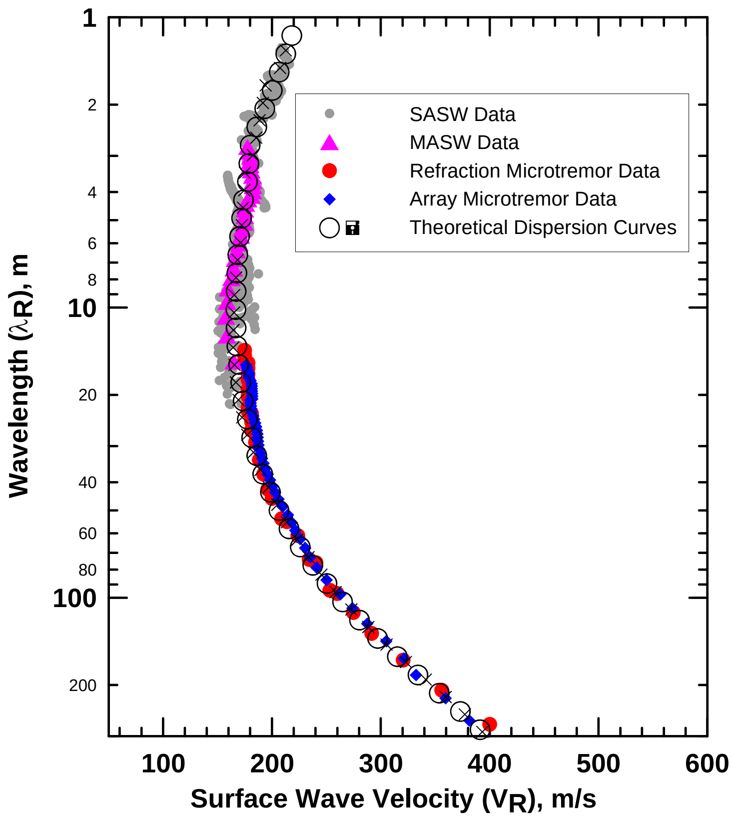
Applications

Active and passive surface wave testing can be used to obtain V_S profiles for:

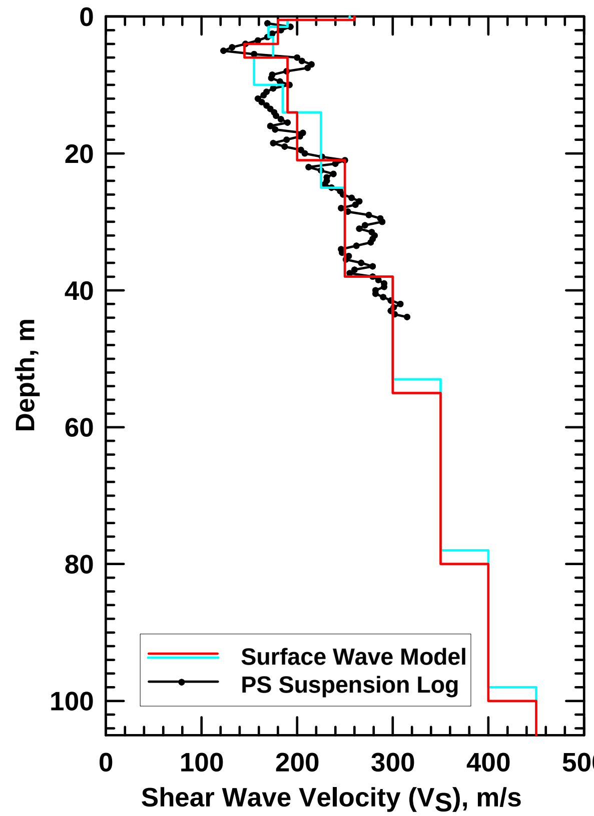
- UBC/IBC site classification for seismic design
- Earthquake site response
- Seismic microzonation
- Liquefaction analysis
- Soil compaction control
- Mapping subsurface stratigraphy
- Locating potentially weak zones in earthen embankments and levees

Case History

The figures below show the surface wave dispersion curves and alternative shear wave velocity models for a site in Los Angeles, California. All of the previous figures illustrating SASW, MASW, array and refraction microtremor techniques were from this site. The dispersion curves from all four methods are shown on the left along with the theoretical dispersion curves for alternative S-wave velocity versus depth models on the right. Conditions at this site were very poor for active surface wave techniques because of the presence of very low velocity hydraulic fill. In fact, with active surface wave techniques it was only possible to image to a depth of about 12.5m with energy sources typically capable of imaging to 30m. There is excellent agreement in the dispersion curves generated from all of the methods over the overlapping wavelength ranges. The minor differences probably result from variable velocity of the hydraulic fill within the sampling volume of the specific methods. Two Vs versus depth models were generated to illustrate the difficulty modeling the highly variable, near surface velocity structure evident in the PS log. The two surface wave models yielded similar values for the average shear-wave velocity of the upper 30m (V_S30), 201 and 202 m/s, illustrating that Vs30 is much more tightly constrained than the actual layer thicknesses and velocities in the models. V_S30 estimated from the PS log (194 m/s) is within 4% of that estimated from the two surface wave models (201 and 202 m/s). The small differences in V_S30 between the two methods may easily result from the different sampling regimes (borehole versus large area) rather than errors in either of the methods.



Field Data and Theoretical Dispersion Curve



V_S Model

In contrast to borehole measurements which are point estimates, surface wave testing is a global measurement, that is, a much larger volume of the subsurface is sampled. The resulting profile is representative of the subsurface properties averaged over distances of up to several hundred feet. Although surface wave techniques do not have the layer sensitivity or accuracy (velocity and layer thickness) of borehole techniques; the average velocity over a large depth interval (i.e. the average shear wave velocity of the upper 30m or 100ft) is very well constrained. Because surface wave methods are non-invasive and non-destructive, it is relatively easy to obtain the necessary permits for testing. At sites that are favorable for surface wave propagation, active and passive surface wave techniques allow appreciable cost and time savings.

HVSR METHOD

HORIZONTAL/VERTICAL SPECTRAL RATIO (HVSR) METHOD

Overview

The HVSR method is a single station passive seismic method for estimating the fundamental site period (frequency), which is related to the thickness and average shear (S) wave velocity of the sediments overlying bedrock. It should be noted that the HVSR frequency peak is typically very close to, but not always identical to, the fundamental site frequency. Passive seismic techniques involve the recording of ambient noise emanating from ocean wave activity, atmospheric conditions, wind effects, traffic, industrial activity, construction activities, etc., and collectively are referred to as microseisms. Typically, microseisms with frequencies below 1 Hz have natural origins, whereas those with frequencies above 1 Hz are largely due to human activities. The HVSR technique is most often utilized as part of seismic microzonation studies of sedimentary basin, but is recently finding use in hydrogeologic studies to identify potential drill sites with bedrock at the greatest depth.

Tromino ENGR used for HVSR measurements in shallow basins

Procedure

The HVSR method uses a 3-component seismometer to record ambient noise for a period of time between 15 minutes and several hours depending on the estimated thickness of the sediments and ambient noise conditions. The ratio of the Fourier amplitude spectra of the horizontal and vertical components is calculated to determine the frequency of the maximum HVSR response, commonly accepted as an approximation of the fundamental frequency (f_0) of the sediment column overlying bedrock.

Trillium Compact 120 second seismometer used for HVSR measurements in deep basins

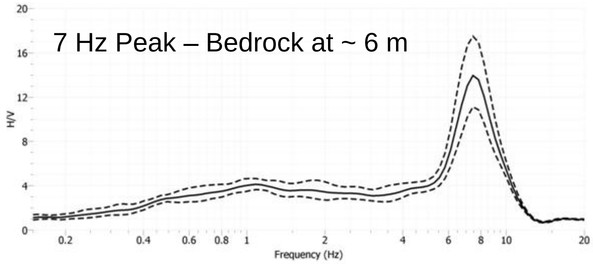
There are several options for interpreting HVSR data, depending upon the objectives of the investigation, including: joint inversion of the HVSR curve or peak frequency with surface wave dispersion curves, quarter-wavelength correlation, or simple empirical analysis using HVSR data collected at locations with known bedrock depth.

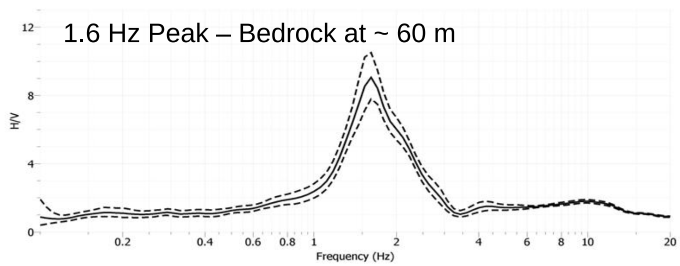
The quarter-wavelength approximation is:

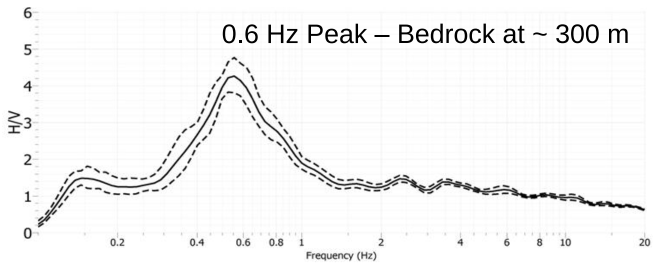
$$f_0 = \frac{\overline{V}_S}{4z}$$

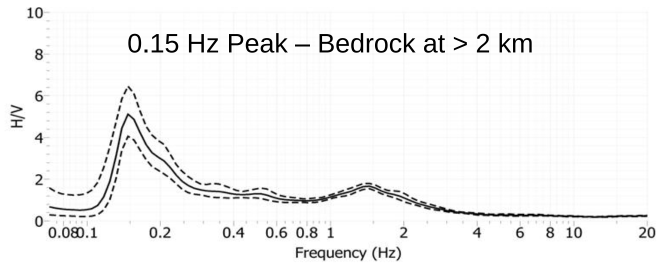
where f_0 is the site fundamental frequency, $\overline{V}_S$ is the average shear-wave velocity of the soil column overlying bedrock at depth *z. This relationship* can be used to estimate the average shear wave velocity profile of the sediments when depth to bedrock is known or vice versa. As evident in this relationship, the fundamental site frequency is inversely proportional to bedrock depth; therefore, shallow bedrock will be associated with a high frequency peak and vice versa. If active and passive surface wave soundings are conducted in the deeper portion of sedimentary basins, it may be possible to develop an average S-wave velocity versus depth profile for the basin and use this along with the HVSR frequency peak to estimate bedrock depth. Alternatively, HVSR measurements can be made at locations with known depth to bedrock and a correlation between HVSR peak frequency and bedrock depth developed.

The figures below show HVSR data collected at sites with different approximate basement depths. Sites with shallow rock will have HVSR peaks at several Hz, while deep sedimentary basins will have HVSR peaks at a fraction of a hertz.

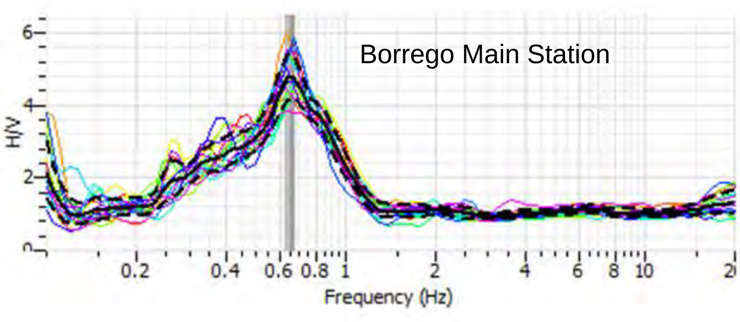


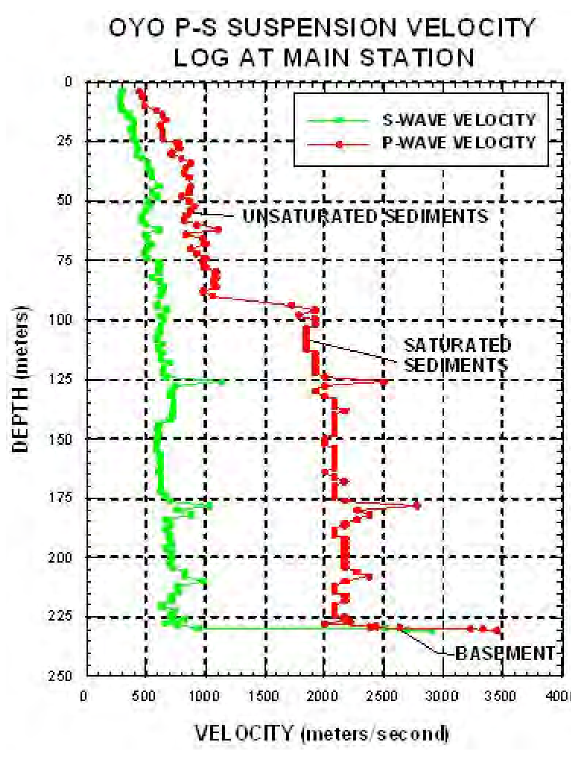






The figures below demonstrate the effectiveness of the quarter-wavelength approximation. At this site near Borrego Springs, California, a PS Suspension log was acquired in a borehole that encountered bedrock at a depth of 229 m. The PS Suspension log indicates that the average S-wave velocity of the sediments overlying bedrock is about 572 m/s. The HVSR peak at this site is 0.65 Hz which, combined with the average velocity of the sediments, indicates that bedrock is about 220 m deep, within 4% of that encountered in the borehole.





HVSR testing can be used for:

- Seismic microzonation studies.
- Confirming that the velocity structure is 1-D beneath large active/passive surface wave arrays.
- Reduce non-uniqueness in S-wave velocity models developed from surface wave testing through joint inversion.
- Estimate relative depth to bedrock for hydrogeologic studies.

APPENDIX C

Figure C-1	Summary of Laboratory Test Results
Figure C-2	HAI Laboratory Test Results Transmittal
Figure C-3	Particle-Size Analysis of Soils
Figure C-4	Particle-Size Analysis of Soils
Figure C-5	Particle-Size Analysis of Soils
Figure C-6	Particle-Size Analysis of Soils
Figure C-7	Particle-Size Analysis of Soils
Figure C-3	Particle-Size Analysis of Soils
Figure C-4	Particle-Size Analysis of Soils
Figure C-5	Particle-Size Analysis of Soils
Figure C-6	Particle-Size Analysis of Soils
Figure C-7	Particle-Size Analysis of Soils
Figure C-8	Particle-Size Analysis of Soils
Figure C-9	Particle-Size Analysis of Soils
Figure C-10	Particle-Size Analysis of Soils
Figure C-11	Particle-Size Analysis of Soils
Figure C-12	Particle-Size Analysis of Soils
Figure C-13	Particle-Size Analysis of Soils
Figure C-14	Particle-Size Analysis of Soils
Figure C-15	Particle-Size Analysis of Soils
Figure C-16	Particle-Size Analysis of Soils
Figure C-17	Particle-Size Analysis of Soils
Figure C-18	Atterberg Limits
Figure C-19	Expansion Index of Soils
Figure C-20	Expansion Index of Soils
Figure C-21	Expansion Index of Soils
Figure C-22	Expansion Index of Soils
Figure C-23	Consolidation Test Results
Figure C-24	Consolidation Test Results
Figure C-25	Consolidation Test Results

Figure C-26	Consolidation Test Results
Figure C-27	Consolidation Test Results
Figure C-28	Consolidation Test Results
Figure C-29	Consolidation Test Results
Figure C-30	Consolidation Test Results
Figure C-31	Consolidation Test Results
Figure C-32	Consolidation Test Results
Figure C-33	Direct Shear Test Results
Figure C-34	Direct Shear Test Results
Figure C-35	Direct Shear Test Results
Figure C-36	Direct Shear Test Results
Figure C-37	Direct Shear Test Results
Figure C-38	Direct Shear Test Results
Figure C-39	Direct Shear Test Results
Figure C-40	Direct Shear Test Results
Figure C-41	Direct Shear Test Results
Figure C-42	Summary of Other Test Results (EI, SO4, Ch, pH & ER; MP•-OMC; and R-Value)
Figure C-43	Corrosivity Series Test Results by Anaheim Test Laboratory
Figure C-44	Corrosivity Series Test Results by M.J. Schiff & Associates
Figure C-45	Laboratory Compaction Test by Modified Effort
Figure C-46	Laboratory Compaction Test by Modified Effort
Figure C-47	Laboratory Compaction Test by Modified Effort
Figure C-48	Resistance R-Value by Anaheim Test Laboratory
Figure C-49	Resistance R-Value by LaBelle Marvin, Inc.

GEOBASE, INC.

SUMMARY OF LABORATORY TEST RESULTS

PROJECT:	MORENO VALLEY MEDICAL CENTER, DIAGNOSTIC AND TREATMENT BUILDING, 27300 IRIS AVENUE, MORENO VALLEY, CALIFORNIA	PROJECT NO:	C.314.81.00	DATE:	August 01, 2017

BORING	DEPTH (feet)	MOISTURE CONTENT (Percent)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-1	5.0-6.5	16					52		48		200 Wash	ML
	10.0-11.5	12										ML
	15.0-16.5	11	138.3									SM
	20.0-21.5	7										SM
	25.0-26.5	11	120.1									SM
	30.0-31.5	5										SM
	35.0-36.5	7	129.7				20		80		200 Wash	SM
	40.0-41.5	8					23		77		200 Wash	SM
	45.0-46.5	7	131.4				21		79		200 Wash	SM
	50.0-51.5	13										SM
	55.0-56.5											SP
	60.0-61.5						27		69	4		SM
	65.0-66.5						17		78	5		SM
B-2	5.0-6.5	8										SM-ML
	5.0-10.0	---									EI=80, pH, S04, ER, Ch, MP, C	Bulk Sample 5.0-10.0 ft.
	10.0-11.5	8	114.2								C, DS	SC
	15.0-16.5	11										SM
	20.0-21.5	7	123.7								C, DS	SM
	25.0-26.5	8										SM
	30.0-31.5	4	119.6									SP-SM
	35.0-36.5	5										SM
	40.0-41.5	9										SM
	45.0-46.5	7										SM
	50.0-51.5	7					30		70		200 Wash	SM
	55.0-56.5											SM
	60.0-61.5											SP

GEOBASE, INC.

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: MORENO VALLEY MEDICAL CENTER, DIAGNOSTIC AND TREATMENT BUILDING, 27300 IRIS AVENUE, MORENO VALLEY, CALIFORNIA

PROJECT NO: C.314.81.00

DATE: August 01, 2017

BORING	DEPTH (feet)	MOISTURE CONTENT (Percent)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-3	5.0-6.5	9										ML
	5.0-10.0	---									EI=10, pH, S04, ER, Ch, MP, C	Bulk Sample 5.0-10.0 ft.
	10.0-11.5	13										SM
	15.0-16.5	13	118.9				57		43		DS, 200 Wash	ML
	20.0-21.5	8										SM
	25.0-26.5	5	119.9									SM
	30.0-31.5	8										SM
	35.0-36.5	4	126.1				16		84		DS, 200 Wash	SM
	40.0-41.5	7										SM
	45.0-46.5	3	123.0									SP
	50.0-51.5	12										SP
	55.0-56.5	15										ML
	60.0-61.5	10										SM
	65.0-66.5	13										SP
	70.0-71.5	10										SP
B-4	5.0-6.5	9										SM
	10.0-11.5	12										ML
	15.0-16.5	11	122.4				30		70		C, DS, 200 Wash	SM
	20.0-21.5	5					17		83		200 Wash	SM
	25.0-26.5	10	126.7									SM
	30.0-31.5	3										SM
	35.0-36.5	10	127.0									SM
	40.0-41.5	8										SM
	45.0-46.5											SM
	50.0-51.5						10		86	3		SP
	55.0-56.5											SM
B-4	60.0-61.5											SP

GEOBASE, INC.
SUMMARY OF LABORATORY TEST RESULTS

PROJECT:	MORENO VALLEY MEDICAL CENTER, DIAGNOSTIC AND TREATMENT BUILDING, 27300 IRIS AVENUE, MORENO VALLEY, CALIFORNIA	PROJECT NO:	C.314.81.00	DATE:	August 01, 2017

BORING	DEPTH (feet)	MOISTURE CONTENT (Percent)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
	65.0-66.5											SP
	70.0-71.5						14		85	1	200 Wash	SM
B-5	5.0-6.5											SM
	10.0-11.5											ML
	15.0-16.5											SM
	20.0-21.5											SM
	25.0-26.5											SM
	30.0-31.5											SM
	35.0-36.5											SM
	40.0-41.5											SM
	45.0-46.5											SM
	50.0-51.5											SP
	55.0-56.5											SM
	60.0-61.5											SP
B-6	5.0-6.5											ML
	10.0-11.5											ML
	15.0-16.5											ML
	20.0-21.5											SM
	25.0-26.5											SM
	30.0-31.5											SM
	35.0-36.5											SM
	40.0-41.5											SM
	45.0-46.5											SM
	50.0-51.5											SM
	55.0-56.5											SW
B-6	60.0-61.5											SW

PROJECT:	MORENO VALLEY MEDICAL CENTER, DIAGNOSTIC AND TREATMENT BUILDING, 27300 IRIS AVENUE, MORENO VALLEY, CALIFORNIA	PROJECT NO:	C.314.81.00	DATE:	August 01, 2017

BORING	DEPTH (feet)	MOISTURE CONTENT (Percent)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
	65.0-66.5											SP
	70.0-71.5											SM
B-7	5.0-6.5	5										ML
	10.0-11.5	5	111.3									ML
	15.0-16.5	11										ML
	20.0-21.5	5	130.3									SM
	25.0-26.5	5										SM
	30.0-31.5	9	125.9									SM
	35.0-36.5	6					22		72	6		SM
	40.0-41.5	8	131.6									SM
	45.0-46.5	12										SP
	50.0-51.5	11	127.6									SM
	55.0-56.5											SP
	60.0-61.5											SM
	65.0-66.5											SP
	70.0-71.5											SP
B-8	0-5.0	---		Non-Plastic			37		63		EI=0, pH, S04, ER, Ch, MP, C, RV	SM
	5.0-6.5	5										ML
	10.0-11.5	12										ML
	15.0-16.5	15	116.3									ML
	20.0-21.5	12										ML
	25.0-26.5	8	123.5									SM
	30.0-31.5	7										SM
	35.0-36.5	11	121.4									SM
	40.0-41.5	13										SM
	45.0-46.5	10	132.0				18		79	3		SM

PROJECT:	MORENO VALLEY MEDICAL CENTER, DIAGNOSTIC AND TREATMENT BUILDING, 27300 IRIS AVENUE, MORENO VALLEY, CALIFORNIA	PROJECT NO:	C.314.81.00	DATE:	August 01, 2017

BORING	DEPTH (feet)	MOISTURE CONTENT (Percent)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
	50.0-51.5											SM
	55.0-56.5											SM
B-8	60.0-61.5											SM
B-9	5.0-6.5	7										ML
	10.0-11.5	8										ML
	15.0-16.5	7					37		63			ML
	20.0-21.5	11	121.0									SM
	25.0-26.5	7										SM
	30.0-31.5	6	122.1									SM
	35.0-36.5											SM
	40.0-41.5											SP
	45.0-46.5											SM
	50.0-51.5											SM
	55.0-56.5											SM
	60.0-61.5						4		80	16		
B-10	0-5.0	---					35		65		EI=4, pH, S04, ER, Ch, RV, 200 Wash	Bulk Sample 0-5.0 ft., SM
	5.0-6.5	10					55		45		200 Wash	ML
	10.0-11.5	11					60		40		200 Wash	ML
	15.0-16.5	6	127.1				32		68		C, DS, 200 Wash	SM
	20.0-21.5	8										SM
	25.0-26.5	9	129.7								C, DS	SM
	30.0-31.5	10					43		58		200 Wash	SM
	35.0-36.5	5	128.7				18		81	2		SM
	40.0-41.5	11					48		48	5		SM
	45.0-46.5	10	130.0									SM
	50.0-51.5	11					21		73	6		SM

GEOBASE, INC.

Figure C-1

SUMMARY OF LABORATORY TEST RESULTS

PROJECT:	MORENO VALLEY MEDICAL CENTER, DIAGNOSTIC AND TREATMENT BUILDING, 27300 IRIS AVENUE, MORENO VALLEY, CALIFORNIA	PROJECT NO:	C.314.81.00	DATE:	August 01, 2017

BORING	DEPTH (feet)	MOISTURE CONTENT (Percent)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-10	55.0-56.5	11					22		74	4		SM
	60.0-61.5	12										SW
B-11	5.0-6.5	---										ML
	10.0-11.5	13	125.8									ML
	15.0-16.5	7					27		74		200 Wash	SM
	20.0-21.5	9	122.2				46		54		200 Wash	SM
	25.0-26.5	8										SM
	30.0-31.5	11	126.0				50		50		200 Wash	SM
	35.0-36.5	6					17		83		200 Wash	SM
	40.0-45.5	4	130.2				11		84	5		SP
	45.0-46.5											SP
	50.0-51.5						20		76	4		SM
	55.0-56.5											SM
	60.0-61.5											SW

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Hushmand Associates, Inc.
1721 E. Lambert Rd, Ste. B
La Habra, CA 90631

p. (562) 690-3737
w. haieng.com
e. hai@haieng.com

July 13, 2017

Geobase, Inc.
23362 Peralta Dr., Unit 4
Laguna Hills. CA 92653

Attention: Mr. Hai Nguyen, P.E.

SUBJECT: Laboratory Test Results
Geobase Project Name: KP Moreno Valley Medical Center
Geobase Project No.: C3148100
HAI Project No.: GBA-17-001

Dear Mr. Nguyen,

Enclosed is the result of the laboratory testing program conducted on samples from the above referenced project. The testing performed for this program was conducted in general accordance with the following test procedure:

Type of Test	Test Procedure
Moisture Content & Dry Density	ASTM D2937
Moisture Content	ASTM D2216
Percentage Passing #200 Sieve	ASTM D1140
Particle Size Analysis (Sieve only)	ASTM D422
Atterberg Limits	ASTM D4318
Expansion Index	ASTM D4829
Modified Proctor Compaction	ASTM D1557
Consolidation	ASTM D2435
Direct Shear (Consolidated & Drained)	ASTM D3080
R-Value	CTM 301

Attached are: forty-one (41) Moisture Content & Dry Density test results; sixty-two (62) Moisture Content test results; twenty-one (21) Percentage passing #200 Sieve test results; fifteen (15) Particle Size Analysis (Sieve only) test results; one (1) Atterberg Limits test result; four (4) Expansion Index test results; three (3) Modified Proctor Compaction test results; ten (10) Consolidation test results; nine (9) 3-point Direct Shear test results; one (1) R-Value test result and three (3) sample remolding.

We appreciate the opportunity to provide our testing services to Geobase, Inc. If you have any questions regarding the test results, please contact us.

Sincerely,

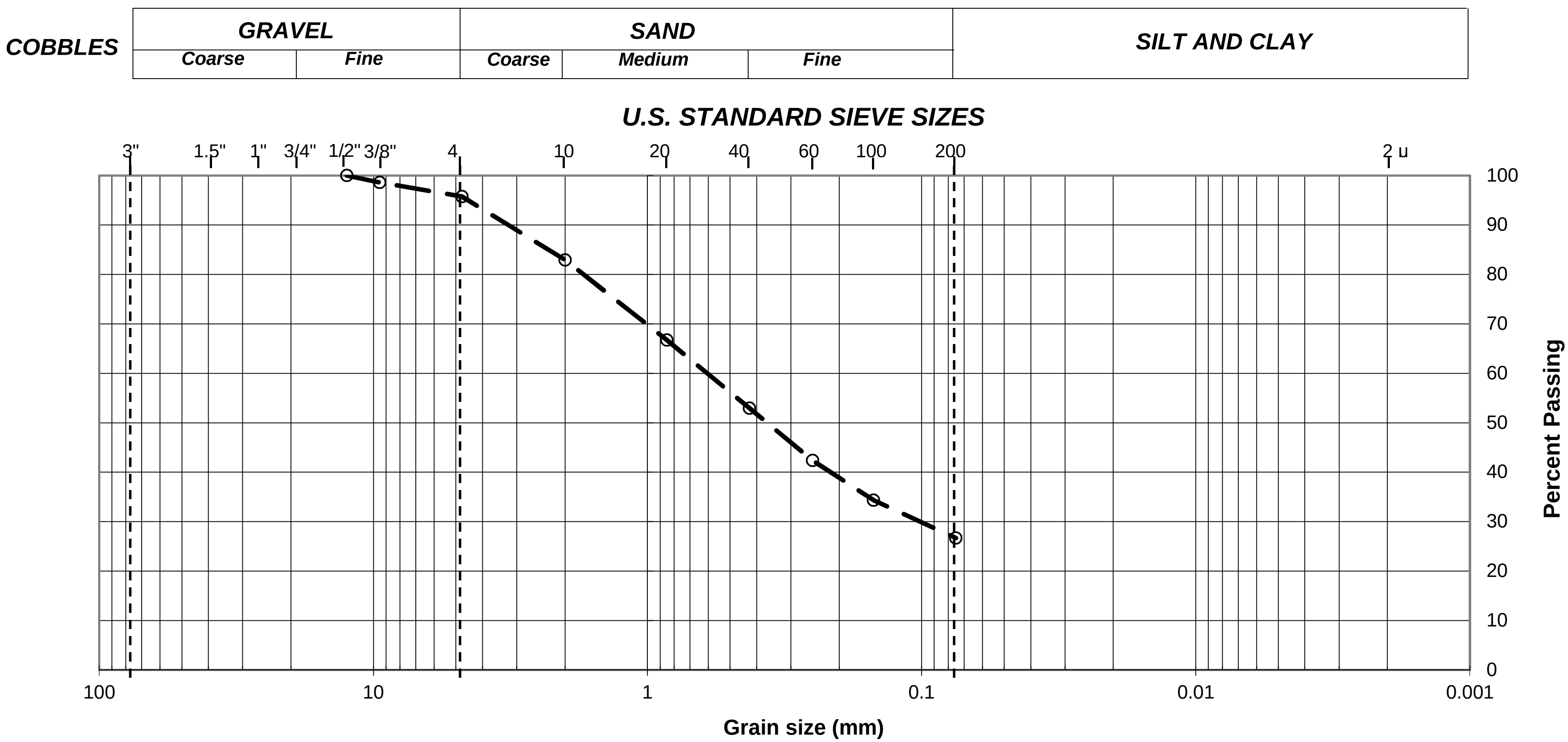
HUSHMAND ASSOCIATES, INC.

Min Zhang, Ph.D., P.E.
Senior Staff Engineer

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

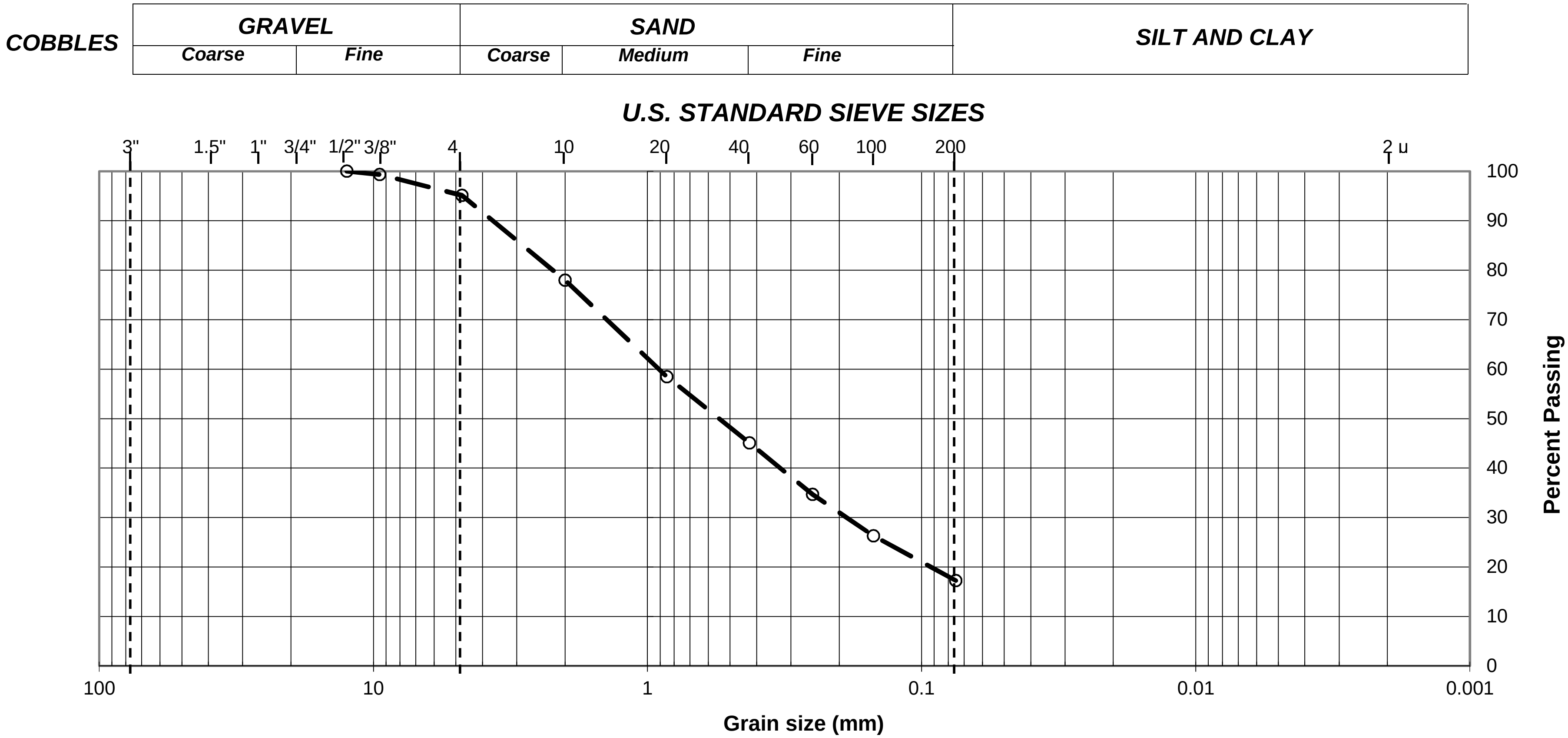


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-1	SPT @ 60-61.5'	○	Brown, Silty Sand (SM)	4.3	69.0	26.7

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

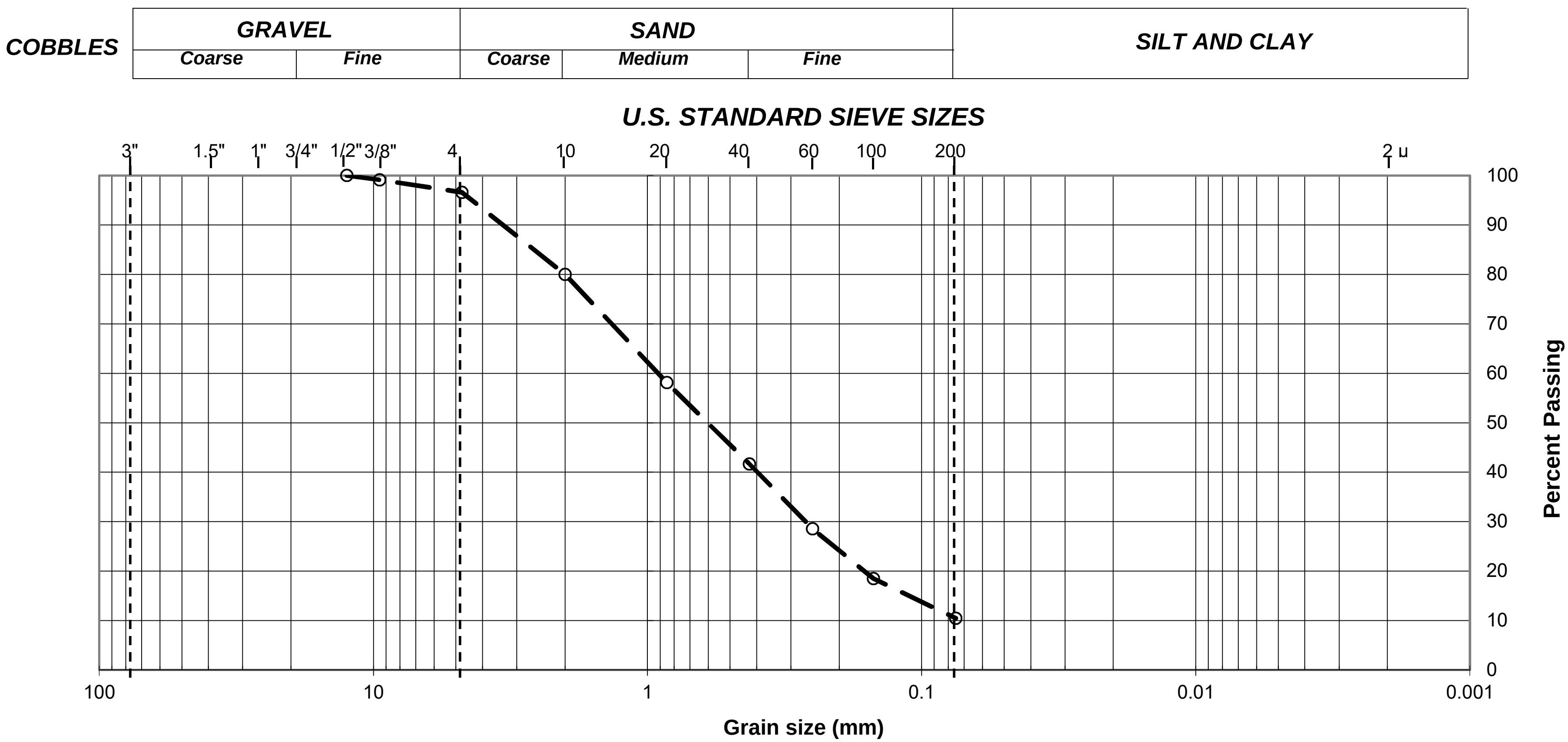


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-1	SPT @ 65-66.5'	○	Brown, Silty Sand (SM)	4.8	77.9	17.3

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

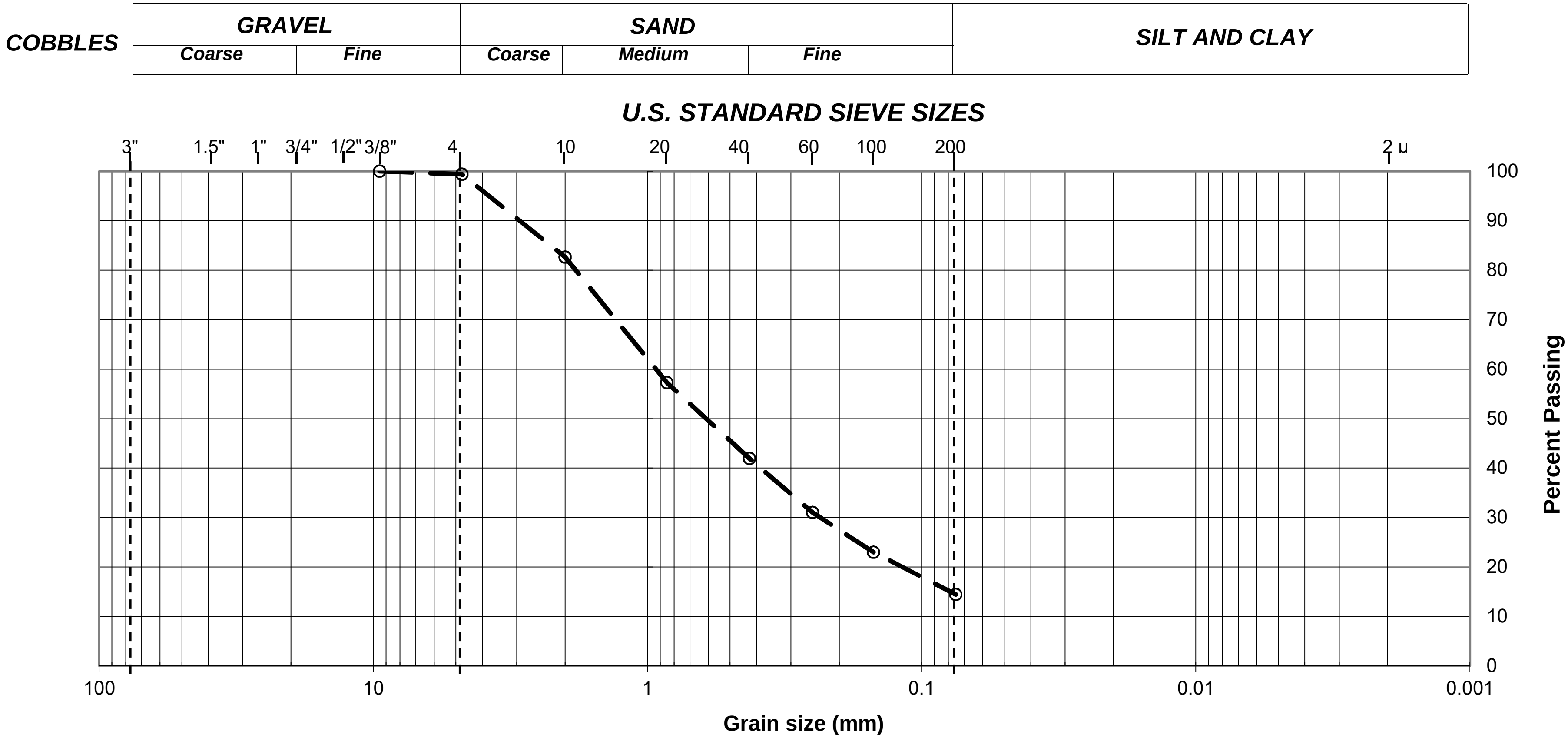


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-4	SPT @ 50-51.5'	○	Brown, Well Graded Sand with Silt (SW-SM)	3.4	86.2	10.4

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project No.: C.314.81.00

HAI Project No.: GBA-17-001
Tested by: MB/KL
Checked by: MZ
Date: 7/7/2017

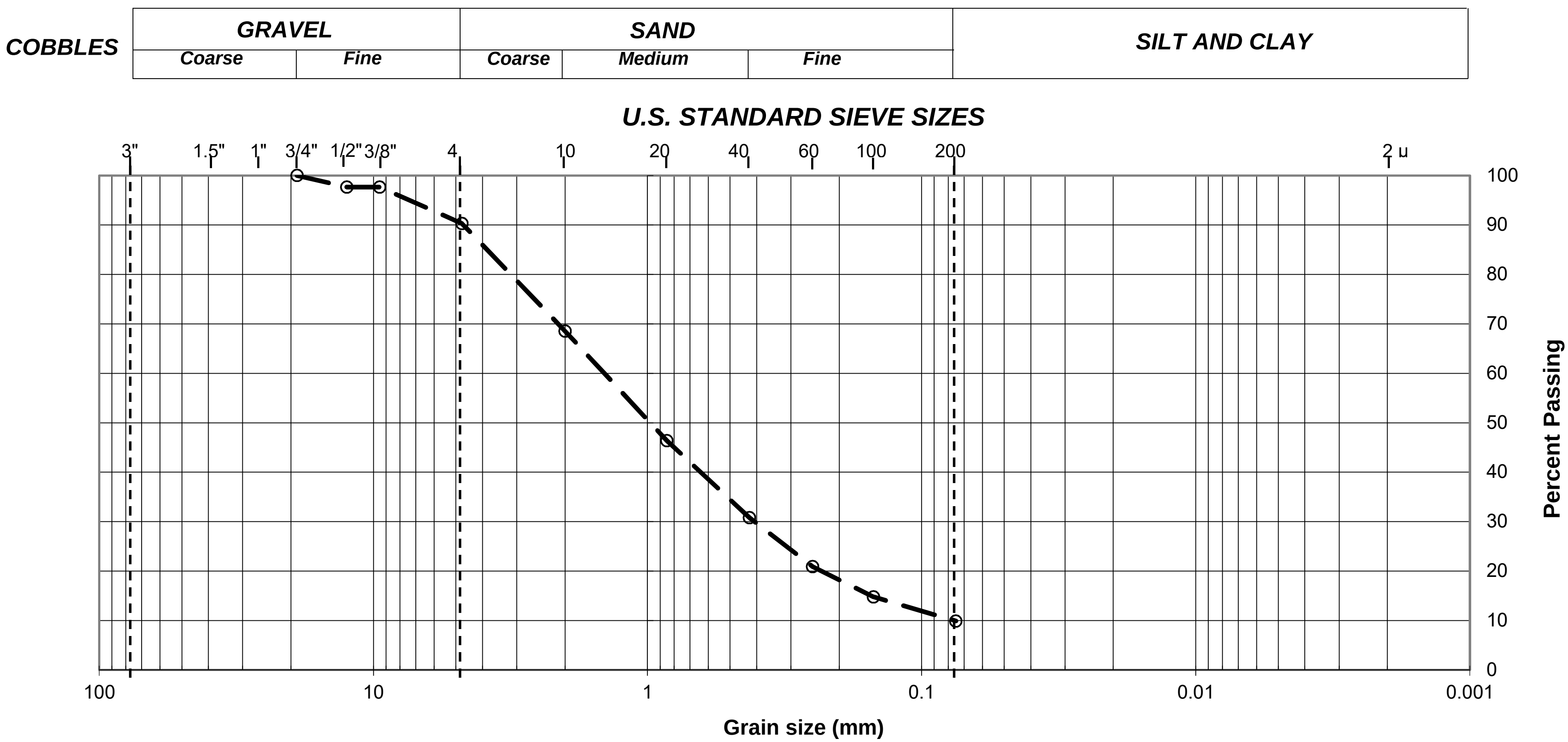


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-4	SPT @ 70-71.5'	○	Brown, Silty Sand (SM)	0.6	84.9	14.4

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

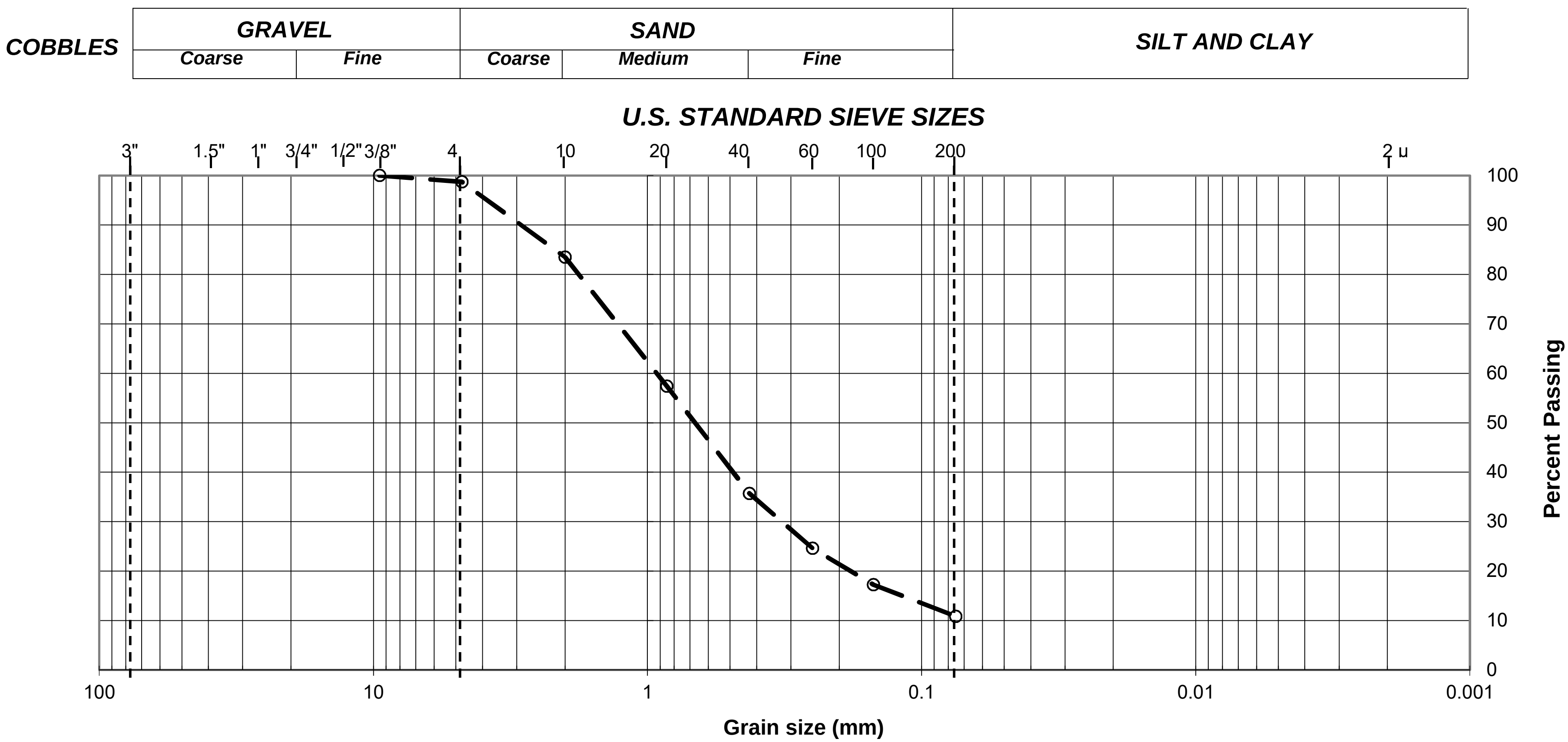


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-6	SPT @ 55-56.5'	○	Brown, Well Graded Sand with Silt (SW-SM)	9.7	80.4	9.9

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

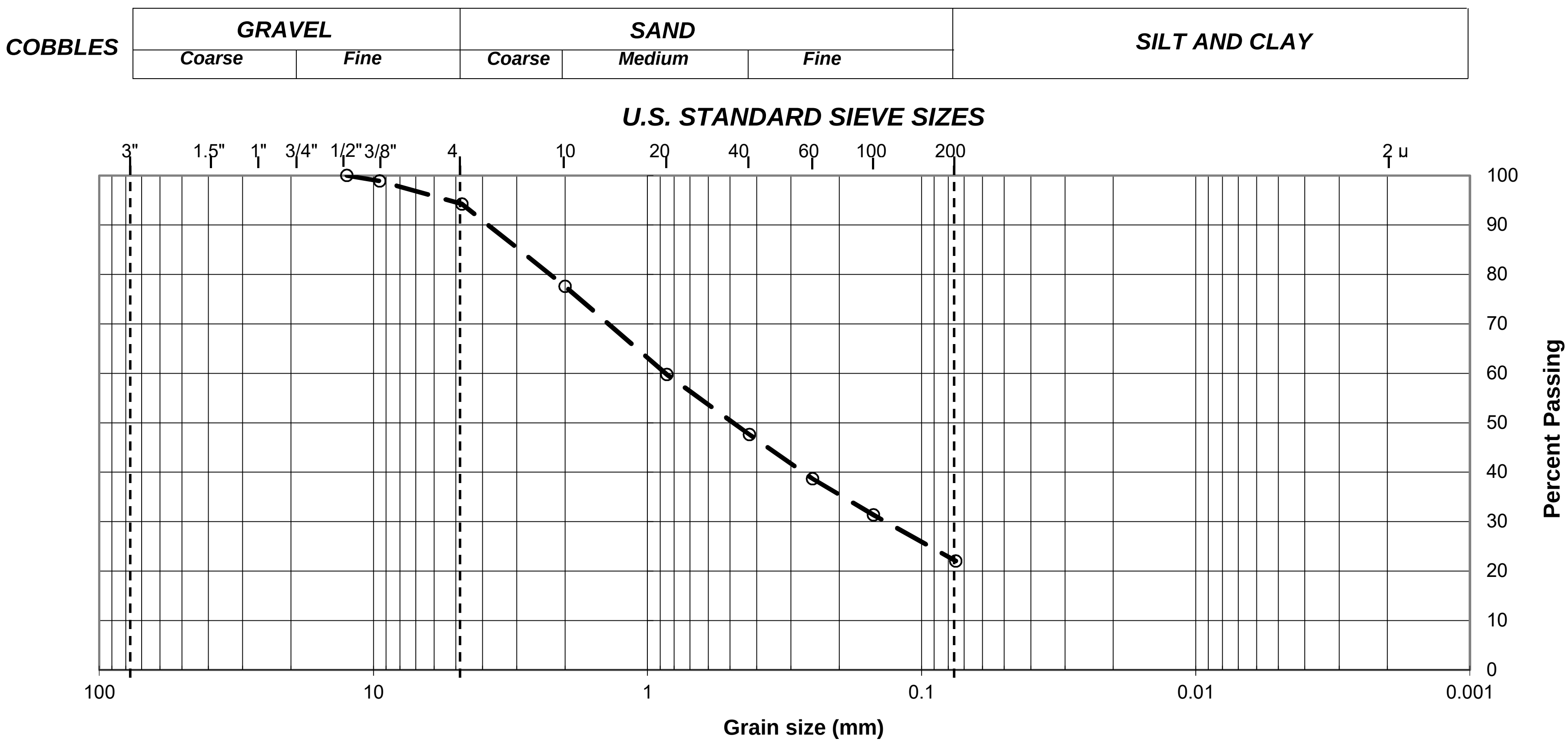


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-6	SPT @ 65-66.5'	○	Brown, Well Graded Sand with Silt (SW-SM)	1.2	87.9	10.8

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

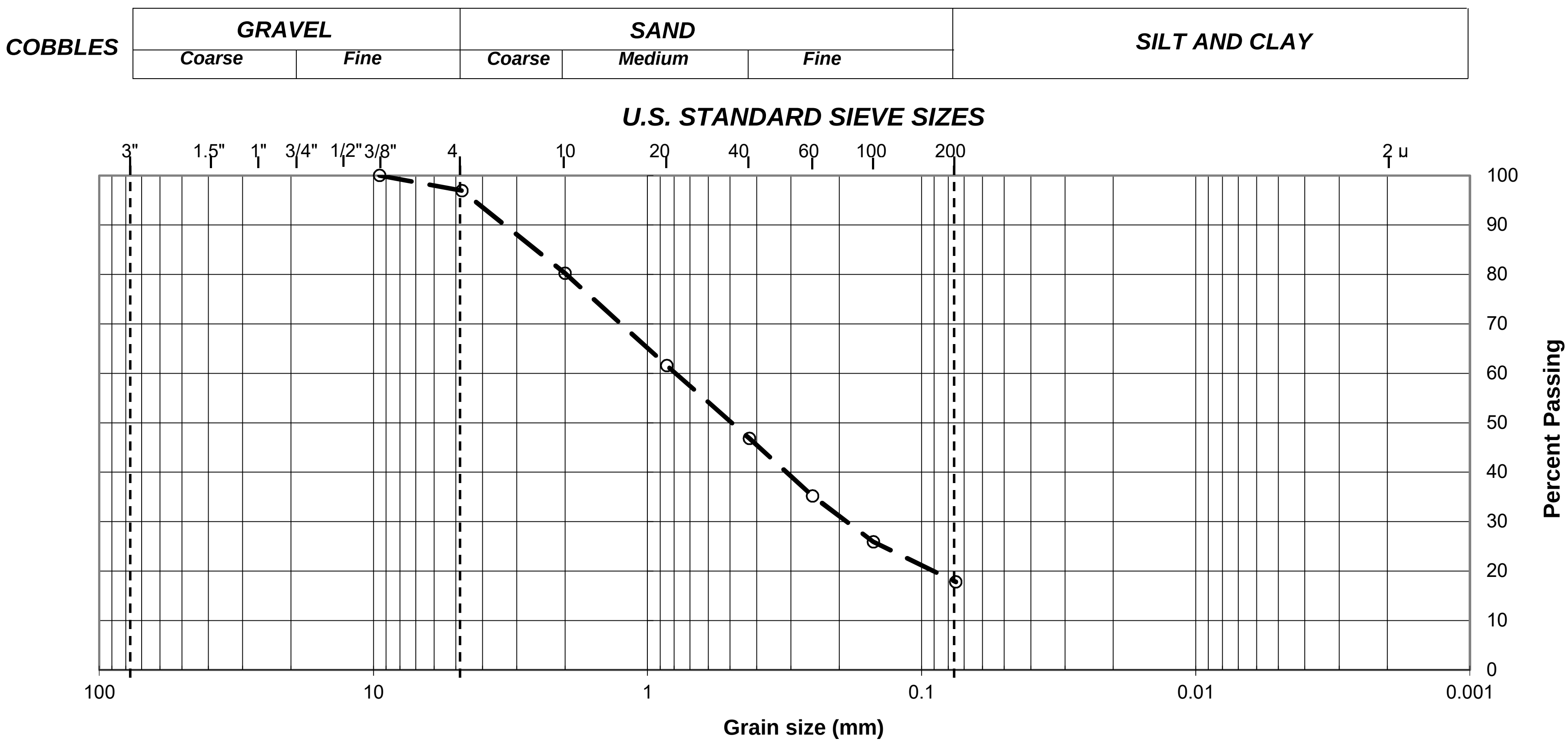


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-7	SPT @ 35-36.5'	○	Brown, Silty Sand (SM)	5.7	72.2	22.0

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

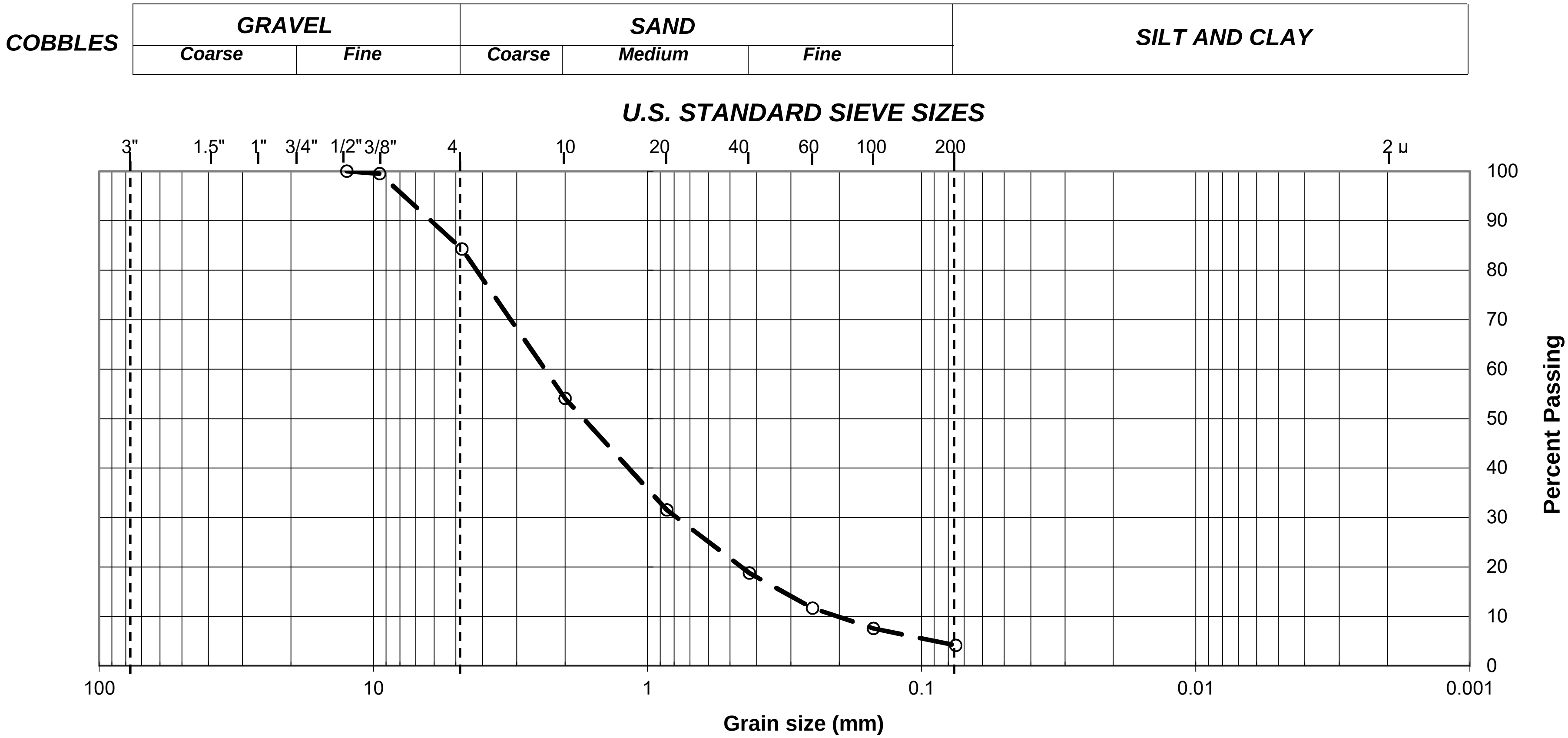


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-8	R @ 45-46.5'	○	Brown, Silty Sand (SM)	3.1	79.2	17.8

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

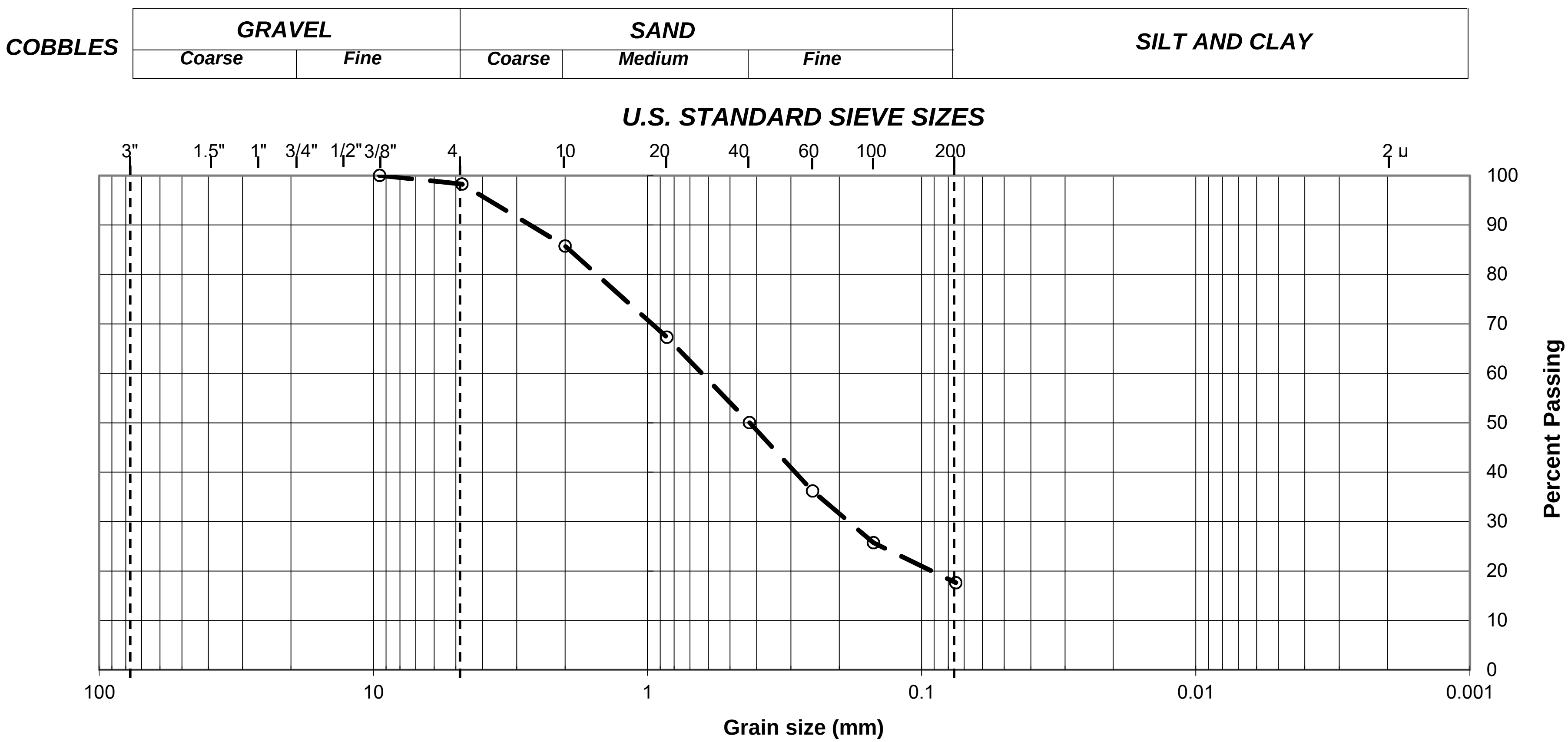


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-9	SPT @ 60-61.5'	○	Brown, Well Graded Sand with Gravel (SW)	15.7	80.1	4.1

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

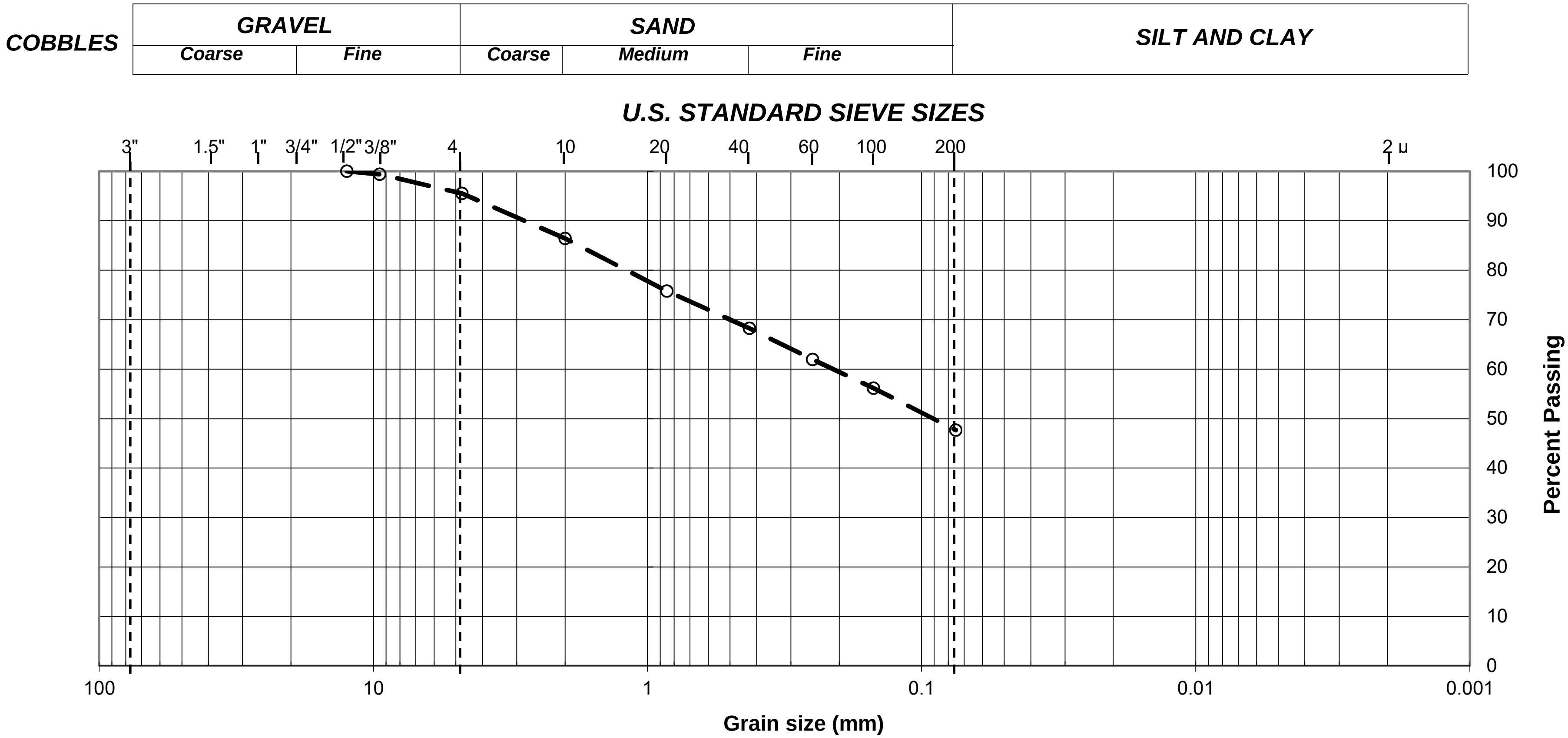


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-10	R @ 35-36.5'	○	Brown, Silty Sand (SM)	1.7	80.6	17.6

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

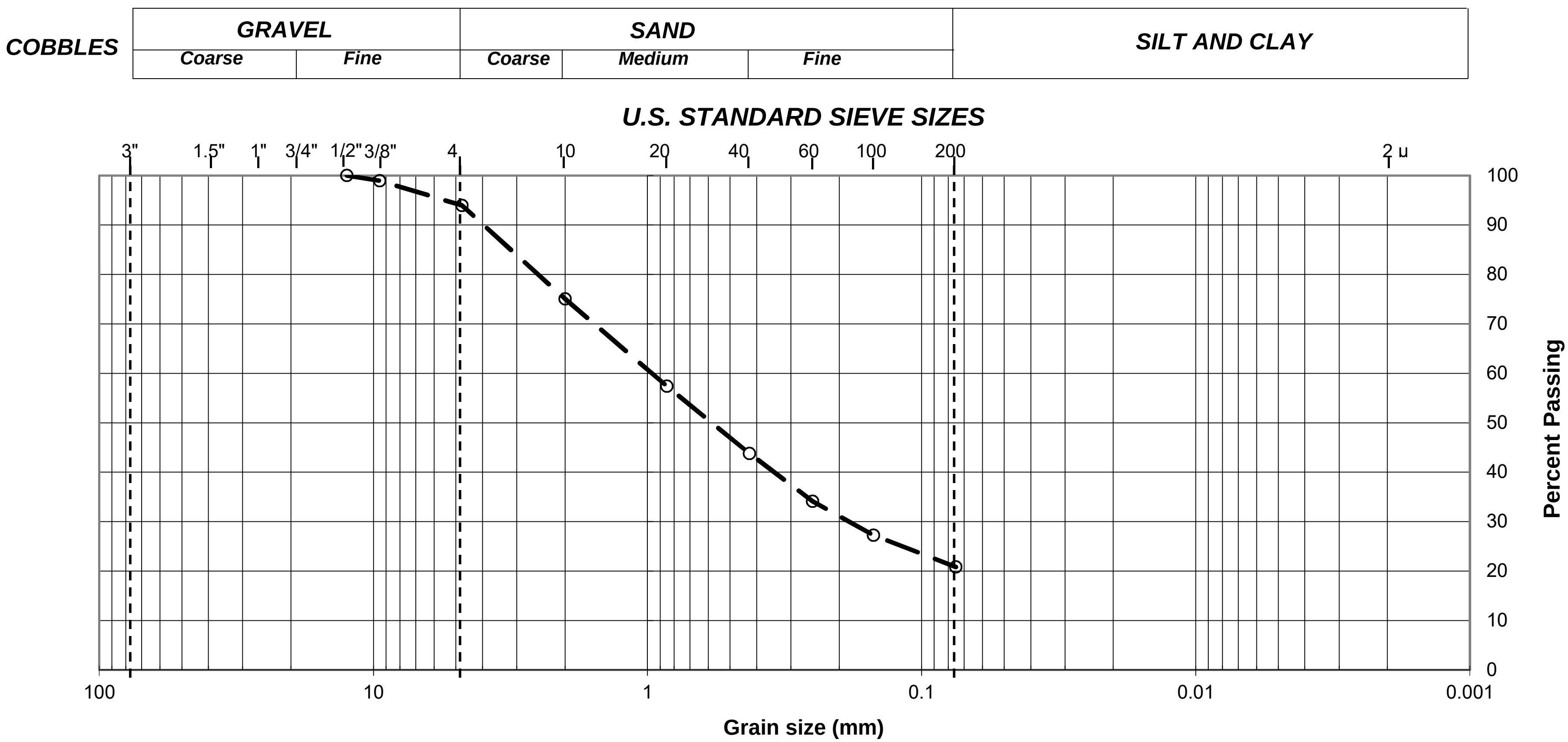


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-10	SPT @ 40-41.5'	○	Brown, Silty Sand (SM)	4.5	47.8	47.7

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

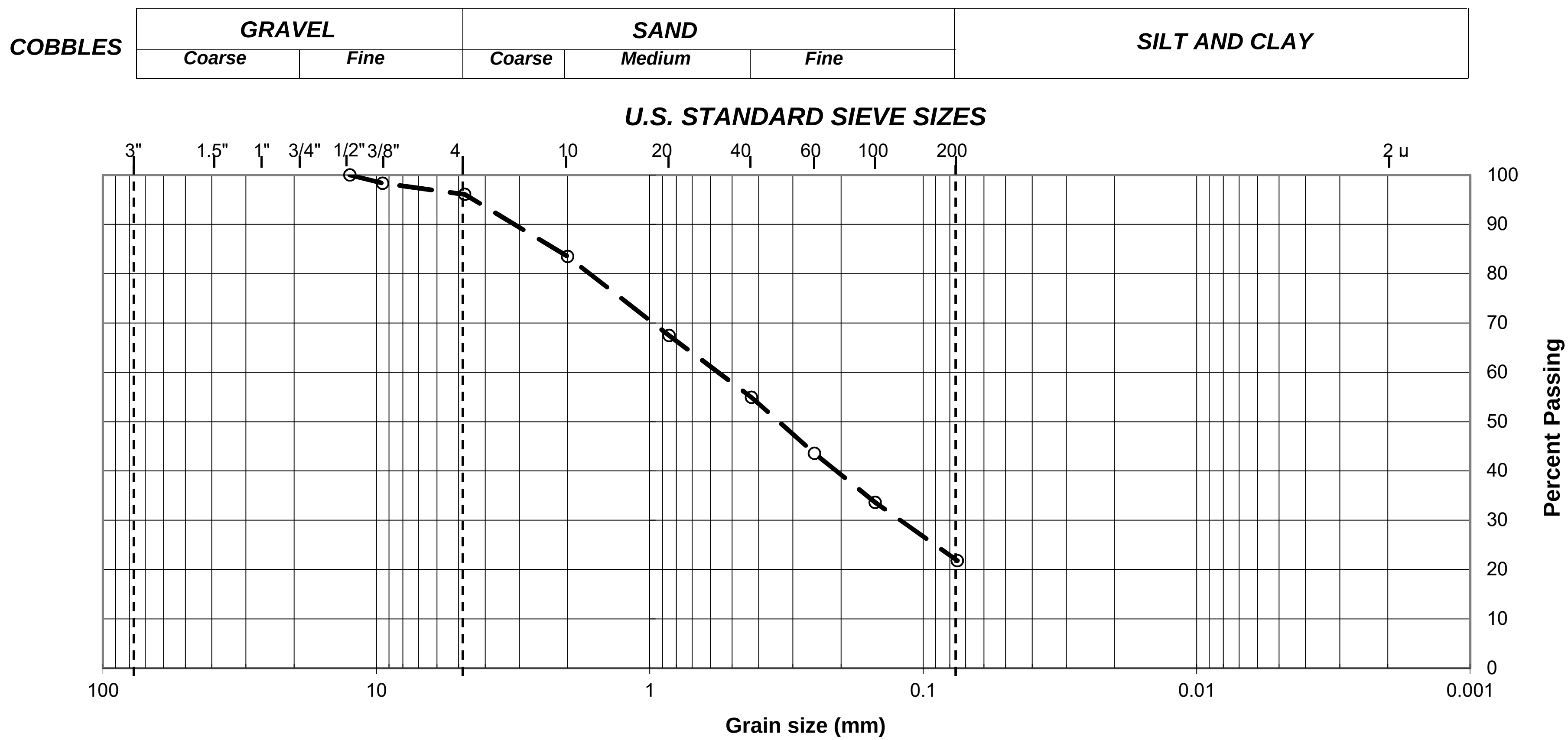


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-10	SPT @ 50-51.5'	○	Brown, Silty Sand (SM)	6.1	73.1	20.8

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

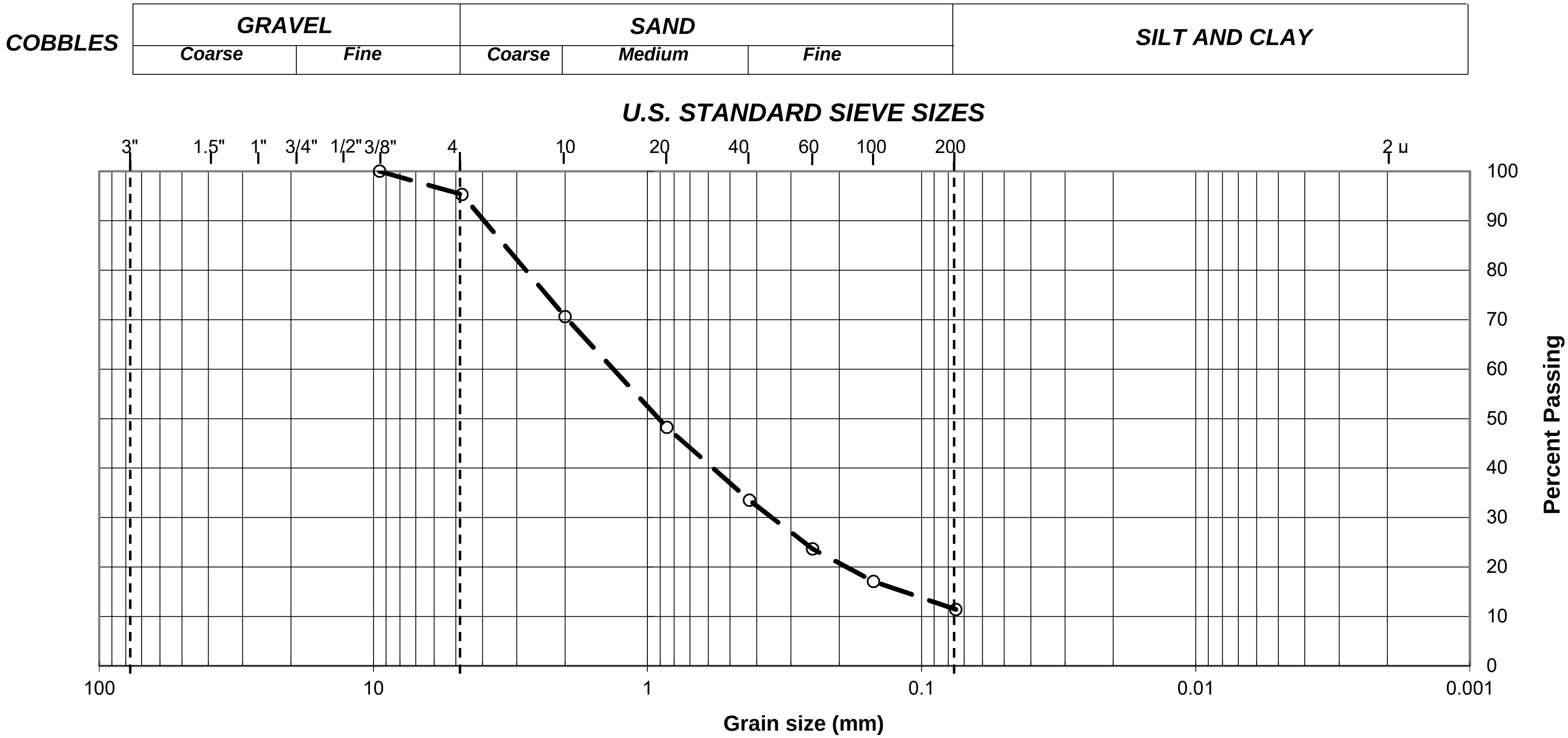
Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017



Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-10	SPT @ 55-56.5'	○	Brown, Silty Sand (SM)	3.9	74.3	21.8

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017

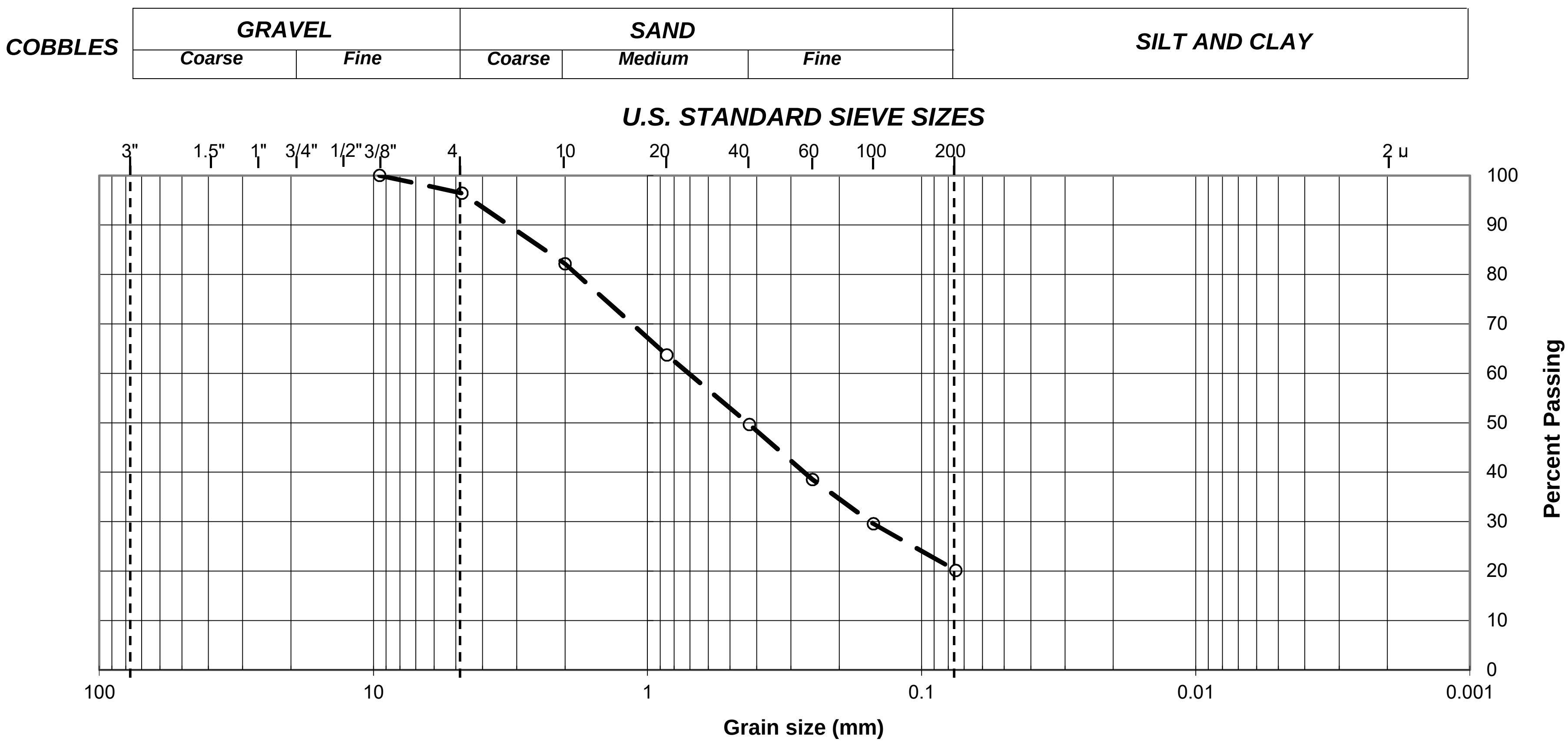


Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-11	R @ 40-41.5'	○	Brown, Well Graded Sand with Silt (SW-SM)	4.7	83.9	11.4

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

PARTICLE-SIZE ANALYSIS OF SOILS
(ASTM D422)

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB/KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017



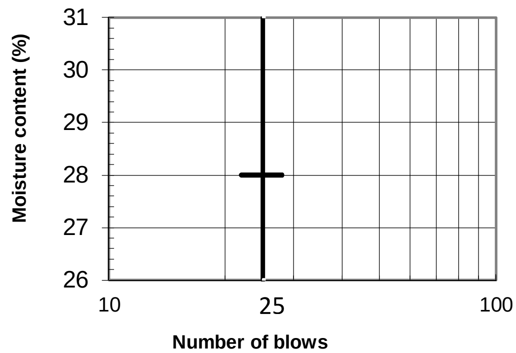
Boring No.	Sample No.	Symbol	USCS	% Gravel	% Sand	% Fines
B-11	R @ 50-51.5'	○	Brown, Silty Sand (SM)	3.6	76.3	20.1

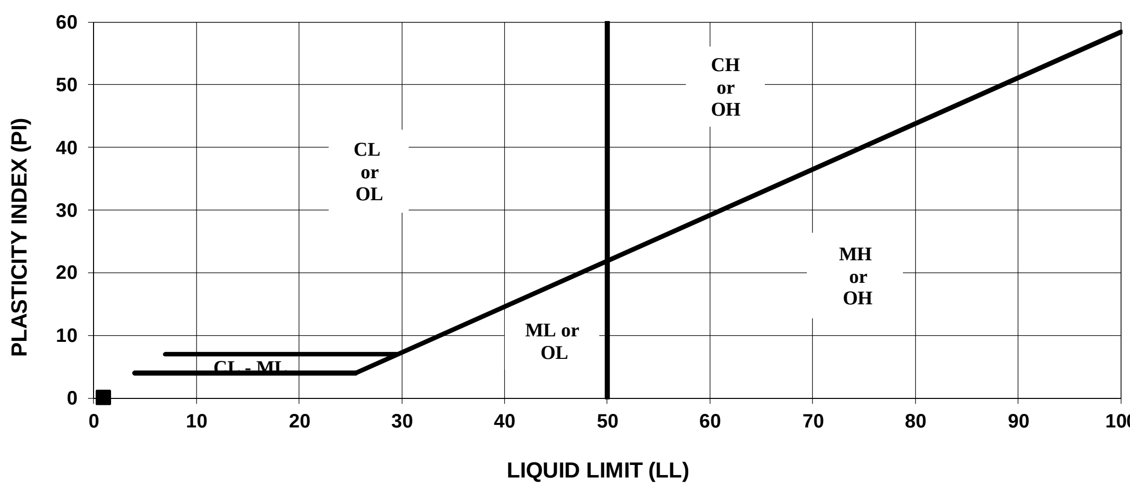
ATTERBERG LIMITS
(ASTM D 4318)

Client:	Geobase, Inc.	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
Boring No.:	B-8	***Date:***	7/7/2017
Sample No.:	B @ 0-5'		
Soil Description:	Brown, Silty Sand (SM)		

Test		LL	LL	LL	PL	PL
Tare No.						
No. of blows						
Wt. of wet soil + tare	(g)					
Wt. of dry soil + tare	(g)					
Wt. of tare	(g)					
Water content	**(%)**					

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
USCS	**SM**





HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

EXPANSION INDEX (ASTM D4829)

Client: Geobase
HAI Project No.: GBA-17-001
Project Name: KP Moreno Valley Medical Center
Tested by: KL
Address: ---
Checked by: MZ
Boring No.: B-2 **Sample No.:** D **Depth:** 5-10' **Date:** 7/7/2017
Soil Description: Tan Brown, Silty Sand (SM)

MOLDED SPECIMEN		
Wt. of wet soil + cont.	**214.90**	g
Wt. of dry soil + cont.	**201.85**	g
Wt. of container	**19.54**	g
Wt. of water	**13.05**	g
Wt. of dry soil	**182.31**	g
Moisture Content	**7.2**	%
Wt. of wet soil + ring	**625.57**	g
Wt. of ring	**197.41**	g
Wt. of wet soil	**428.16**	g
Wet density of soil	**129.7**	pcf
Dry density of soil	**121.1**	pcf
Specific gravity of soil	**2.68**	
Saturation	**50.3**	%

MOISTURE CONTENT AFTER TEST		
Wt. of wet soil + cont.	**656.49**	g
Wt. of dry soil + cont.	**598.90**	g
Wt. of container	**197.41**	g
Wt. of water	**57.59**	g
Wt. of dry soil	**401.49**	g
Moisture Content	**14.3**	%

Date & time	Elapsed time (min)	Dial Reading	Δh, Expansion
6/16/2017 14:15	0	0	
6/16/2017 14:25	10	-0.0042	
Add distilled water to sample			
6/19/2017 14:15	4320	0.0040	0.0082

Expansion Index = 8

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

EXPANSION INDEX (ASTM D4829)

Client:	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Address:	---	***Checked by:***	MZ
Boring No.:	B-3	***Date:***	7/7/2017

Sample No.: D ***Depth:*** 5-10'

Soil Description: Tan Brown, Silty Sand (SM)

MOLDED SPECIMEN		
Wt. of wet soil + cont.	**247.03**	g
Wt. of dry soil + cont.	**230.82**	g
Wt. of container	**24.80**	g
Wt. of water	**16.21**	g
Wt. of dry soil	**206.02**	g
Moisture Content	**7.9**	**%**
Wt. of wet soil + ring	**629.43**	g
Wt. of ring	**206.65**	g
Wt. of wet soil	**422.78**	g
Wet density of soil	**128.1**	pcf
Dry density of soil	**118.8**	pcf
Specific gravity of soil	**2.70**	
Saturation	**50.7**	**%**

MOISTURE CONTENT AFTER TEST		
Wt. of wet soil + cont.	**657.66**	g
Wt. of dry soil + cont.	**602.26**	g
Wt. of container	**206.65**	g
Wt. of water	**55.40**	g
Wt. of dry soil	**395.61**	g
Moisture Content	**14.0**	**%**

Date & time	Elapsed time (min)	Dial Reading	Δh, Expansion
6/16/2017 14:27	0	0	
6/16/2017 14:37	10	-0.0005	
Add distilled water to sample			
6/19/2017 14:27	4320	0.0090	0.0095

Expansion Index = 10

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

EXPANSION INDEX (ASTM D4829)

Client: Geobase
HAI Project No.: GBA-17-001
Project Name: KP Moreno Valley Medical Center
Tested by: KL
Address: ---
Checked by: MZ
Boring No.: B-8 **Sample No.:** D **Depth:** 0-5' **Date:** 7/7/2017
Soil Description: Brown, Silty Sand (SM)

MOLDED SPECIMEN		
Wt. of wet soil + cont.	**241.05**	g
Wt. of dry soil + cont.	**226.65**	g
Wt. of container	**22.05**	g
Wt. of water	**14.40**	g
Wt. of dry soil	**204.60**	g
Moisture Content	**7.0**	%
Wt. of wet soil + ring	**626.11**	g
Wt. of ring	**201.99**	g
Wt. of wet soil	**424.12**	g
Wet density of soil	**128.5**	pcf
Dry density of soil	**120.1**	pcf
Specific gravity of soil	**2.68**	
Saturation	**48.0**	%

MOISTURE CONTENT AFTER TEST		
Wt. of wet soil + cont.	**649.16**	g
Wt. of dry soil + cont.	**595.26**	g
Wt. of container	**201.99**	g
Wt. of water	**53.90**	g
Wt. of dry soil	**393.27**	g
Moisture Content	**13.7**	%

Date & time	Elapsed time (min)	Dial Reading	Δh, Expansion
6/16/2017 15:02	0	0	
6/16/2017 15:12	10	-0.0024	
Add distilled water to sample			
6/19/2017 15:02	4320	-0.0024	0.0000

Expansion Index = 0

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

EXPANSION INDEX (ASTM D4829)

Client:	Geobase			**HAI Project No.:**	GBA-17-001
Project Name:	KP Moreno Valley Medical Center			**Tested by:**	KL
Address:	---			**Checked by:**	MZ
Boring No.:	B-10	**Sample No.:** D	**Depth:** 0-5'	**Date:**	7/7/2017
Soil Description:	Brown, Silty Sand with Few Clay (SM)				

MOLDED SPECIMEN

Wt. of wet soil + cont.	**220.80**	g
Wt. of dry soil + cont.	**209.01**	g
Wt. of container	**25.53**	g
Wt. of water	**11.79**	g
Wt. of dry soil	**183.48**	g
Moisture Content	**6.4**	%
Wt. of wet soil + ring	**647.11**	g
Wt. of ring	**206.64**	g
Wt. of wet soil	**440.47**	g
Wet density of soil	**133.5**	pcf
Dry density of soil	**125.4**	pcf
Specific gravity of soil	**2.68**	
Saturation	**51.6**	%

MOISTURE CONTENT AFTER TEST

Wt. of wet soil + cont.	**659.76**	g
Wt. of dry soil + cont.	**600.74**	g
Wt. of container	**206.64**	g
Wt. of water	**59.02**	g
Wt. of dry soil	**394.10**	g
Moisture Content	**15.0**	%

Date & time	Elapsed time (min)	Dial Reading	Δh, Expansion
6/16/2017 15:02	0	0	
6/16/2017 15:12	10	-0.0038	
Add distilled water to sample			
6/19/2017 15:02	4320	0.0002	0.0040

Expansion Index = 4

CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-2	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	5-10
Type of Sample:	Remolded to 95% Max Density		

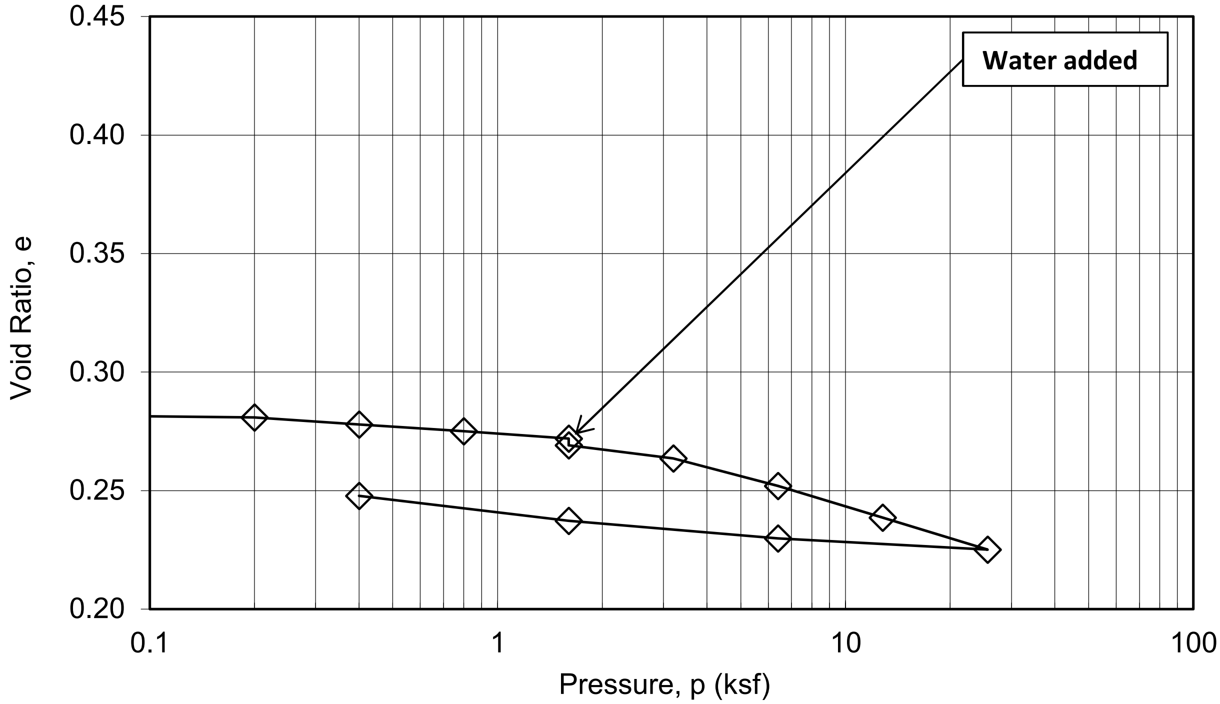
Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
168.07	173.14	157.24

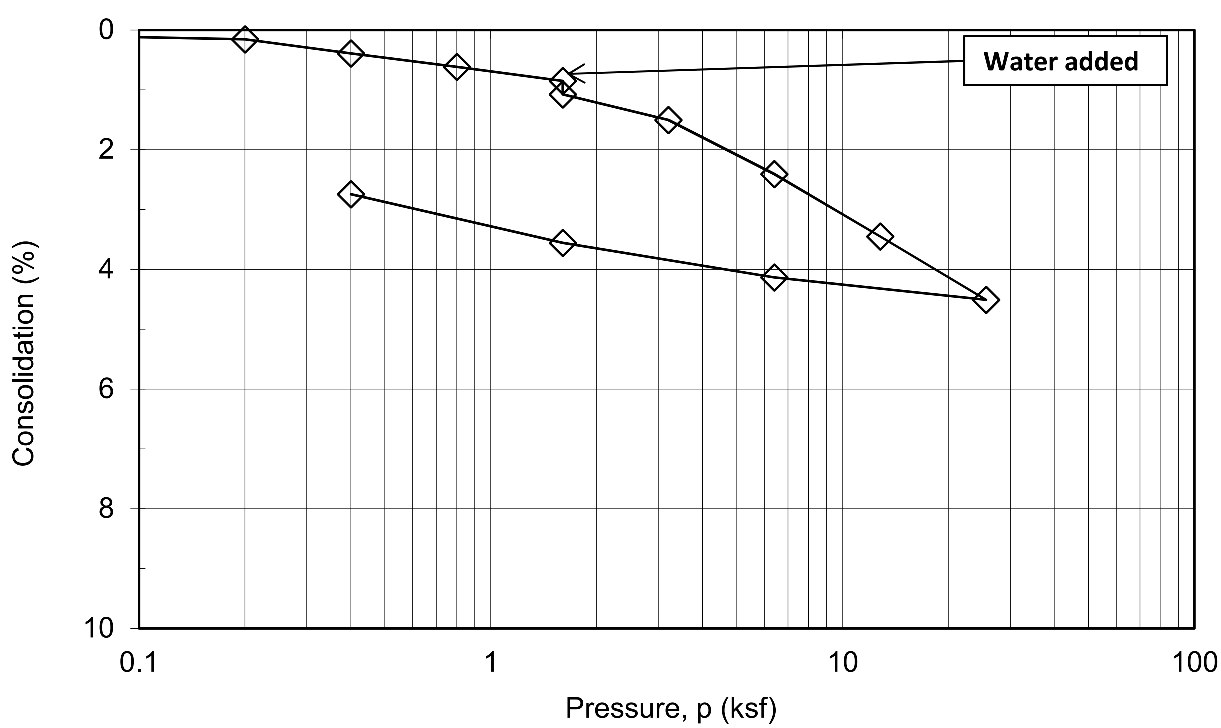
			Initial Conditions	Unload
Height	**H**	**(in)**	1.021	0.9930
Height of Solids	**Hs**	**(in)**	0.796	0.796
Height of Water	**Hw**	**(in)**	0.144	0.212
Height of Air	**Ha**	**(in)**	0.081	0.000
Dry Density		**(pcf)**	**131.3**	**132.3**
Water Content		**(%)**	**6.9**	**10.1**
Saturation		**(%)**	**64.0**	**100.0**

Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0210	0.225	0.283	0.00			
0.2	0.0016	1.0194	0.224	0.281	0.16	1.1E-02	8.3E-03	
0.4	0.0040	1.0170	0.221	0.278	0.39	1.5E-02	1.2E-02	
0.8	0.0063	1.0147	0.219	0.275	0.62	7.2E-03	5.7E-03	
1.6	0.0087	1.0123	0.216	0.272	0.85	3.8E-03	3.0E-03	
1.6	0.0110	1.0100	0.214	0.269	1.08			Water Added
3.2	0.0154	1.0057	0.210	0.264	1.50	3.4E-03	2.7E-03	
6.4	0.0246	0.9964	0.201	0.252	2.41	3.6E-03	2.9E-03	
12.8	0.0353	0.9858	0.190	0.239	3.45	2.1E-03	1.7E-03	
25.6	0.0461	0.9750	0.179	0.225	4.51	1.1E-03	8.7E-04	
6.4	0.0422	0.9788	0.183	0.230	4.13			Unloaded
1.6	0.0363	0.9847	0.189	0.237	3.56			
0.4	0.0280	0.9930	0.197	0.248	2.74			

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-2	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	5-10
Type of Sample:	Remolded to 95% Max Density		





CONSOLIDATION TEST
(ASTM D2435)

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	NB
		Date:	7/7/2017
Boring No.:	B-2	***Sample No. (#):***	27
Soil Description:	SM, BROWN	***Depth (ft):***	10-11.5
Type of Sample:	Undisturbed Ring Sample		

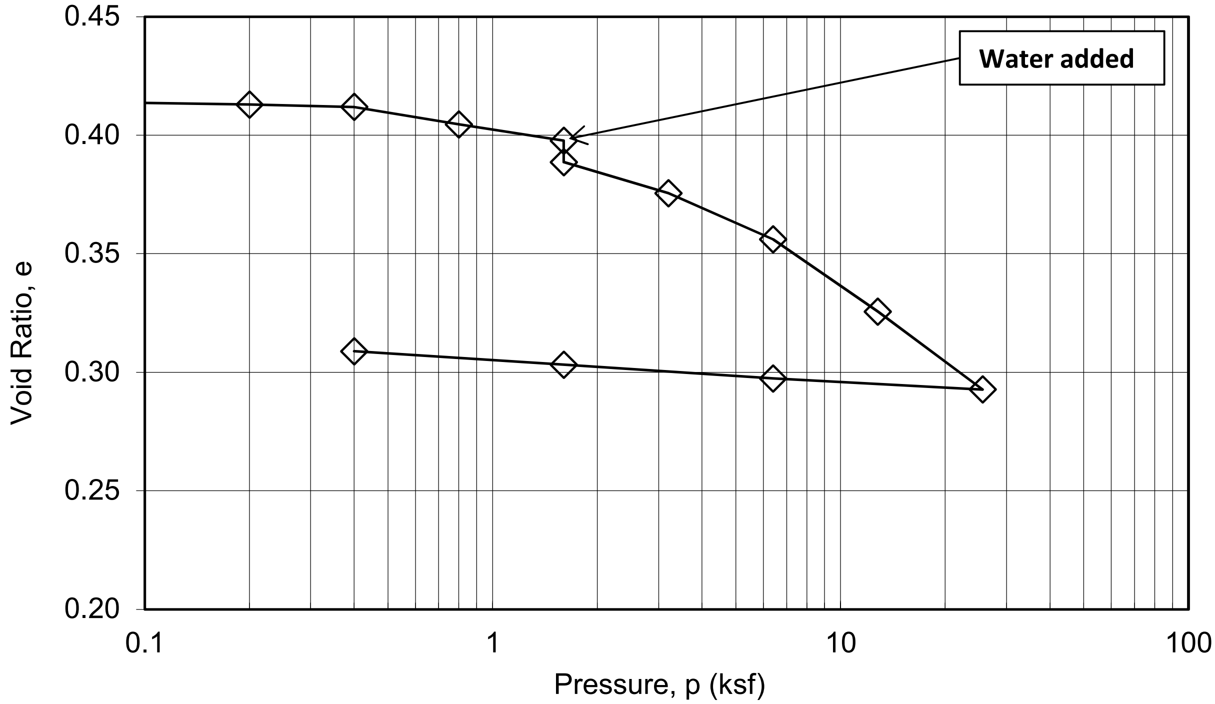
Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
159.32	164.12	148.63

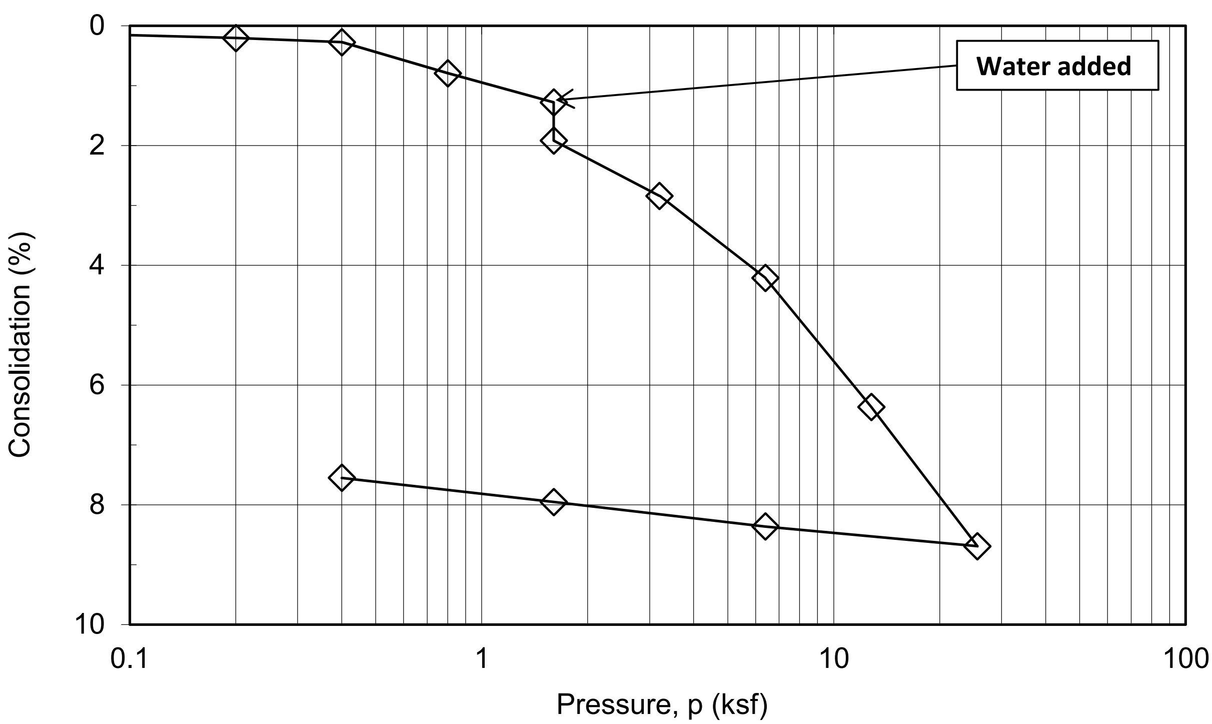
			Initial Conditions	Unload
Height	**H**	**(in)**	1.065	0.9846
Height of Solids	**Hs**	**(in)**	0.752	0.752
Height of Water	**Hw**	**(in)**	0.142	0.206
Height of Air	**Ha**	**(in)**	0.170	0.026
Dry Density		**(pcf)**	**124.1**	**126.1**
Water Content		**(%)**	**7.2**	**10.4**
Saturation		**(%)**	**45.5**	**88.7**

Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0650	0.313	0.416	0.00			
0.2	0.0021	1.0629	0.311	0.413	0.20	1.5E-02	1.1E-02	
0.4	0.0029	1.0621	0.310	0.412	0.27	5.0E-03	3.5E-03	
0.8	0.0084	1.0566	0.304	0.405	0.79	1.8E-02	1.3E-02	
1.6	0.0136	1.0514	0.299	0.398	1.28	8.6E-03	6.1E-03	
1.6	0.0204	1.0446	0.292	0.389	1.92			Water Added
3.2	0.0303	1.0348	0.282	0.376	2.84	8.2E-03	5.9E-03	
6.4	0.0449	1.0202	0.268	0.356	4.21	6.1E-03	4.5E-03	
12.8	0.0678	0.9972	0.245	0.326	6.37	4.8E-03	3.6E-03	
25.6	0.0926	0.9725	0.220	0.293	8.69	2.6E-03	2.0E-03	
6.4	0.0891	0.9760	0.224	0.297	8.36			Unloaded
1.6	0.0847	0.9803	0.228	0.303	7.95			
0.4	0.0804	0.9846	0.232	0.309	7.55			

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-2	***Sample No. (#):***	27
Soil Description:	SM, BROWN	***Depth (ft):***	10-11.5
Type of Sample:	Undisturbed Ring Sample		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-2	***Sample No.:***	-
Soil Description:	SM, BROWN	***Depth (ft):***	20-21.5
Type of Sample:	Undisturbed ring		

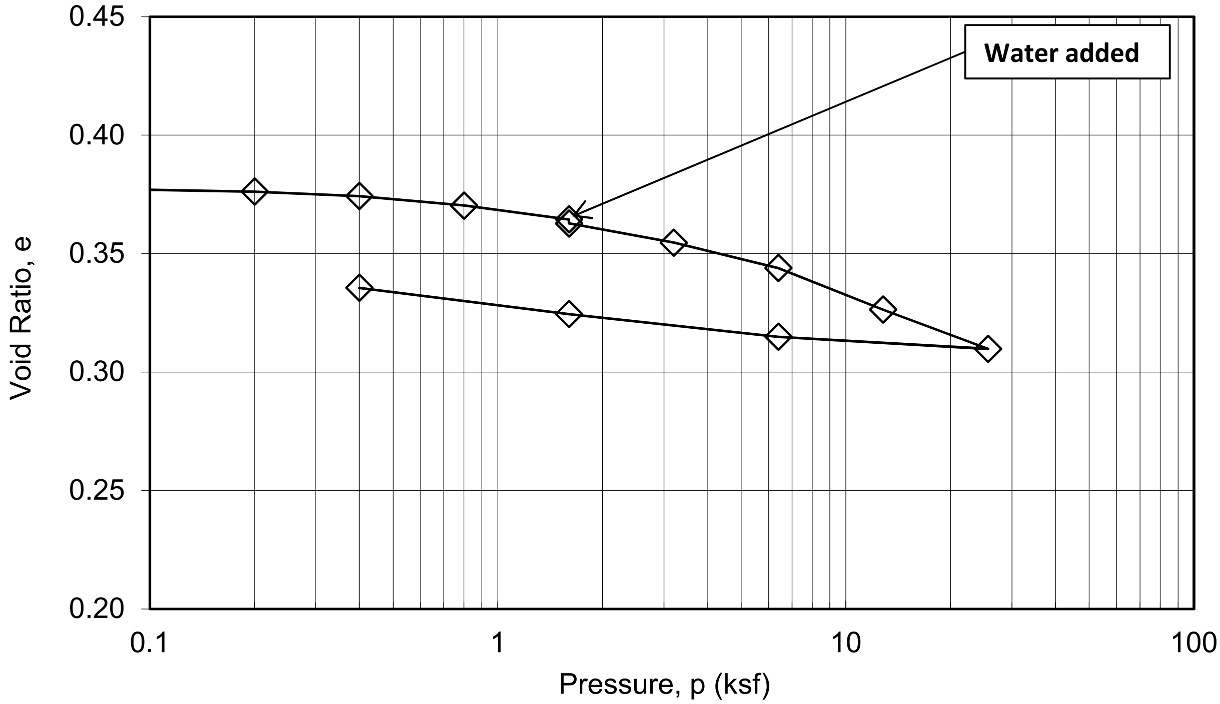
Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
160.00	163.81	146.37

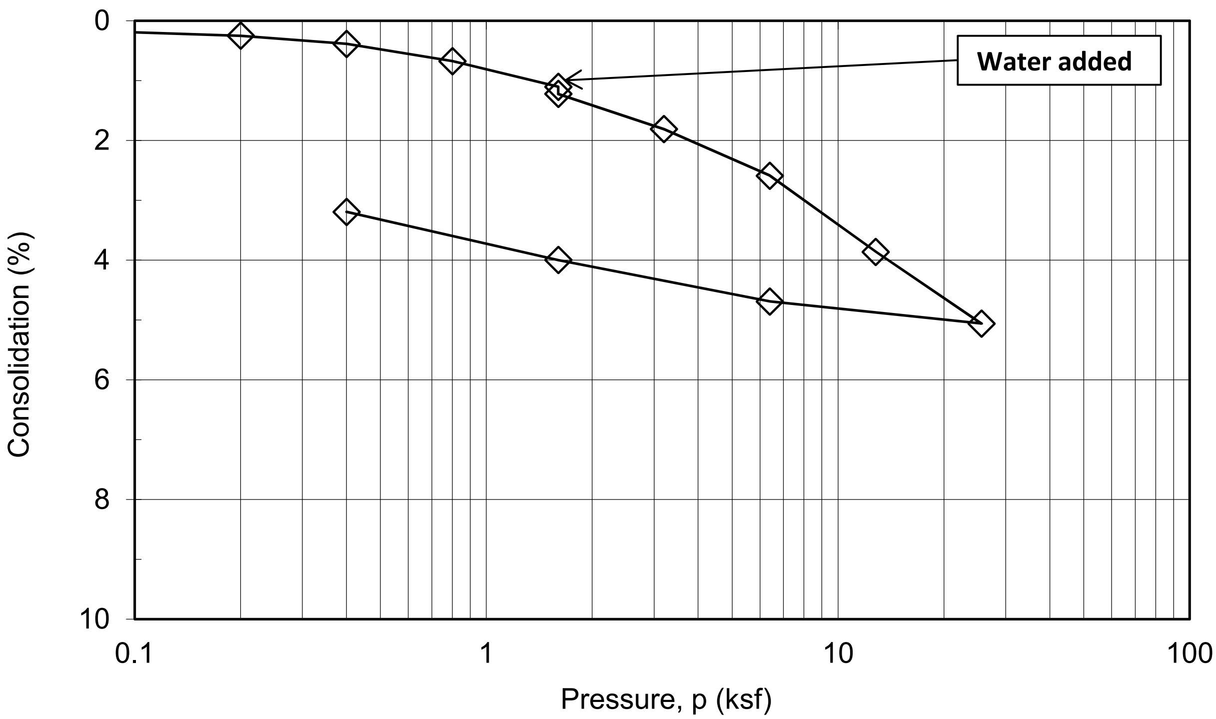
			Initial Conditions	Unload
Height	**H**	**(in)**	1.022	0.9894
Height of Solids	**Hs**	**(in)**	0.741	0.741
Height of Water	**Hw**	**(in)**	0.181	0.232
Height of Air	**Ha**	**(in)**	0.100	0.016
Dry Density		**(pcf)**	**122.2**	**123.6**
Water Content		**(%)**	**9.3**	**11.9**
Saturation		**(%)**	**64.5**	**93.4**

Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0220	0.281	0.380	0.00			
0.2	0.0025	1.0195	0.279	0.376	0.25	1.8E-02	1.3E-02	
0.4	0.0039	1.0181	0.277	0.374	0.39	9.4E-03	6.9E-03	
0.8	0.0069	1.0151	0.274	0.370	0.68	1.0E-02	7.3E-03	
1.6	0.0113	1.0108	0.270	0.364	1.10	7.3E-03	5.4E-03	
1.6	0.0125	1.0095	0.269	0.363	1.22			Water Added
3.2	0.0185	1.0035	0.263	0.355	1.81	5.1E-03	3.7E-03	
6.4	0.0265	0.9956	0.255	0.344	2.59	3.4E-03	2.5E-03	
12.8	0.0395	0.9826	0.242	0.326	3.86	2.7E-03	2.1E-03	
25.6	0.0517	0.9703	0.229	0.310	5.06	1.3E-03	9.9E-04	
6.4	0.0480	0.9741	0.233	0.315	4.69			Unloaded
1.6	0.0409	0.9812	0.240	0.324	4.00			
0.4	0.0327	0.9894	0.249	0.335	3.19			

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-2	***Sample No.:***	-
Soil Description:	SM, BROWN	***Depth (ft):***	20-21.5
Type of Sample:	Undisturbed ring		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client : Geobase
Project Name: KP Moreno Valley Medical Center
Project No.: C.314.81.00
HAI Project No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/7/2017

Boring No.: B-3
Soil Description: SM, BROWN
Type of Sample: Remolde 95% of Maximum Dry Density
Sample No.: -
Depth (ft): 0-5

Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
168.63	175.31	157.41

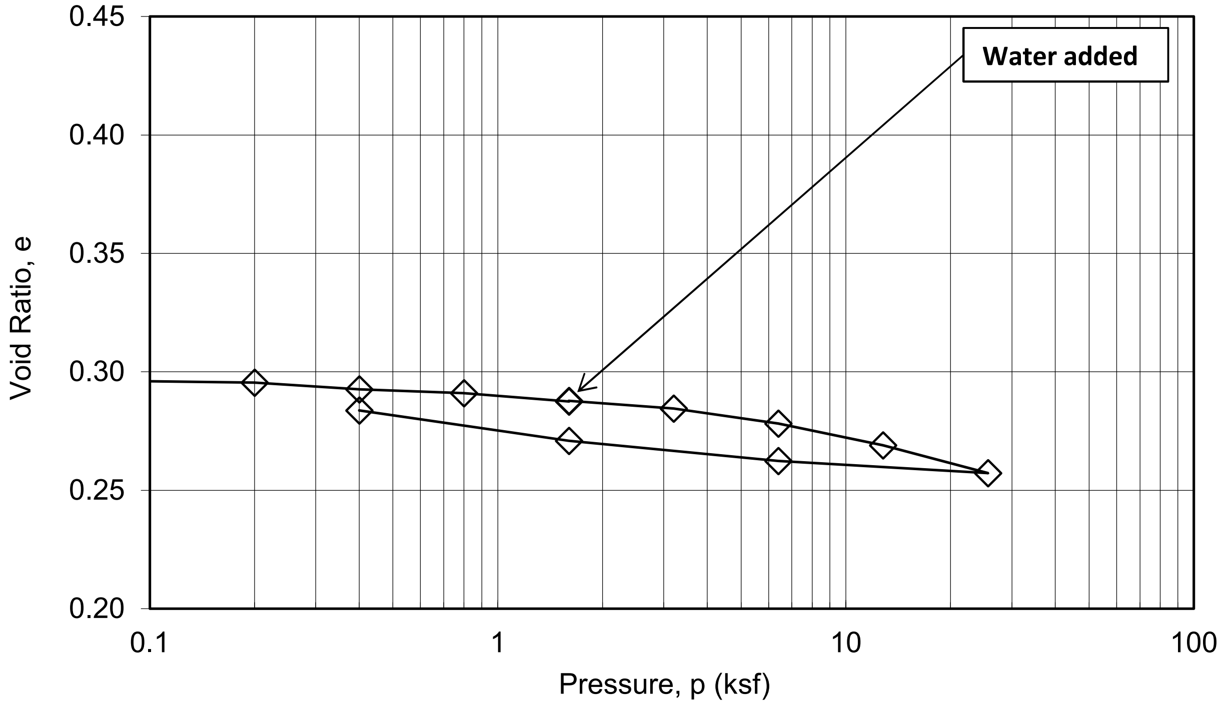
			Initial Conditions	Unload
Height	**H**	**(in)**	1.034	1.0228
Height of Solids	**Hs**	**(in)**	0.797	0.797
Height of Water	**Hw**	**(in)**	0.149	0.238
Height of Air	**Ha**	**(in)**	0.088	0.000
Dry Density		**(pcf)**	**131.5**	**128.5**
Water Content		**(%)**	**7.1**	**11.4**
Saturation		**(%)**	**62.9**	**100.0**

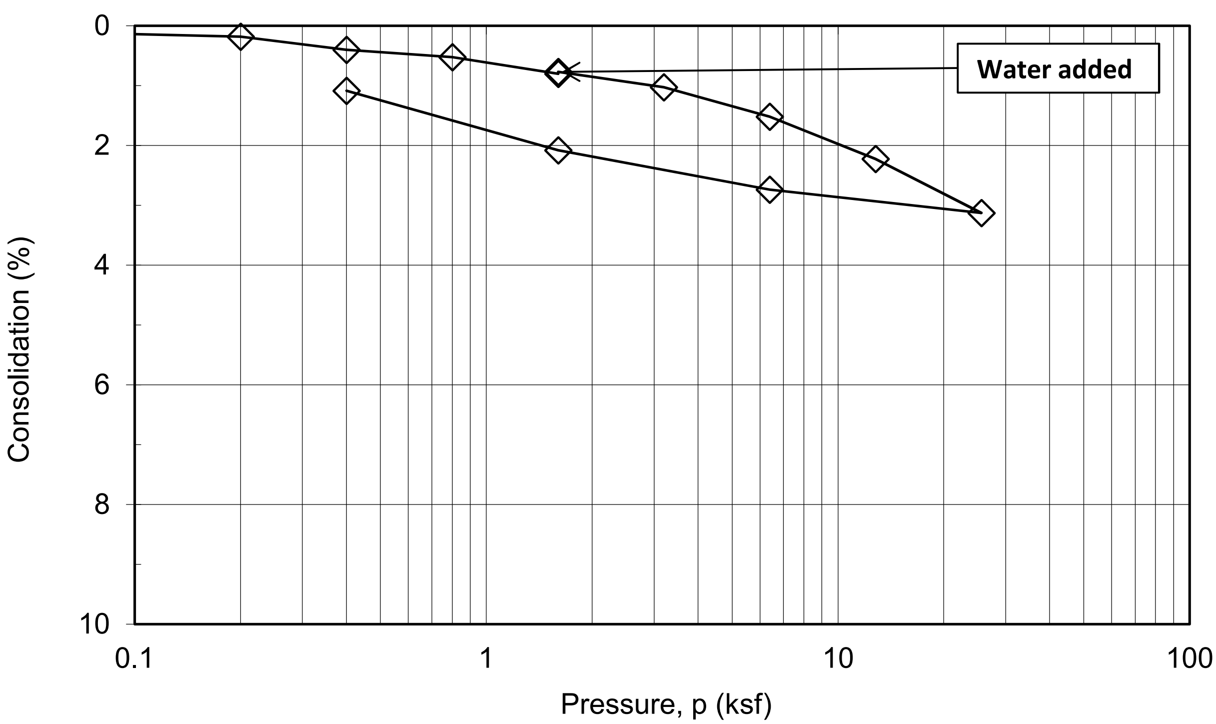
Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0340	0.237	0.298	0.00			
0.2	0.0019	1.0321	0.235	0.295	0.18	1.3E-02	9.7E-03	
0.4	0.0041	1.0299	0.233	0.293	0.40	1.4E-02	1.1E-02	
0.8	0.0054	1.0286	0.232	0.291	0.53	4.1E-03	3.2E-03	
1.6	0.0083	1.0257	0.229	0.287	0.80	4.5E-03	3.5E-03	
1.6	0.0080	1.0260	0.229	0.288	0.77			Water Added
3.2	0.0107	1.0234	0.227	0.284	1.03	2.1E-03	1.6E-03	
6.4	0.0157	1.0183	0.222	0.278	1.52	2.0E-03	1.5E-03	
12.8	0.0230	1.0110	0.214	0.269	2.22	1.4E-03	1.1E-03	
25.6	0.0324	1.0017	0.205	0.257	3.13	9.2E-04	7.3E-04	
6.4	0.0283	1.0057	0.209	0.262	2.74			Unloaded
1.6	0.0215	1.0125	0.216	0.271	2.08			
0.4	0.0113	1.0228	0.226	0.284	1.09			

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-3	***Sample No.:***	-
Soil Description:	SM, BROWN	***Depth (ft):***	0-5
Type of Sample:	Remolde 95% of Maximum Dry Density		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-4	***Sample No.:***	-
Soil Description:	SM, BROWN	***Depth (ft):***	15-16.5
Type of Sample:	Undisurbed Ring		

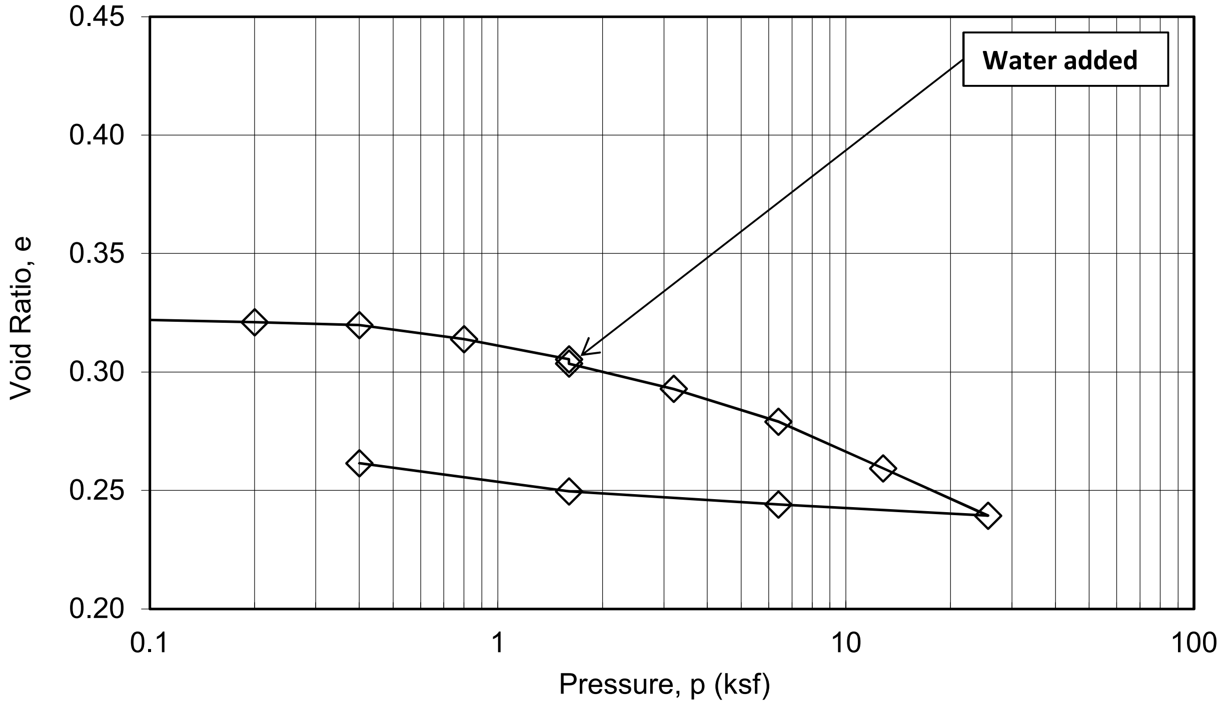
Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
169.29	170.11	154.47

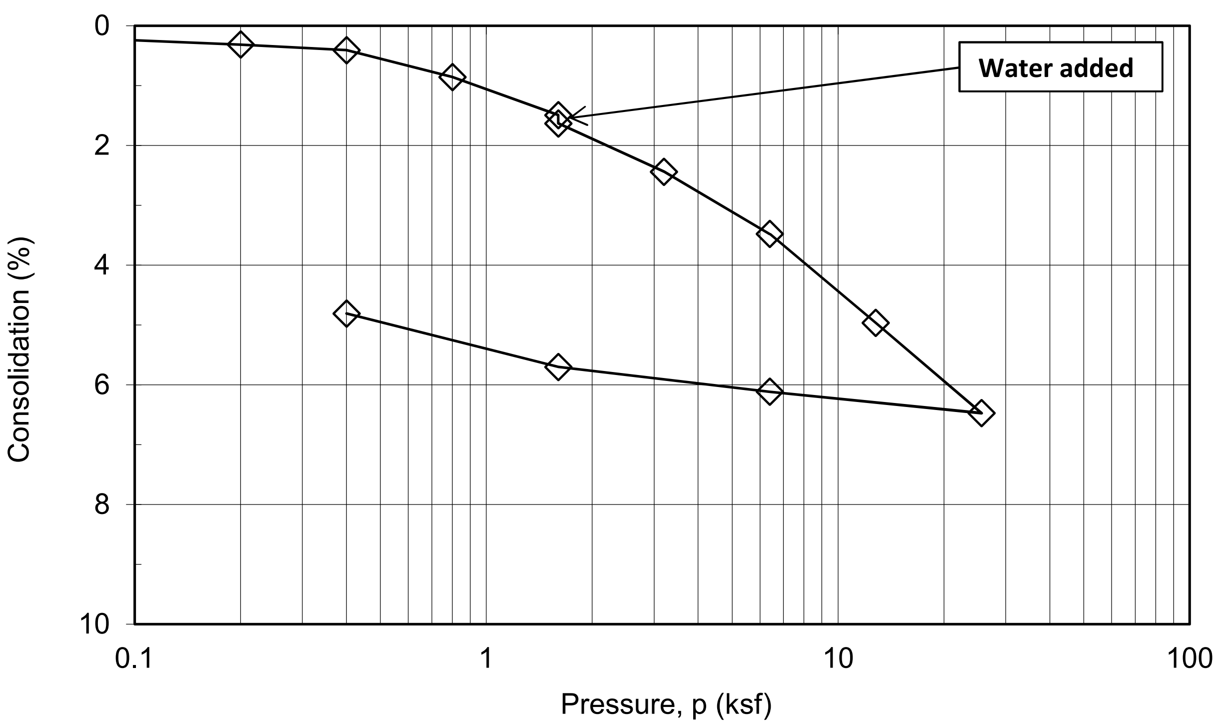
			Initial Conditions	Unload
Height	**H**	**(in)**	1.036	0.9862
Height of Solids	**Hs**	**(in)**	0.782	0.782
Height of Water	**Hw**	**(in)**	0.197	0.208
Height of Air	**Ha**	**(in)**	0.057	0.000
Dry Density		**(pcf)**	**129.0**	**130.8**
Water Content		**(%)**	**9.6**	**10.1**
Saturation		**(%)**	**77.6**	**100.0**

Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0360	0.254	0.325	0.00			
0.2	0.0032	1.0328	0.251	0.321	0.31	2.2E-02	1.7E-02	
0.4	0.0042	1.0318	0.250	0.320	0.41	6.1E-03	4.6E-03	
0.8	0.0089	1.0272	0.245	0.314	0.85	1.5E-02	1.1E-02	
1.6	0.0155	1.0206	0.239	0.305	1.49	1.1E-02	8.1E-03	
1.6	0.0169	1.0191	0.237	0.304	1.63			Water Added
3.2	0.0253	1.0108	0.229	0.293	2.44	6.7E-03	5.2E-03	
6.4	0.0361	1.0000	0.218	0.279	3.48	4.3E-03	3.4E-03	
12.8	0.0515	0.9846	0.203	0.259	4.97	3.1E-03	2.4E-03	
25.6	0.0671	0.9690	0.187	0.239	6.47	1.6E-03	1.3E-03	
6.4	0.0634	0.9727	0.191	0.244	6.11			Unloaded
1.6	0.0591	0.9770	0.195	0.250	5.70			
0.4	0.0498	0.9862	0.204	0.261	4.81			

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-4	***Sample No.:***	-
Soil Description:	SM, BROWN	***Depth (ft):***	15-16.5
Type of Sample:	Undisurbed Ring		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-7	***Sample No.:***	-
Soil Description:	ML/SM, Light Brown	***Depth (ft):***	10-11.5
Type of Sample:	Undisturbed Ring		

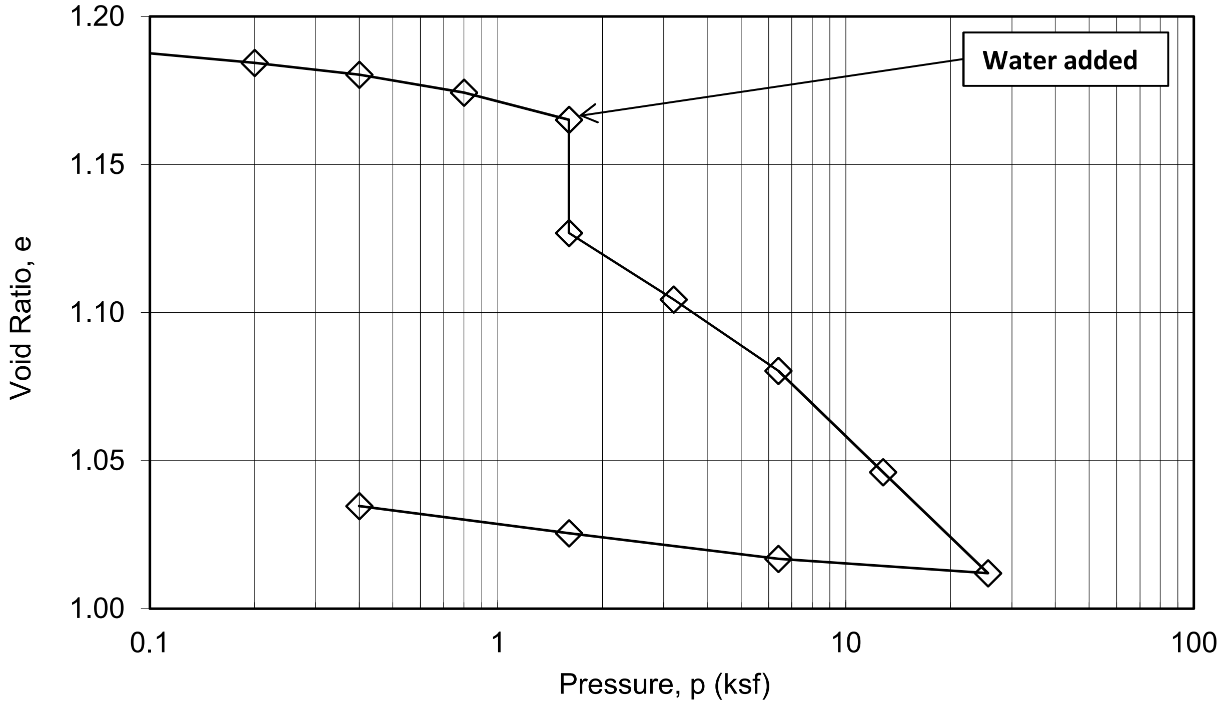
Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
144.82	155.70	137.11

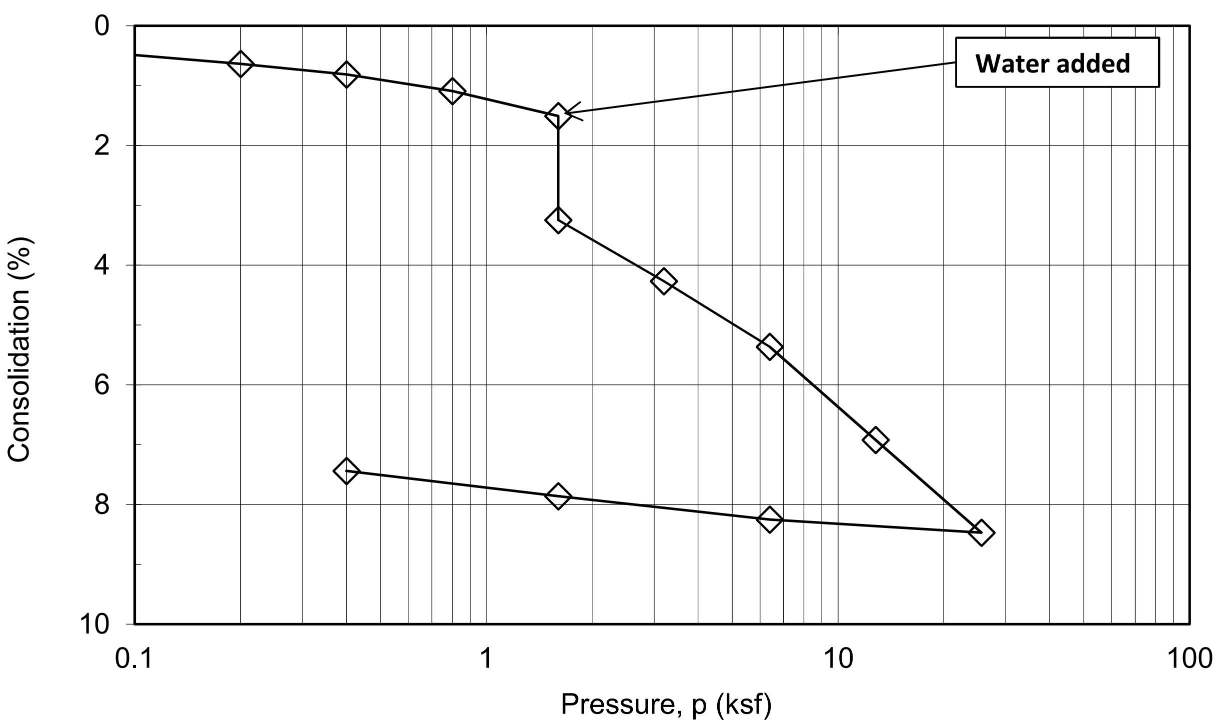
			Initial Conditions	Unload
Height	**H**	**(in)**	1.526	1.4120
Height of Solids	**Hs**	**(in)**	0.694	0.694
Height of Water	**Hw**	**(in)**	0.103	0.247
Height of Air	**Ha**	**(in)**	0.729	0.471
Dry Density		**(pcf)**	**114.5**	**81.1**
Water Content		**(%)**	**5.6**	**13.6**
Saturation		**(%)**	**12.3**	**34.5**

Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.5255	0.832	1.198	0.00			
0.2	0.0097	1.5158	0.822	1.184	0.64	7.4E-02	3.4E-02	
0.4	0.0124	1.5131	0.819	1.180	0.81	1.9E-02	8.9E-03	
0.8	0.0167	1.5089	0.815	1.174	1.09	1.5E-02	7.0E-03	
1.6	0.0230	1.5025	0.809	1.165	1.51	1.1E-02	5.3E-03	
1.6	0.0495	1.4760	0.782	1.127	3.24			Water Added
3.2	0.0652	1.4604	0.766	1.104	4.27	1.4E-02	6.7E-03	
6.4	0.0818	1.4437	0.750	1.080	5.36	7.5E-03	3.6E-03	
12.8	0.1056	1.4199	0.726	1.046	6.92	5.4E-03	2.6E-03	
25.6	0.1293	1.3963	0.702	1.012	8.47	2.7E-03	1.3E-03	
6.4	0.1259	1.3996	0.706	1.017	8.25			Unloaded
1.6	0.1199	1.4056	0.712	1.026	7.86			
0.4	0.1135	1.4120	0.718	1.035	7.44			

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-7	***Sample No.:***	-
Soil Description:	ML/SM, Light Brown	***Depth (ft):***	10-11.5
Type of Sample:	Undisturbed Ring		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-8	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	0-5
Type of Sample:	Remolded to 95% Max Density		

Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
164.87	170.34	152.22

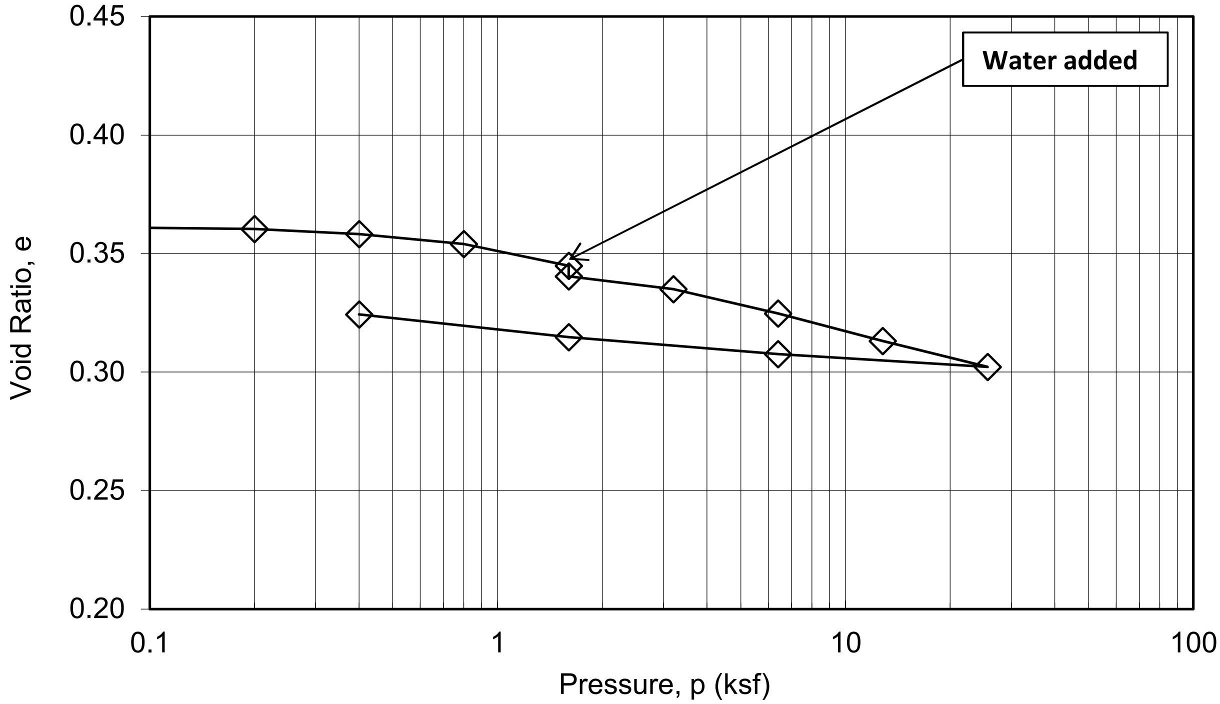
			Initial Conditions	Unload
Height	**H**	**(in)**	1.050	1.0203
Height of Solids	**Hs**	**(in)**	0.770	0.770
Height of Water	**Hw**	**(in)**	0.168	0.241
Height of Air	**Ha**	**(in)**	0.111	0.009
Dry Density		**(pcf)**	**127.1**	**124.6**
Water Content		**(%)**	**8.3**	**11.9**
Saturation		**(%)**	**60.3**	**96.5**

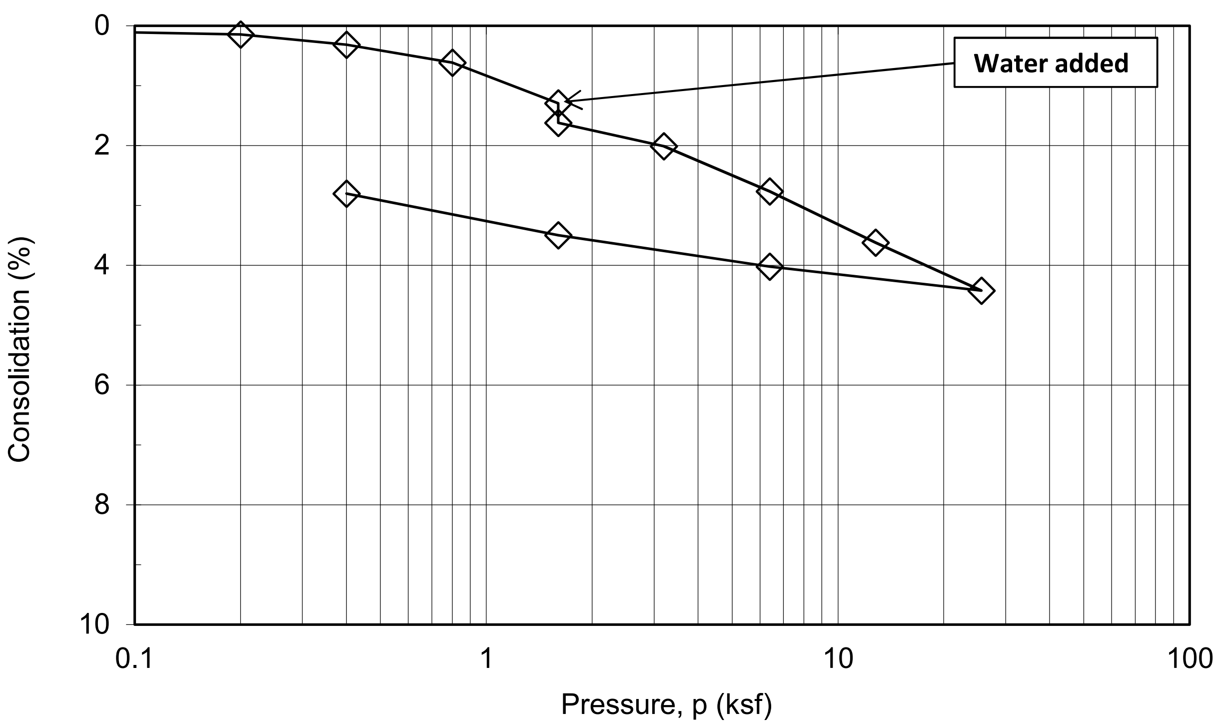
Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0497	0.279	0.362	0.00			
0.2	0.0016	1.0481	0.278	0.360	0.15	1.1E-02	7.8E-03	
0.4	0.0033	1.0464	0.276	0.358	0.31	1.1E-02	8.4E-03	
0.8	0.0064	1.0432	0.273	0.354	0.61	1.0E-02	7.5E-03	
1.6	0.0136	1.0361	0.266	0.345	1.30	1.2E-02	8.6E-03	
1.6	0.0170	1.0327	0.262	0.340	1.62			Water Added
3.2	0.0211	1.0286	0.258	0.335	2.01	3.3E-03	2.5E-03	
6.4	0.0291	1.0206	0.250	0.325	2.77	3.2E-03	2.4E-03	
12.8	0.0380	1.0117	0.241	0.313	3.62	1.8E-03	1.4E-03	
25.6	0.0465	1.0032	0.233	0.302	4.43	8.6E-04	6.6E-04	
6.4	0.0423	1.0074	0.237	0.308	4.03			Unloaded
1.6	0.0367	1.0130	0.243	0.315	3.50			
0.4	0.0294	1.0203	0.250	0.324	2.80			

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-8	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	0-5
Type of Sample:	Remolded to 95% Max Density		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-9	***Sample No.:***	-
Soil Description:	Silty Sand (SM), Brown	***Depth (ft):***	20-21.5
Type of Sample:	Undisturbed Ring		

Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
163.62	166.59	146.86

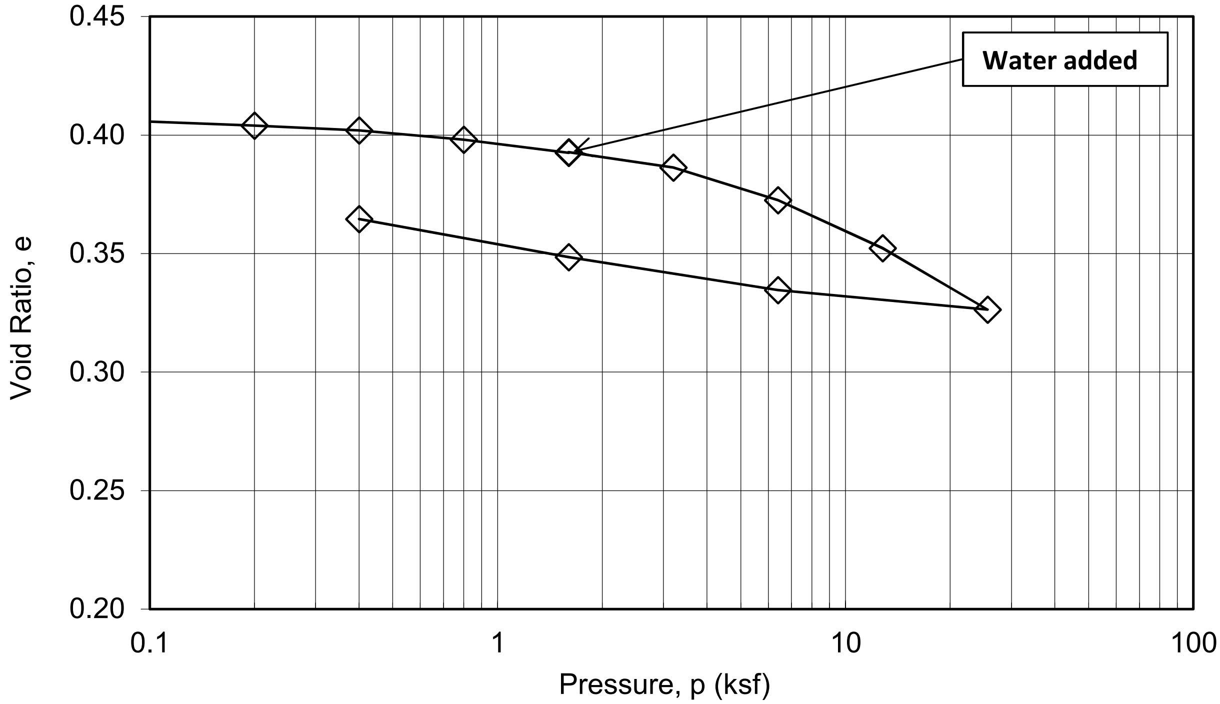
			Initial Conditions	Unload
Height	**H**	**(in)**	1.049	1.0143
Height of Solids	**Hs**	**(in)**	0.743	0.743
Height of Water	**Hw**	**(in)**	0.223	0.263
Height of Air	**Ha**	**(in)**	0.083	0.008
Dry Density		**(pcf)**	**122.7**	**120.9**
Water Content		**(%)**	**11.4**	**13.4**
Saturation		**(%)**	**73.0**	**96.9**

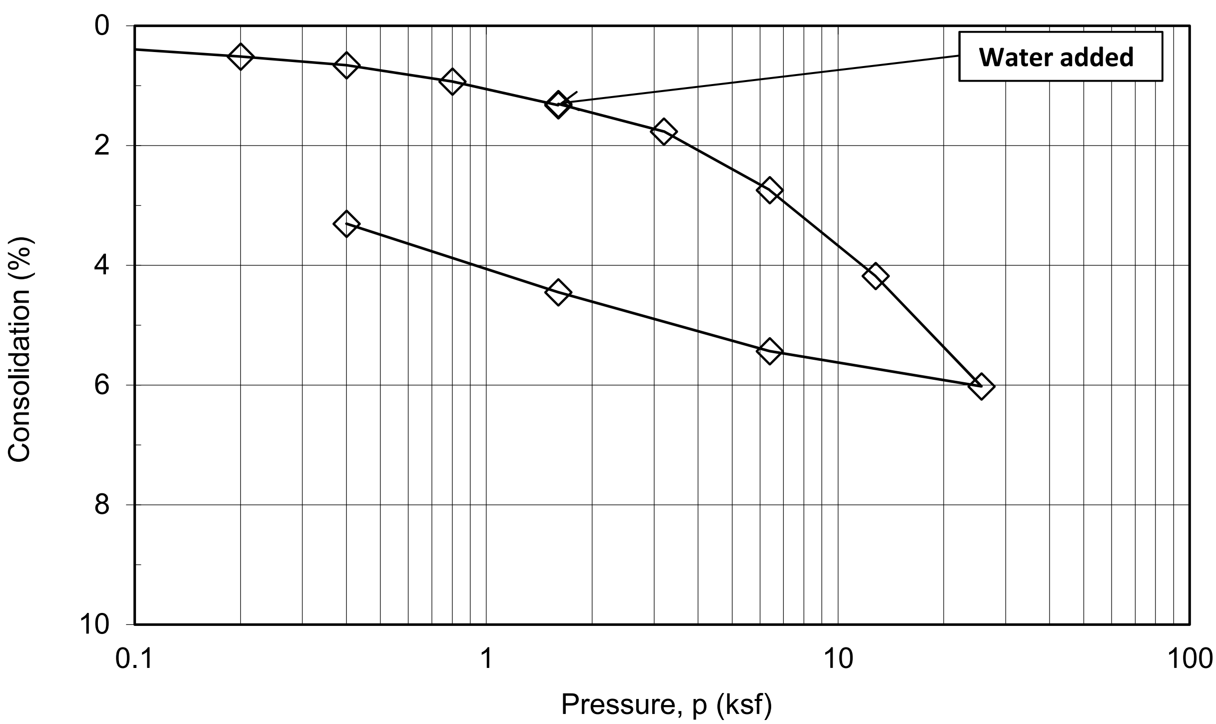
Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0490	0.306	0.411	0.00			
0.2	0.0054	1.0436	0.300	0.404	0.51	3.8E-02	2.7E-02	
0.4	0.0069	1.0421	0.299	0.402	0.66	1.0E-02	7.2E-03	
0.8	0.0098	1.0392	0.296	0.398	0.93	9.8E-03	7.0E-03	
1.6	0.0139	1.0351	0.292	0.393	1.33	6.9E-03	5.0E-03	
1.6	0.0137	1.0353	0.292	0.393	1.31			Water Added
3.2	0.0186	1.0305	0.287	0.386	1.77	4.1E-03	2.9E-03	
6.4	0.0288	1.0202	0.277	0.373	2.75	4.3E-03	3.1E-03	
12.8	0.0439	1.0052	0.262	0.352	4.18	3.2E-03	2.3E-03	
25.6	0.0632	0.9859	0.243	0.326	6.02	2.0E-03	1.5E-03	
6.4	0.0570	0.9920	0.249	0.335	5.43			Unloaded
1.6	0.0467	1.0023	0.259	0.348	4.45			
0.4	0.0347	1.0143	0.271	0.365	3.31			

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-9	***Sample No.:***	-
Soil Description:	Silty Sand (SM), Brown	***Depth (ft):***	20-21.5
Type of Sample:	Undisturbed Ring		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-10	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	15-16.5
Type of Sample:	Undisturbed Ring		

Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
153.04	160.97	143.32

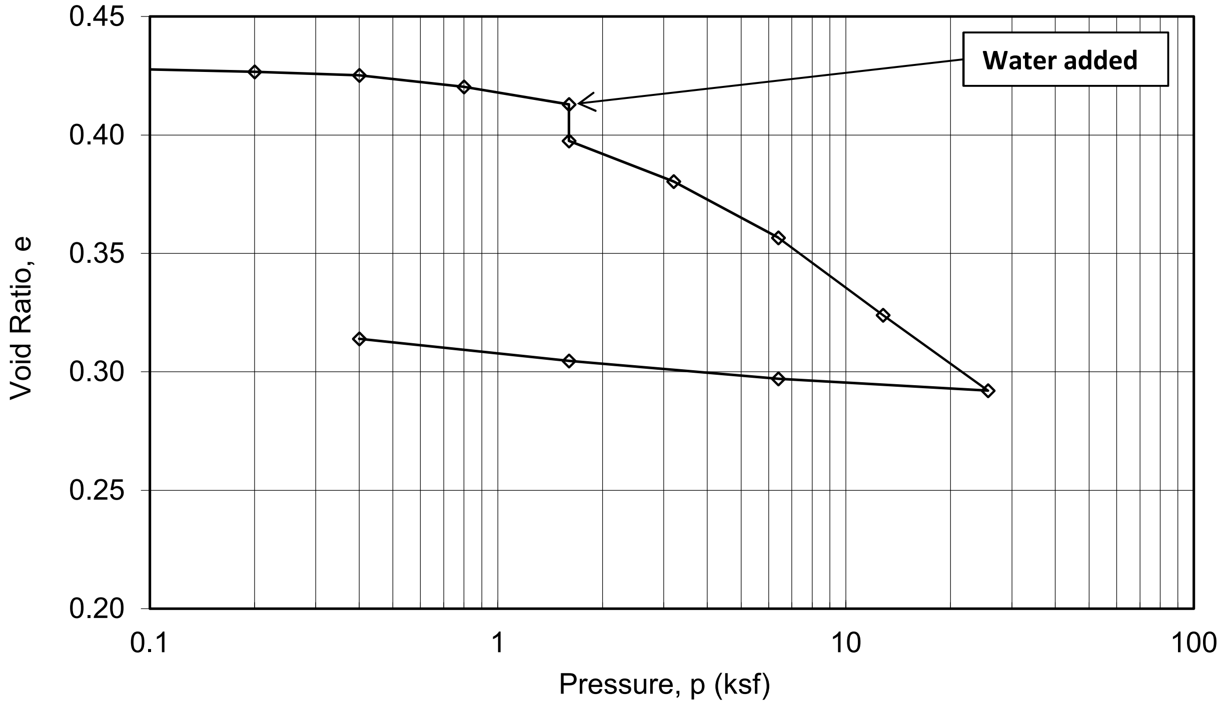
			Initial Conditions	Unload
Height	**H**	**(in)**	1.038	0.9531
Height of Solids	**Hs**	**(in)**	0.725	0.725
Height of Water	**Hw**	**(in)**	0.129	0.235
Height of Air	**Ha**	**(in)**	0.183	0.000
Dry Density		**(pcf)**	**119.7**	**125.6**
Water Content		**(%)**	**6.8**	**12.3**
Saturation		**(%)**	**41.4**	**100.0**

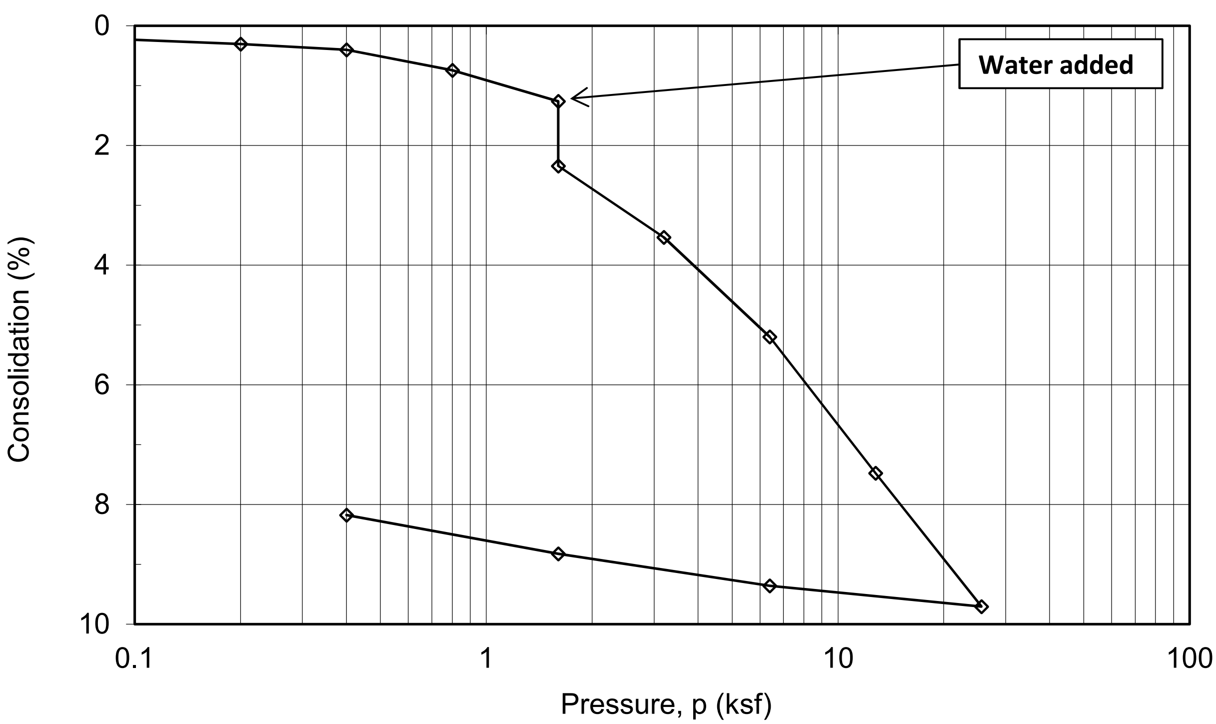
Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0380	0.313	0.431	0.00			
0.2	0.0031	1.0349	0.309	0.427	0.30	2.3E-02	1.6E-02	
0.4	0.0042	1.0338	0.308	0.425	0.40	7.2E-03	5.1E-03	
0.8	0.0078	1.0303	0.305	0.420	0.75	1.2E-02	8.6E-03	
1.6	0.0131	1.0249	0.300	0.413	1.26	9.2E-03	6.5E-03	
1.6	0.0243	1.0137	0.288	0.397	2.34			Water Added
3.2	0.0367	1.0013	0.276	0.380	3.54	1.1E-02	7.7E-03	
6.4	0.0540	0.9841	0.259	0.357	5.20	7.4E-03	5.5E-03	
12.8	0.0776	0.9604	0.235	0.324	7.48	5.1E-03	3.8E-03	
25.6	0.1008	0.9373	0.212	0.292	9.71	2.5E-03	1.9E-03	
6.4	0.0972	0.9409	0.215	0.297	9.36			Unloaded
1.6	0.0916	0.9464	0.221	0.305	8.82			
0.4	0.0849	0.9531	0.228	0.314	8.18			

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-10	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	15-16.5
Type of Sample:	Undisturbed Ring		





CONSOLIDATION TEST
(ASTM D2435)

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	KL
Project No.:	C.314.81.00	***Checked by:***	MZ
		Date:	7/7/2017
Boring No.:	B-10	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	25-216.5
Type of Sample:	Undisturbed Ring		

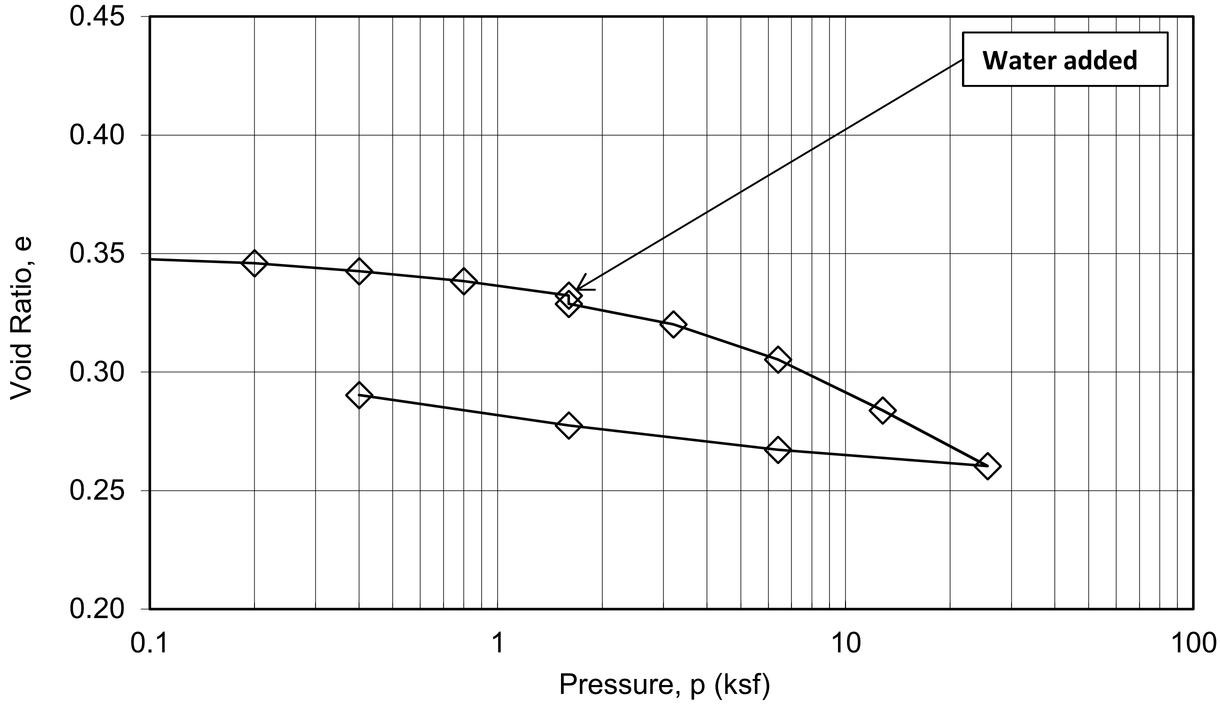
Initial Total Weight (g)	Final Total Weight (g)	Final Dry Weight (g)
162.06	165.70	147.63

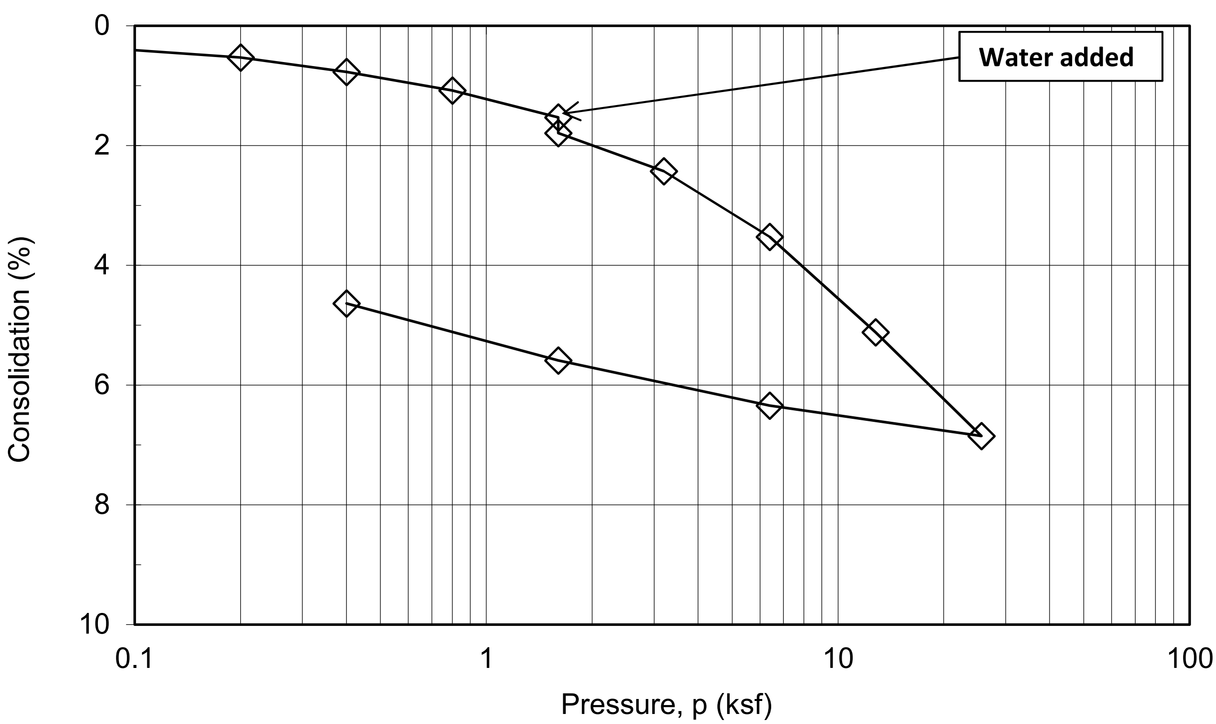
			Initial Conditions	Unload
Height	**H**	**(in)**	1.011	0.9641
Height of Solids	**Hs**	**(in)**	0.747	0.747
Height of Water	**Hw**	**(in)**	0.192	0.241
Height of Air	**Ha**	**(in)**	0.072	0.000
Dry Density		**(pcf)**	**123.3**	**127.9**
Water Content		**(%)**	**9.8**	**12.2**
Saturation		**(%)**	**72.8**	**100.0**

Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consolidation (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0110	0.264	0.353	0.00			
0.2	0.0054	1.0057	0.258	0.346	0.53	3.8E-02	2.8E-02	
0.4	0.0078	1.0032	0.256	0.343	0.77	1.6E-02	1.2E-02	
0.8	0.0110	1.0001	0.253	0.338	1.08	1.1E-02	7.9E-03	
1.6	0.0155	0.9955	0.248	0.332	1.53	7.6E-03	5.7E-03	
1.6	0.0181	0.9929	0.246	0.329	1.79			Water Added
3.2	0.0246	0.9865	0.239	0.320	2.43	5.4E-03	4.1E-03	
6.4	0.0357	0.9754	0.228	0.305	3.53	4.6E-03	3.6E-03	
12.8	0.0518	0.9593	0.212	0.284	5.12	3.4E-03	2.6E-03	
25.6	0.0693	0.9418	0.195	0.260	6.85	1.8E-03	1.5E-03	
6.4	0.0642	0.9469	0.200	0.267	6.35			Unloaded
1.6	0.0565	0.9545	0.207	0.277	5.59			
0.4	0.0469	0.9641	0.217	0.290	4.64			

CONSOLIDATION TEST
(ASTM D2435)

Client:	Geobase		
Project Name:	KP Moreno Valley Medical Center		
Project No.:	C.314.81.00		
Boring No.:	B-10	***Sample No.:***	-
Soil Description:	SM, Brown	***Depth (ft):***	25-216.5
Type of Sample:	Undisturbed Ring		





HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

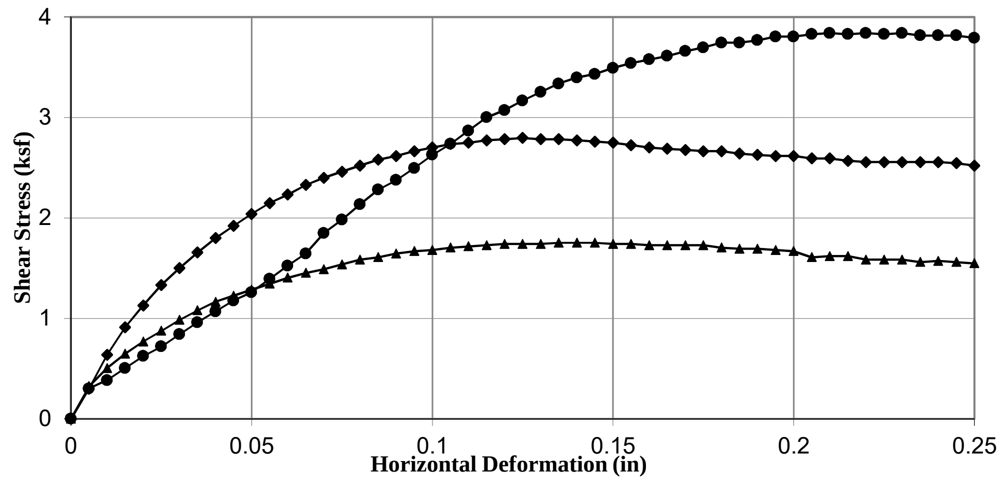
HAI Pr No.:	GBA-17-001
Tested by:	KL
Checked by:	MZ
Date:	7/10/2017

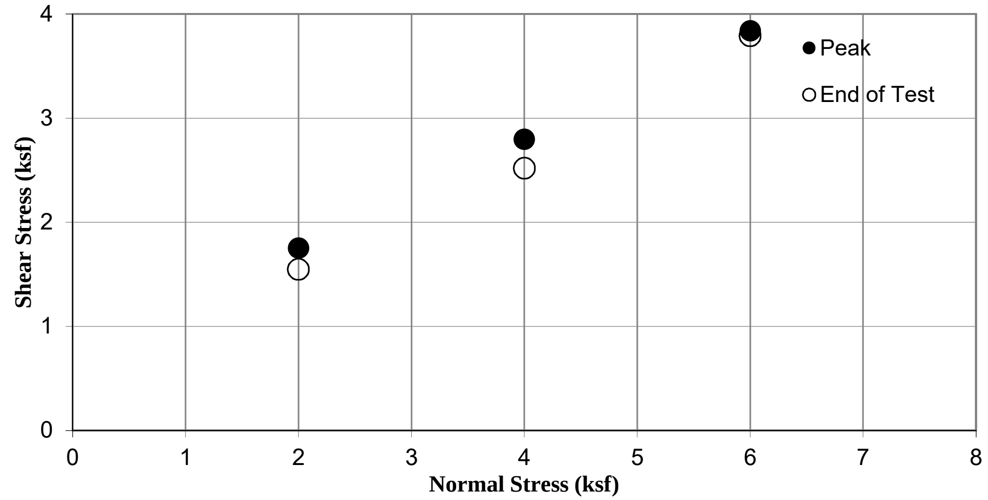
Client:	Geobase
Project Name:	KP Moreno Valley Medical Center
Project Number:	C.314.81.00
Boring No.:	B-2
Sample No.:	Ring
Depth (ft):	10-11.5'
Soil description:	Brown, Silty Sand with Few Gravel (SM)
Sample type:	Undisturbed ring
Type of test:	Consolidated

	▲	◆	●
Normal Stress (ksf)	**2**	**4**	**6**
Deformation Rate (in/min)	**0.002**		

Peak Shear Stress (ksf) ●	**1.75**	**2.80**	**3.84**
Shear stress @ end of test (ksf) ○	**1.55**	**2.52**	**3.79**

Initial height of sample (in)	**1**	**1**	**1**
Height of sample before shear (in)	**1.0135**	**0.9879**	**0.9617**
Diameter of sample (in)	**2.42**	**2.42**	**2.42**
Initial Moisture Content (%)	**7.5**	**7.5**	**7.5**
Final Moisture Content (%)	**13.0**	**13.4**	**13.1**
Dry Density (pcf)	**114.6**	**115.6**	**109.7**
Final Saturation (%)	**82.1**	**95.2**	**84.8**





DIRECT SHEAR TEST

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

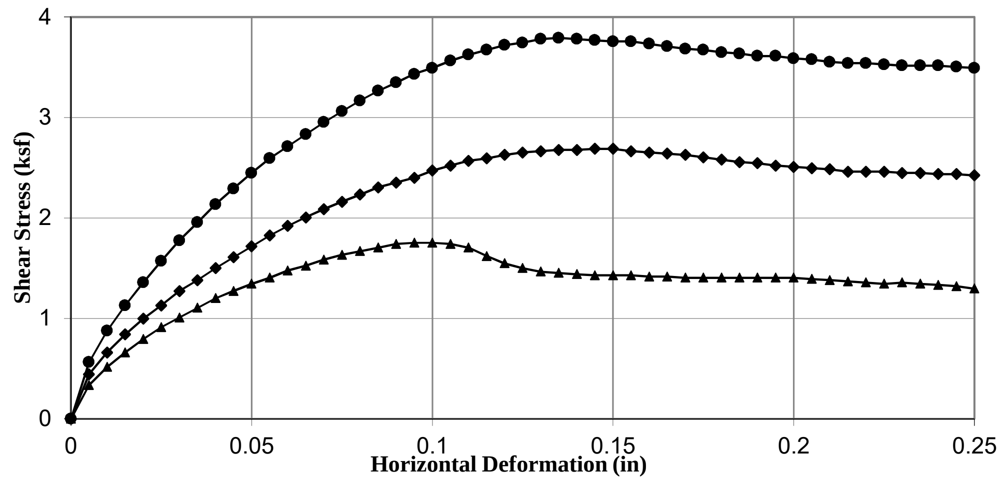
HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

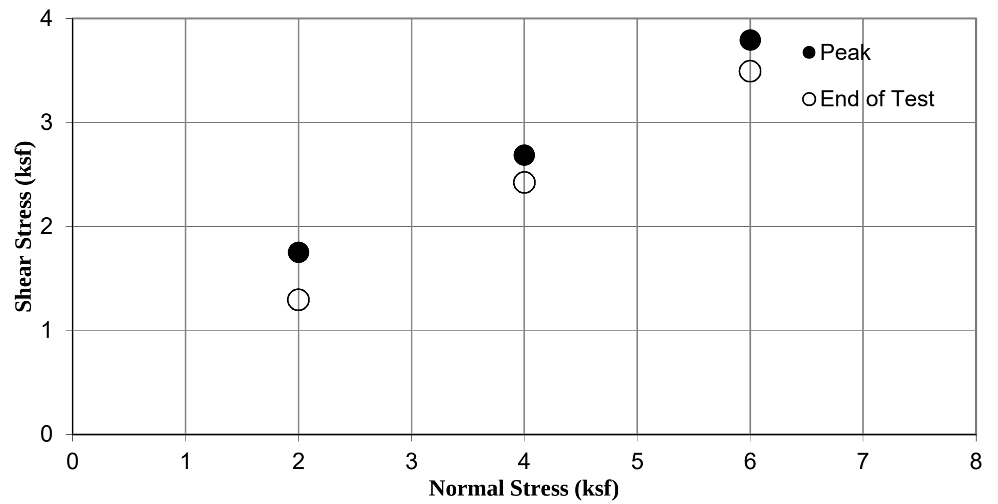
Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00
Boring No.: B-2
Sample No.: Ring
Depth (ft): 20-21.5'
Soil description: Brown, Silty Sand with Gravel (SM)
Sample type: Undisturbed ring
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	**2**	**4**	**6**
Deformation Rate (in/min)	**0.002**		

Peak Shear Stress (ksf) ●	**1.75**	**2.69**	**3.79**
Shear stress @ end of test (ksf) ○	**1.30**	**2.42**	**3.49**

Initial height of sample (in)	**1**	**1**	**1**
Height of sample before shear (in)	**0.9961**	**0.9813**	**0.9605**
Diameter of sample (in)	**2.42**	**2.42**	**2.42**
Initial Moisture Content (%)	**6.5**	**6.5**	**6.5**
Final Moisture Content (%)	**16.0**	**16.3**	**15.5**
Dry Density (pcf)	**118.5**	**119.4**	**119.1**
Final Saturation (%)	**91.1**	**99.2**	**100.0**





DIRECT SHEAR TEST

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

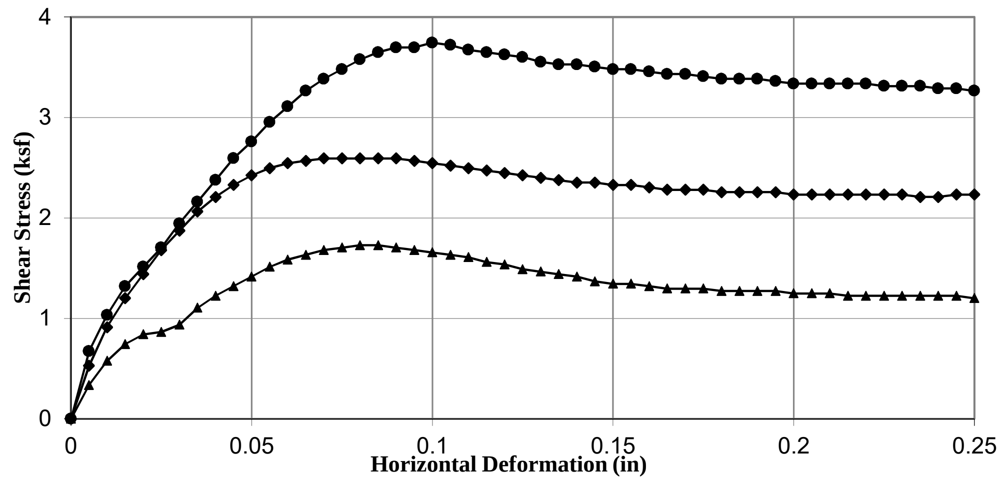
HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

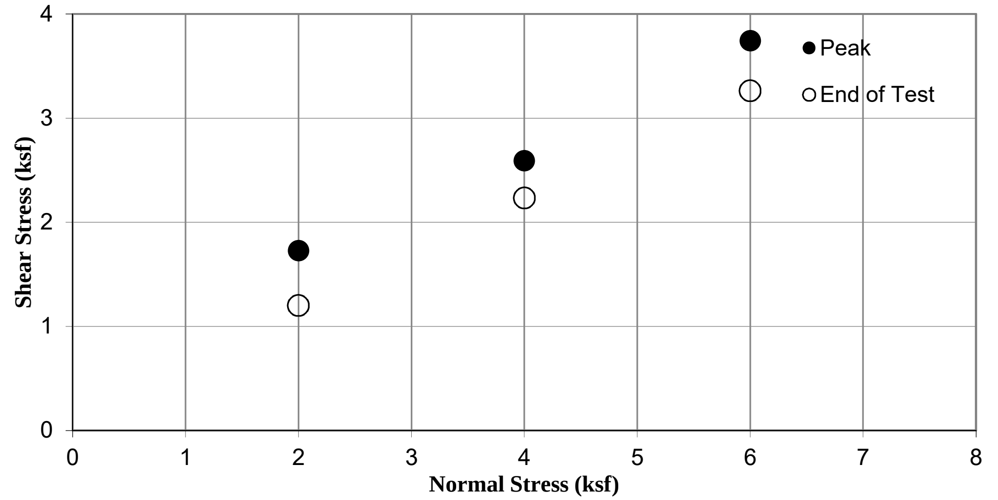
Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00
Boring No.: B-3
Sample No.: Ring
Depth (ft): 15-16.5'
Soil description: Brown, Clayey Sand (SC)
Sample type: Undisturbed ring
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	**2**	**4**	**6**
Deformation Rate (in/min)		**0.002**	

Peak Shear Stress (ksf) ●	**1.73**	**2.59**	**3.74**
Shear stress @ end of test (ksf) ○	**1.20**	**2.23**	**3.26**

Initial height of sample (in)	**1**	**1**	**1**
Height of sample before shear (in)	**0.9353**	**0.9762**	**0.9849**
Diameter of sample (in)	**2.42**	**2.42**	**2.42**
Initial Moisture Content (%)	**13.2**	**13.2**	**13.2**
Final Moisture Content (%)	**18.4**	**18.3**	**16.5**
Dry Density (pcf)	**121.7**	**119.7**	**126.7**
Final Saturation (%)	**115.5**	**96.8**	**100.0**





DIRECT SHEAR TEST

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

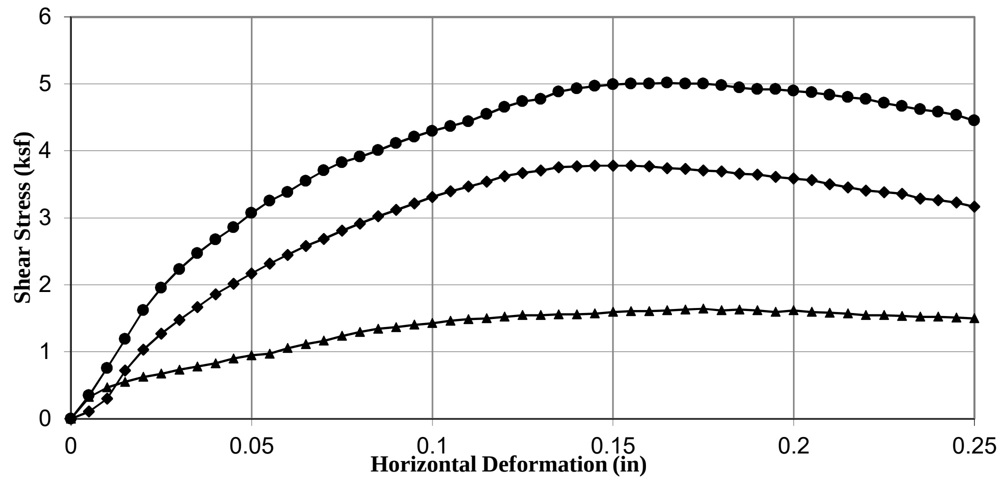
HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

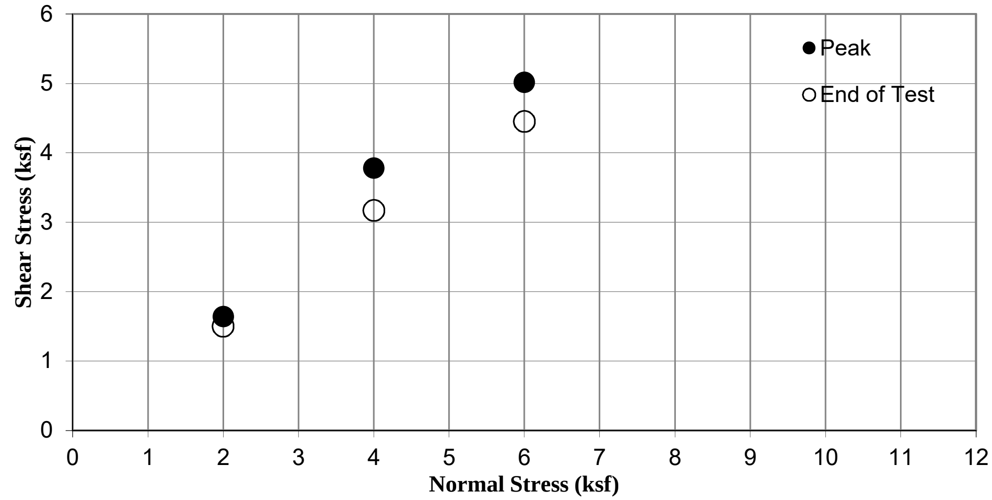
Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00
Boring No.: B-3
Sample No.: Ring
Depth (ft): 35-36.5'
Soil description: Brown, Silty Sand (SM)
Sample type: Undisturbed ring
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	**2**	**4**	**6**
Deformation Rate (in/min)	**0.002**		

Peak Shear Stress (ksf) ●	**1.64**	**3.78**	**5.02**
Shear stress @ end of test (ksf) ○	**1.50**	**3.17**	**4.45**

Initial height of sample (in)	**1**	**1**	**1**
Height of sample before shear (in)	**1.0247**	**1.0051**	**0.9641**
Diameter of sample (in)	**2.42**	**2.42**	**2.42**
Initial Moisture Content (%)	**4.4**	**4.4**	**4.4**
Final Moisture Content (%)	**13.7**	**13.1**	**13.0**
Dry Density (pcf)	**115.4**	**115.9**	**115.2**
Final Saturation (%)	**85.0**	**88.3**	**100.0**





DIRECT SHEAR TEST

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

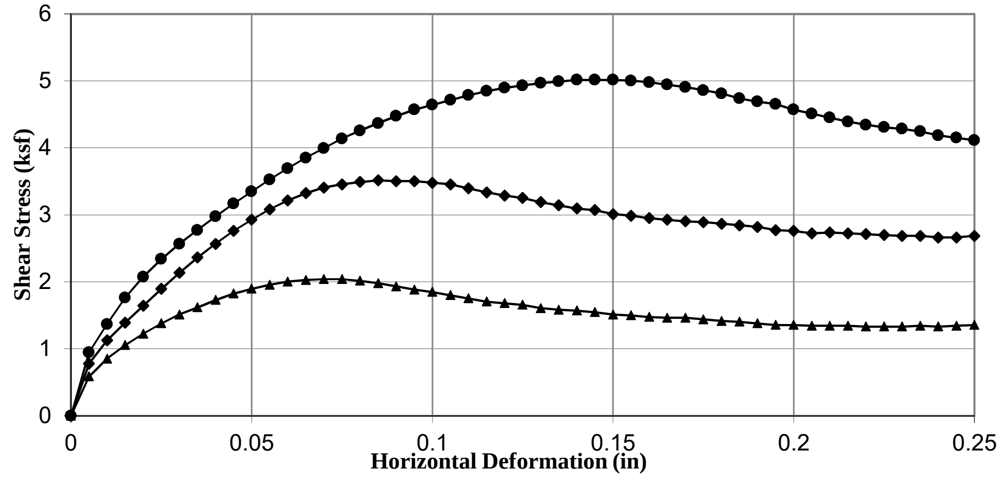
HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

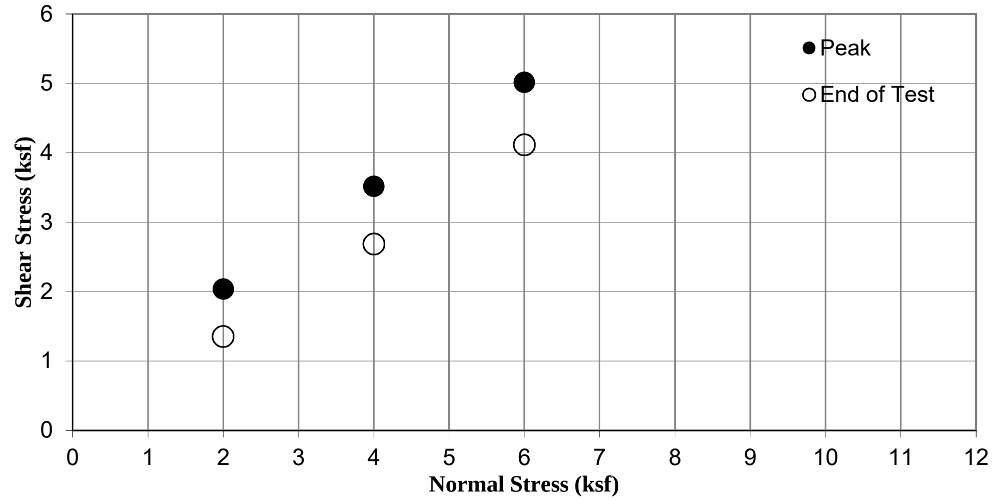
Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00
Boring No.: B-4
Sample No.: Ring
Depth (ft): 15-16.5'
Soil description: Brown, Clayey Sand (SC)
Sample type: Undisturbed ring
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	**2**	**4**	**6**
Deformation Rate (in/min)		**0.002**	

Peak Shear Stress (ksf) ●	**2.04**	**3.52**	**5.02**
Shear stress @ end of test (ksf) ○	**1.36**	**2.69**	**4.12**

Initial height of sample (in)	**1**	**1**	**1**
Height of sample before shear (in)	**1.0152**	**1.0032**	**1.0181**
Diameter of sample (in)	**2.42**	**2.42**	**2.42**
Initial Moisture Content (%)	**11.0**	**11.0**	**11.0**
Final Moisture Content (%)	**12.5**	**12.5**	**12.9**
Dry Density (pcf)	**126.7**	**127.0**	**127.4**
Final Saturation (%)	**94.1**	**99.5**	**97.9**





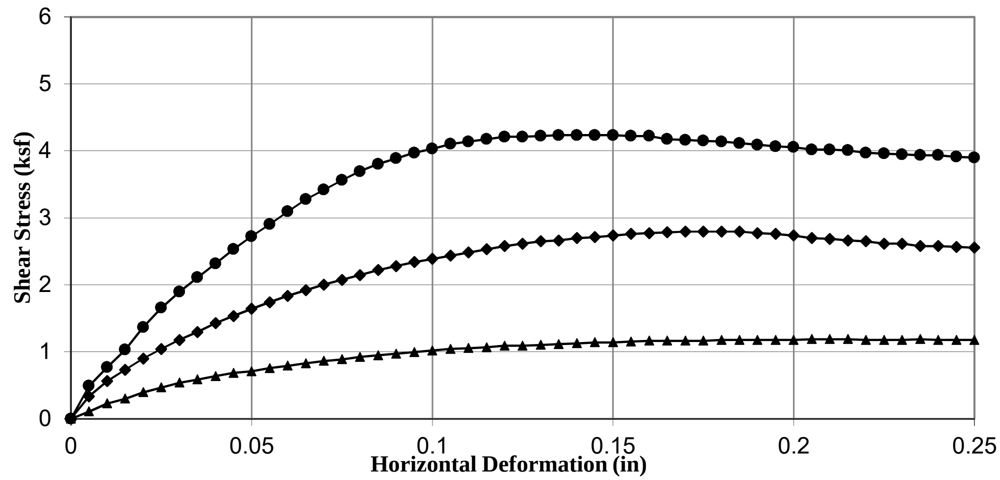
HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

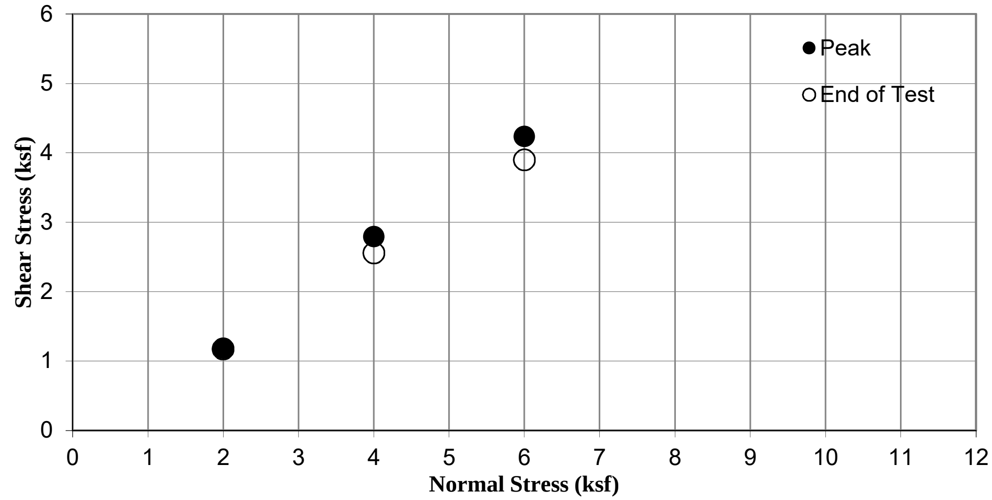
DIRECT SHEAR TEST

HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00
Boring No.: B-7
Sample No.: Ring
Depth (ft): 10-11.5'
Soil description: Brown, Silty Sand (SM)
Sample type: Undisturbed ring
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	2	4	6
Deformation Rate (in/min)	0.002		
Peak Shear Stress (ksf) ●	1.19	2.80	4.24
Shear stress @ end of test (ksf) ○	1.18	2.56	3.90
Initial height of sample (in)	1	1	1
Height of sample before shear (in)	0.9886	0.9602	0.9558
Diameter of sample (in)	2.42	2.42	2.42
Initial Moisture Content (%)	5.2	5.2	5.2
Final Moisture Content (%)	16.3	14.7	14.0
Dry Density (pcf)	99.5	104.8	106.5
Final Saturation (%)	75.9	87.8	89.8





DIRECT SHEAR TEST

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

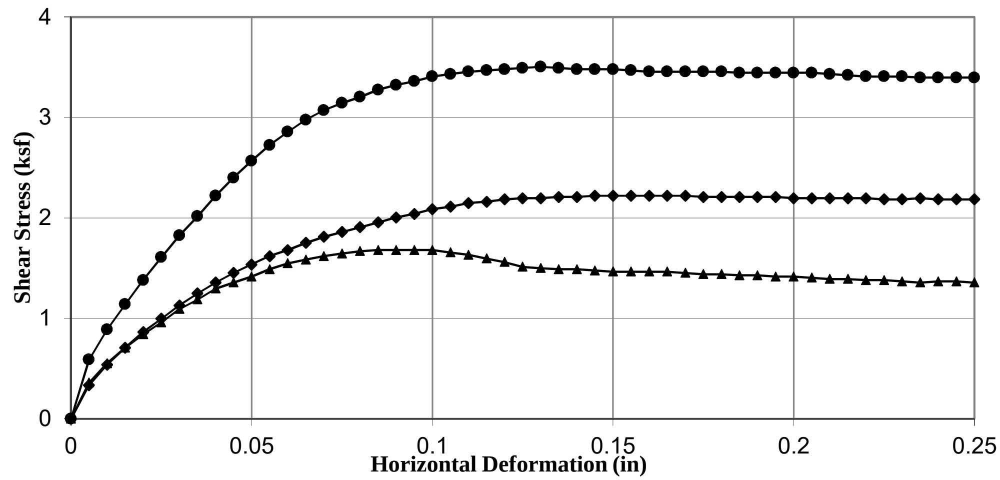
HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

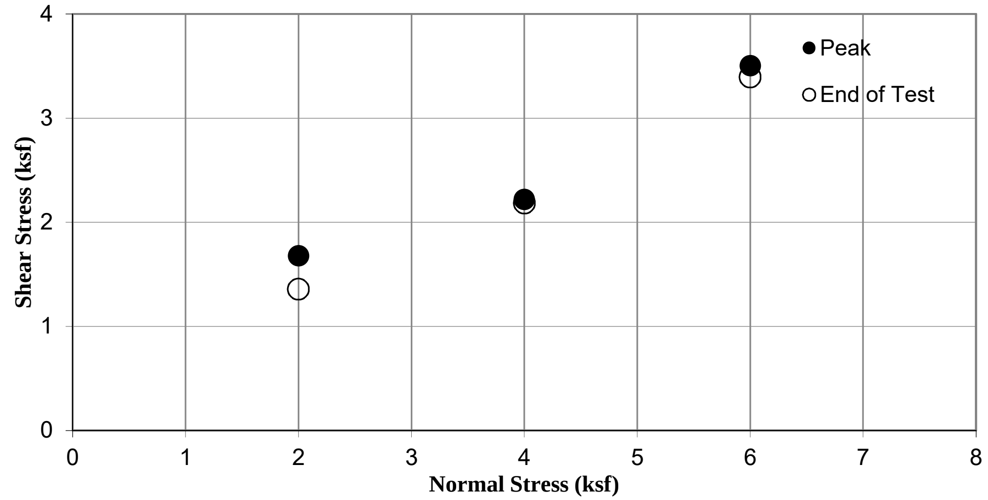
Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00
Boring No.: B-9
Sample No.: Ring
Depth (ft): 20-21.5'
Soil description: Brown, Silty Sand (SM)
Sample type: Undisturbed ring
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	2	4	6
Deformation Rate (in/min)	0.002		

Peak Shear Stress (ksf) ●	1.68	2.22	3.50
Shear stress @ end of test (ksf) ○	1.36	2.18	3.40

Initial height of sample (in)	1	1	1
Height of sample before shear (in)	1.0107	0.9813	0.9998
Diameter of sample (in)	2.42	2.42	2.42
Initial Moisture Content (%)	11.0	11.0	11.0
Final Moisture Content (%)	16.4	16.0	15.1
Dry Density (pcf)	112.5	112.6	114.4
Final Saturation (%)	92.6	99.9	93.6





HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00

Boring No.: B-10

Sample No.: Ring

Depth (ft): 15-16.5'

Soil description: Brown, Silty Sand with Gravel (SM)

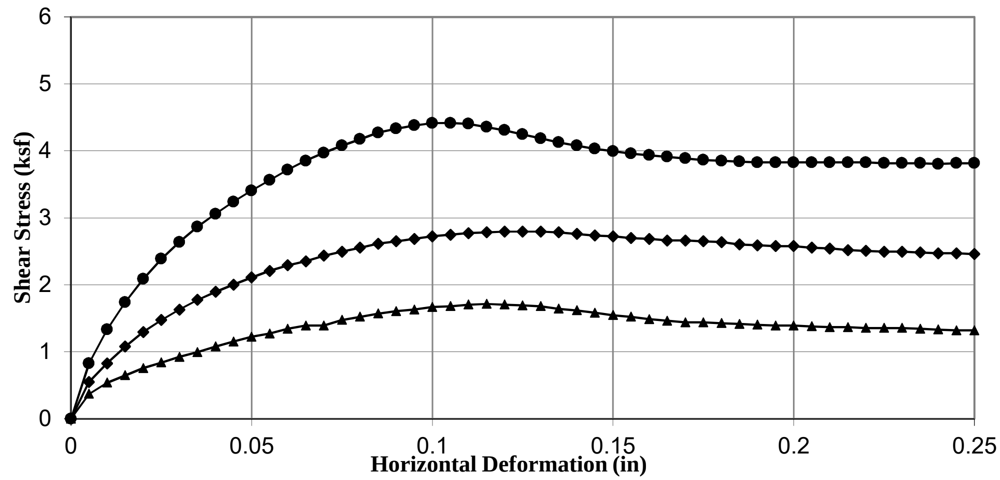
Sample type: Undisturbed ring

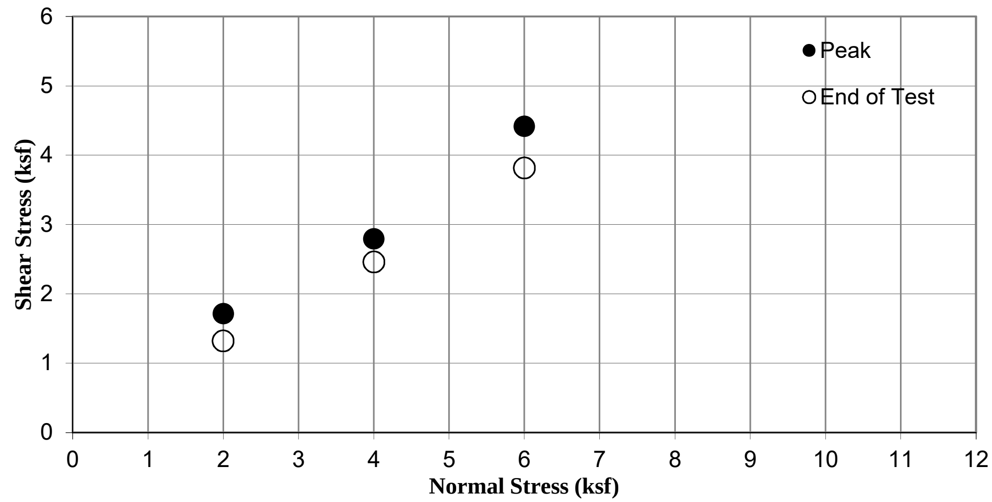
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	**2**	**4**	**6**
Deformation Rate (in/min)	**0.002**		

Peak Shear Stress (ksf) ●	**1.72**	**2.80**	**4.42**
Shear stress @ end of test (ksf) ○	**1.32**	**2.46**	**3.82**

Initial height of sample (in)	**1**	**1**	**1**
Height of sample before shear (in)	**0.9702**	**0.9496**	**0.9504**
Diameter of sample (in)	**2.42**	**2.42**	**2.42**
Initial Moisture Content (%)	**6.4**	**6.4**	**6.4**
Final Moisture Content (%)	**14.5**	**13.8**	**12.7**
Dry Density (pcf)	**125.4**	**126.4**	**129.1**
Final Saturation (%)	**93.6**	**98.2**	**96.7**





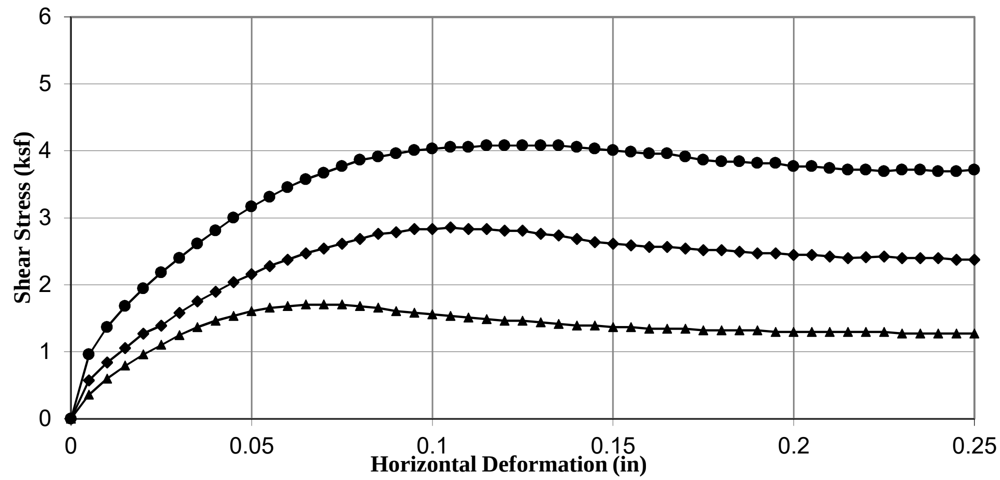
HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

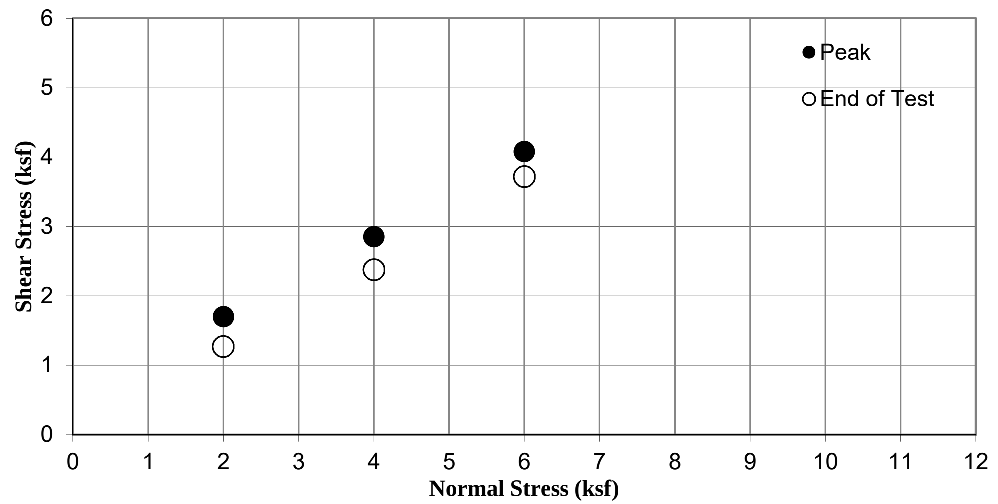
DIRECT SHEAR TEST

HAI Pr No.: GBA-17-001
Tested by: KL
Checked by: MZ
Date: 7/10/2017

Client: Geobase
Project Name: KP Moreno Valley Medical Center
Project Number: C.314.81.00
Boring No.: B-10
Sample No.: Ring
Depth (ft): 25-26.5'
Soil description: Brown, Silty Sand with Gravel (SM)
Sample type: Undisturbed ring
Type of test: Consolidated

	▲	◆	●
Normal Stress (ksf)	2	4	6
Deformation Rate (in/min)		0.002	
Peak Shear Stress (ksf) ●	1.70	2.86	4.08
Shear stress @ end of test (ksf) ○	1.27	2.38	3.72
Initial height of sample (in)	1	1	1
Height of sample before shear (in)	0.9852	0.9930	0.9537
Diameter of sample (in)	2.42	2.42	2.42
Initial Moisture Content (%)	8.8	8.8	8.8
Final Moisture Content (%)	13.6	15.0	15.2
Dry Density (pcf)	129.7	130.4	125.5
Final Saturation (%)	89.7	98.4	100.1





EXPANSION POTENTIAL
ASTM D 4829

SOIL SAMPLE LOCATION (feet)	EXPANSION INDEX	EXPANSION POTENTIAL
B-2 at 5.0-10.0	8	Very Low
B-3 at 5.0-10.0	10	Very Low
B-8 at 0-5.0	0	Very Low
B-10 at 0-5.0	4	Very Low
B-1 at 5.0 -10.0 (GEOBASE, 2010)	10	Very Low

WATER-SOLUBLE SULFATES
CT. 417

SOIL SAMPLE LOCATION (feet)	SOLUBLE SULFATES PPM	POTENTIAL FOR ATTACK ON CONCRETE
B-2 at 5.0-10.0	115	Low
B-3 at 5.0-10.0	95	Low
B-8 at 0-5.0	62	Low
B-10 at 0-5.0	74	Low
B-1 at 5.0 -10.0 (GEOBASE, 2010)	43	Low

CORROSIVITY SERIES TEST

SOIL SAMPLE LOCATION (feet)	pH (CT 643)	SOLUBLE CHLORIDE (CT.422) (PPM)	ELEC. RESISTIVITY (CT.643) (OHM-CM)	DEGREE OF CORROSIVITY
B-2 at 5.0-10.0	6.7	101	2100	moderately corrosive
B-3 at 5.0-10.0	6.8	153	2100	moderately corrosive
B-8 at 0-5.0	7.2	17	7600	moderately corrosive
B-10 at 0-5.0	6.9	47	1600	corrosive
B-1 at 5.0 -10.0 (GEOBASE, 2010)	7.0	15	5600	moderately corrosive

R-VALUE
(CALTRANS CT 301)

SOIL SAMPLE LOCATION (feet)	R-VALUE BY EXUDATION
B-8 at 0-5.0	59
B-10 at 0-5.0	54

MAXIMUM DRY DENSITY/OPTIMUM MOISTURE CONTENT
ASTM D1557

Boring No.	Maximum Dry Density (PCF)	Optimum Moisture Contents (%)
B-2 at 5.0-10.0	136.7	7.3
B-3 at 5.0-10.0	137.8	6.7
B-8 at 0-5.0	134.9	7.4

ANAHEIM TEST LABORATORY

3008 ORANGE AVENUE
SANTA ANA, CALIFORNIA 92707
PHONE (714) 549-7267

TO:

GEOBASE
23362 PERALTA DRIVE, # 4&6
LAGUNA HILLS, CA. 92653

DATE: 06/16/17

P.O. NO: VERBAL

LAB NO: C-0661 1-4

SPECIFICATION: CA-417/422/643

MATERIAL: SOIL

PROJECT #: C.314.81.00
KP MVMC PS
Date sampled: 06/09/17

ANALYTICAL REPORT

CORROSION SERIES
SUMMARY OF DATA

	PH	SOLUBLE SULFATES per CA. 417 ppm	SOLUBLE CHLORIDES per CA. 422 ppm	MIN. RESISTIVITY per CA. 643 ohm-cm
1) B-2 @ 0-10’	6.7	115	101	2,100
2) B-3 @ 0-10’	6.8	95	153	2,100
3) B-8 @ 0-5’	7.2	62	17	7,600
4) B-10 @ 0-5’	6.9	74	47	1,600

RESPECTFULLY SUBMITTED

WES BRIDGER CHEMIST

Table 1 - Laboratory Tests on Soil Samples

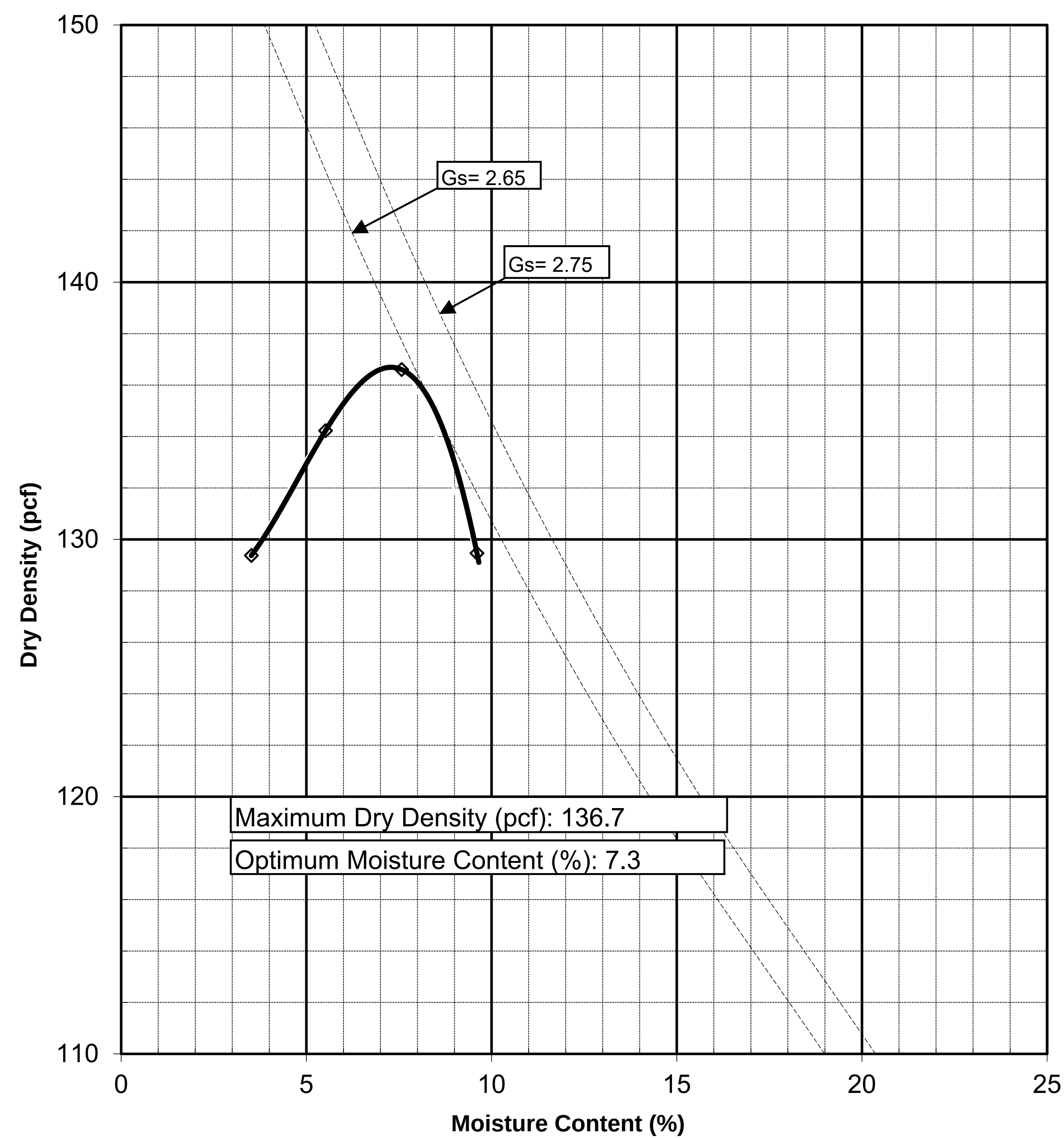
GEOBASE, INC.
MUCH
Your #C.314.39.00, SA #10-333LAB
7-Apr-10

Sample ID			B-1 @ 5-10' SM	B-3 @ 5-10' SM	B-5 5-10' SM
Resistivity		**Units**			
as-received		ohm-cm	13,600	14,400	22,800
saturated		ohm-cm	5,600	8,400	3,000
pH			7.0	7.4	7.6
Electrical Conductivity		mS/cm	0.05	0.05	0.13
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	28	28	67
magnesium	Mg^{2+}	mg/kg	5.2	5.3	14
sodium	Na^{1+}	mg/kg	51	40	66
potassium	K^{1+}	mg/kg	5.4	17	5.9
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	49	64	156
flouride	F^{1-}	mg/kg	2.8	3.0	1.1
chloride	Cl^{1-}	mg/kg	15	6.9	46
sulfate	SO_4^{2-}	mg/kg	43	22	70
phosphate	PO_4^{3-}	mg/kg	11	35	8.8
Other Tests					
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	4.4	15	43
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

COMPACTION CURVE
(ASTM D1557)

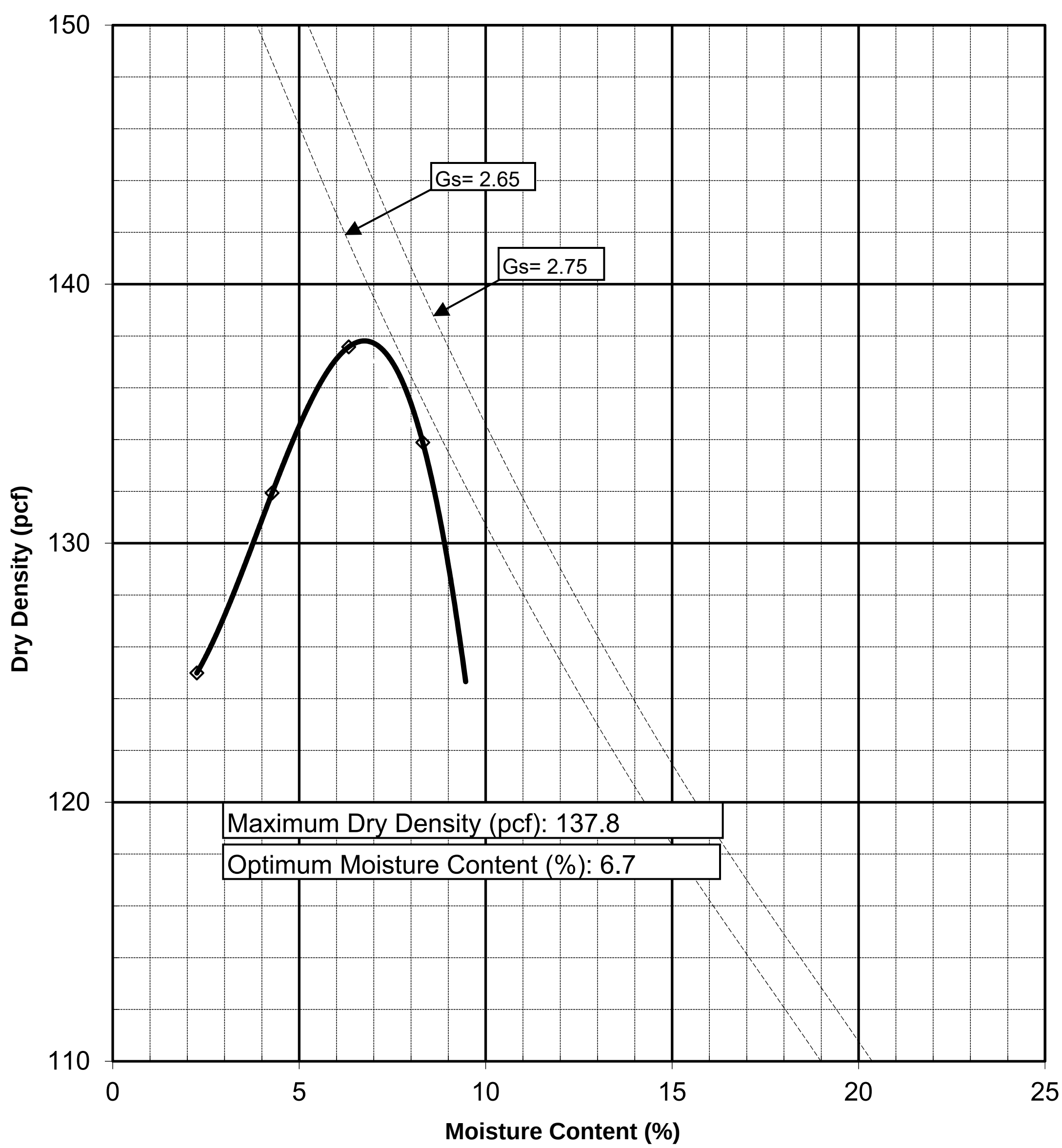
Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB
Project No.:	C.314.81.00	***Checked by:***	MZ
Boring No:	B-2	***Date:***	6/28/2017
Sample No.:	5-10'	***Mold size:***	4 "
Soil Description:	Brown, Silty Sand (SM)	***Procedure:***	A
		% Ret. On # 4	0.9



COMPACTION CURVE
(ASTM D1557)

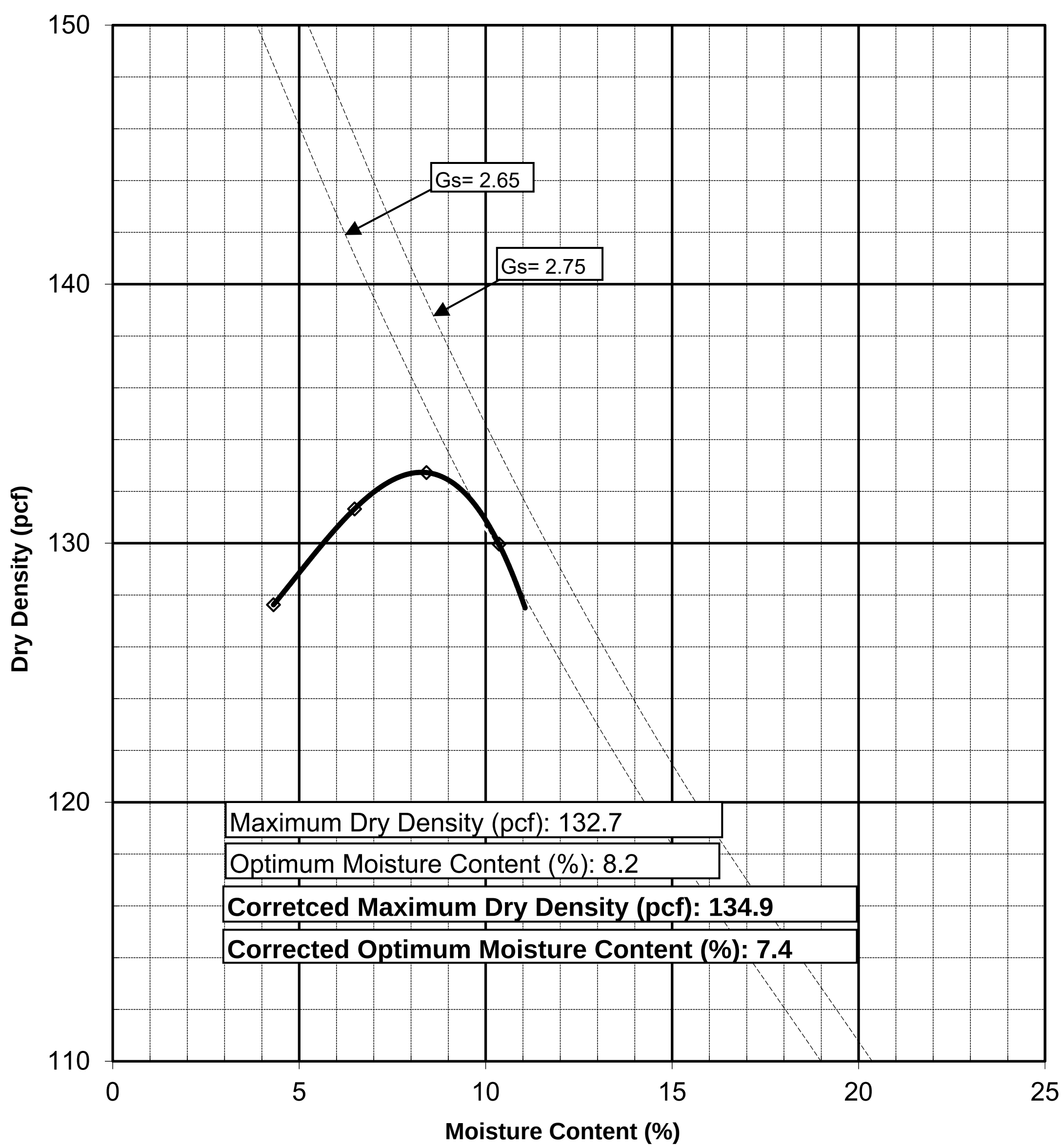
HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	MB
Project No.:	C.314.81.00	***Checked by:***	MZ
Boring No:	B-3	***Date:***	6/28/2017
Sample No.:	5-10'	***Mold size:***	4 "
Soil Description:	Brown, Silty Sand (SM)	***Procedure:***	A
		% Ret. On # 4	0.7



COMPACTION CURVE
(ASTM D1557)

Client :	Geobase	***HAI Project No.:***	GBA-17-001
Project Name:	KP Moreno Valley Medical Center	***Tested by:***	RH
Project No.:	C.314.81.00	***Checked by:***	MZ
Boring No:	B-8	***Date:***	6/28/2017
Sample No.:	0-5'	***Mold size:***	4 "
Soil Description:	Brown, Silty Sand (SM)	***Procedure:***	A
		% Ret. On # 4	10.7



ANAHEIM TEST LAB, INC

3008 ORANGE AVENUE
SANTA ANA, CALIFORNIA 92707
PHONE (714) 549-7267

TO:

GEOBASE
23362 PERALTA DRIVE, # 4&6
LAGUNA HILLS, CA. 92653

DATE: 06/19/17

P.O. NO: VERBAL

LAB NO: C-0661-4

SPECIFICATION: CT 301

MATERIAL: Brown, F.C. Silty Sand

PROJECT #: C.314.81.00

ANALYTICAL REPORT

"R" VALUE

	BY EXUDATION	BY EXPANSION
B-10 @ 0-5'	54	N/A

RESPECTFULLY SUBMITTED

WES BRIDGER CHEMIST

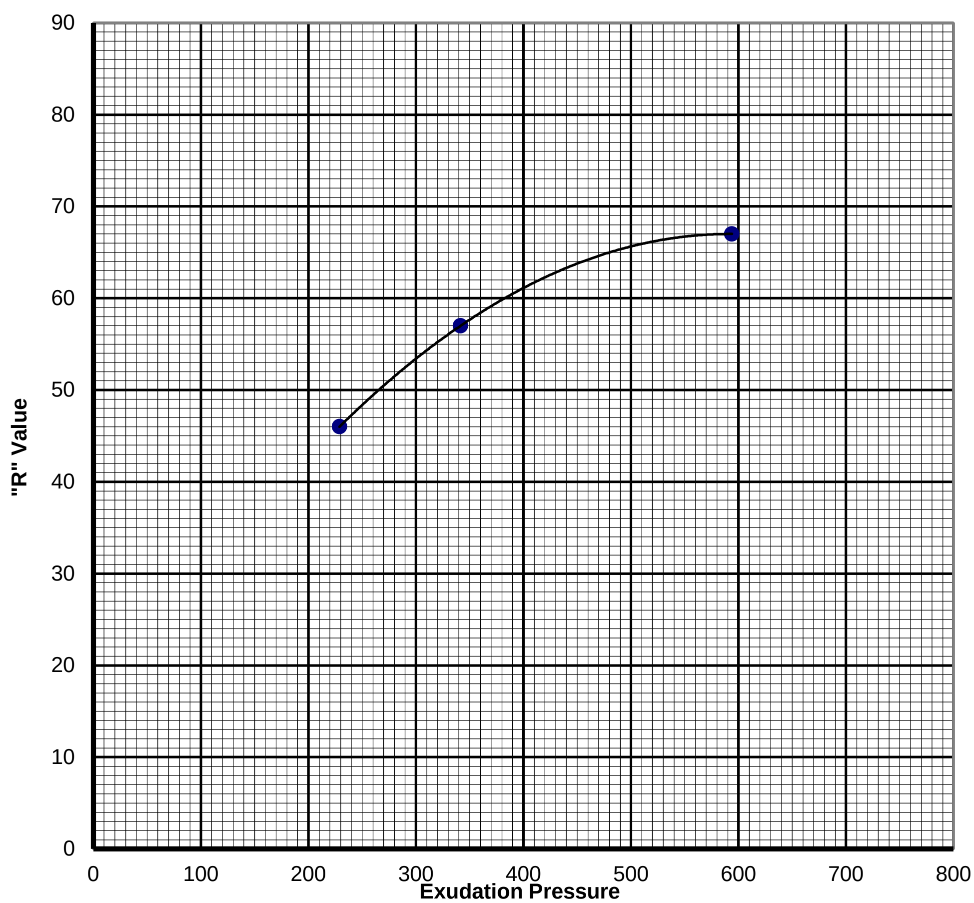
Client: Geobase **ATL No.:** C 0661-4 **Date:** 6/18/2017

Client Reference No. C3148100

Sample: B-10 @ 0 - 5' **Soil Type:** Brown, F.C. Silty Sand

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	200	100	300	
Initial Moisture Content	%	3.5	3.5	3.5	
Moisture at Compaction	%	9.1	10.0	8.3	
Briquette Height	in.	2.45	2.53	2.48	
Dry Density	pcf	127.7	124.1	122.5	
EXUDATION PRESSURE	psi	342	229	594	
EXPANSION dial	(x .0001)	5	0	11	
Ph at 1000 pounds	psi	25	31	19	
Ph at 2000 pounds	psi	48	61	38	
Displacement	turns	4.33	4.69	3.88	
"R" Value		57	46	67	
CORRECTED "R" VALUE		57	46	67	

Final "R" Value	
BY EXUDATION: @ 300 psi	**54**
BY EXPANSION: TI = 5.0	**N/A**



R - VALUE DATA SHEET

LaBelle Marvin

PROJECT No. 42535

DATE: 6/19/2017

BORING NO. B-8 @ 0'-5'

KP Moreno Medical Center

P.N. GBA-17-001 / C.314.81.00

SAMPLE DESCRIPTION: Brown Sandy Silt

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	10	11	12
Water added, grams	59	79	50
Initial Test Water, %	8.9	10.8	8.0
Compact Gage Pressure,psi	75	40	130
Exudation Pressure, psi	330	184	589
Height Sample, Inches	2.47	2.51	2.42
Gross Weight Mold, grams	3075	3098	3065
Tare Weight Mold, grams	1947	1953	1948
Sample Wet Weight, grams	1128	1145	1117
Expansion, Inches x 10exp-4	7	0	16
Stability 2,000 lbs (160psi)	19 / 39	42 / 97	18 / 34
Turns Displacement	4.82	5.02	4.60
R-Value Uncorrected	62	24	67
R-Value Corrected	62	24	66
Dry Density, pcf	127.1	124.7	129.5

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.39	0.78	0.35
G. E. by Expansion	0.23	0.00	0.53

Equilibrium R-Value	**59** by EXUDATION	Examined & Checked: 6 /19/ 17
REMARKS:	Gf = 1.25 0.0% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 CIVIL STATE OF CALIFORNIA Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 42535

DATE: 6 /19/ 16

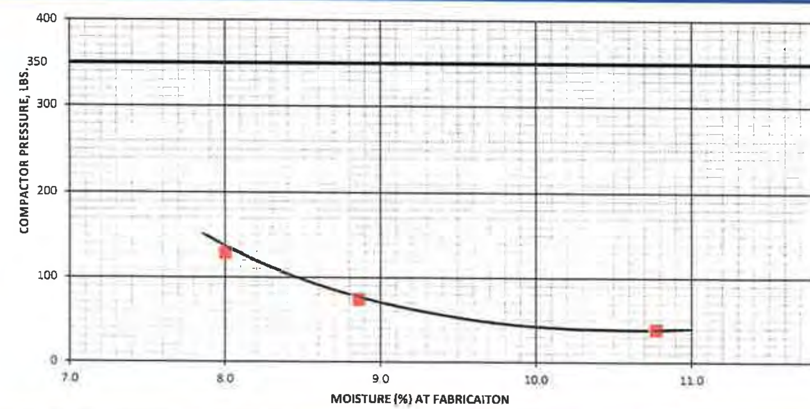
REMARKS:

BORING NO. B-8 @ 0'-5'

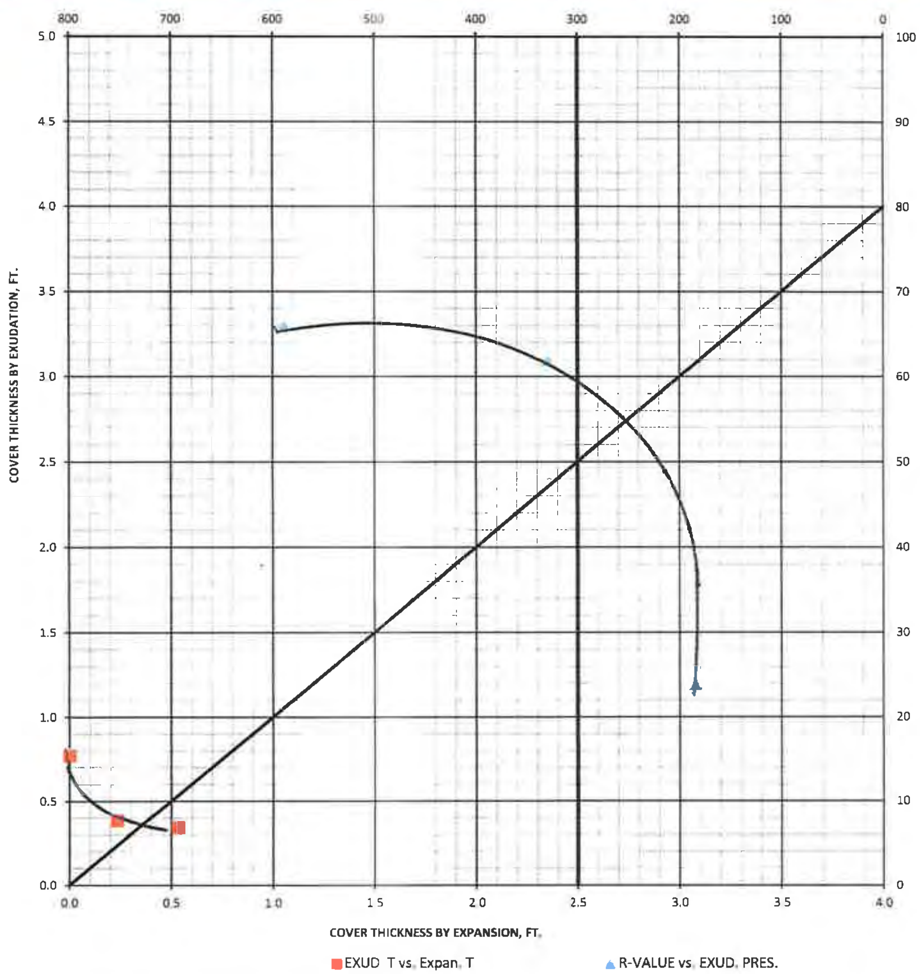
KP Moreno Medical Center

P.N. GBA-17-001 / C.314.81.00

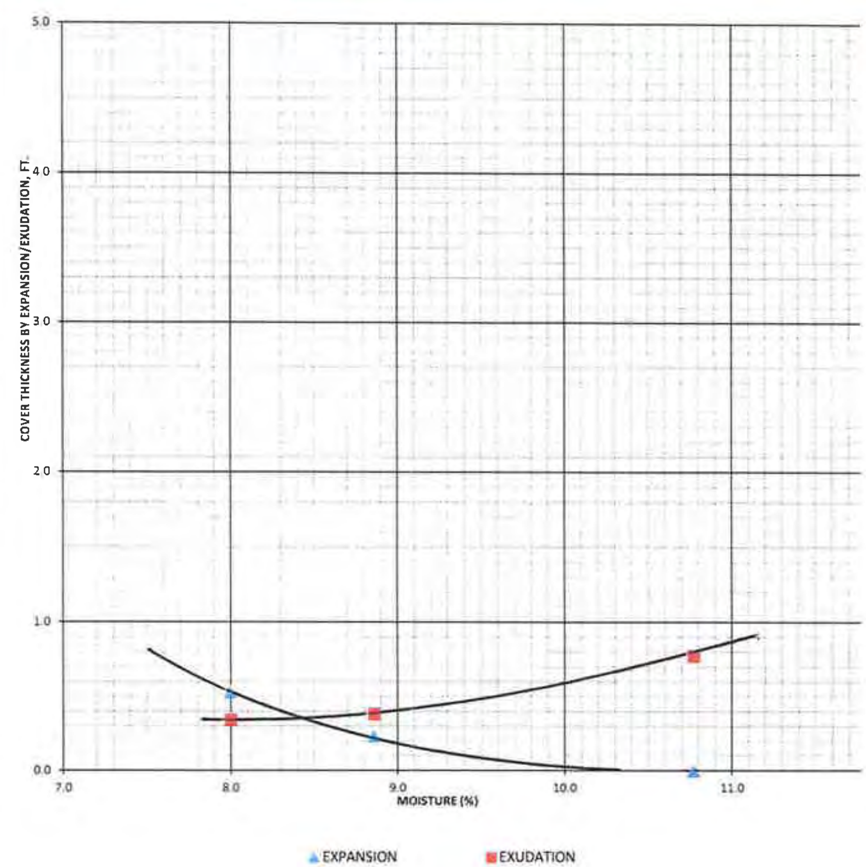
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %

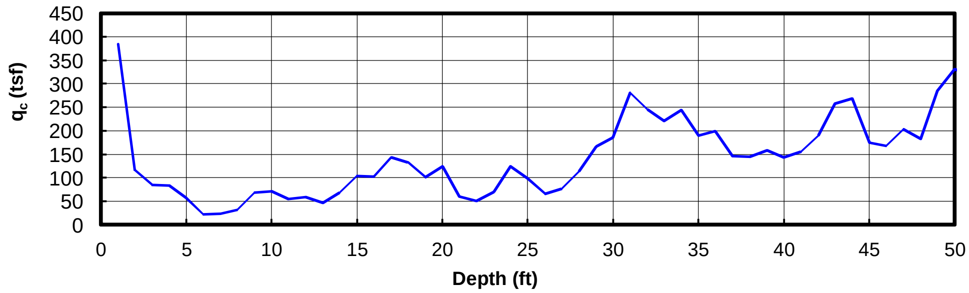


APPENDIX D

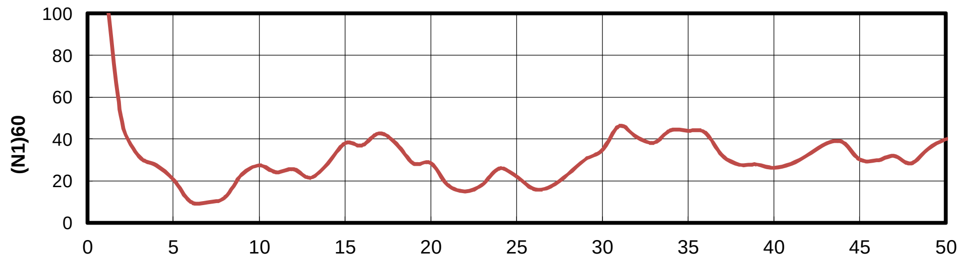
Figure D-1	Dry Seismic Settlement CPT-1
Figure D-2	Dry Seismic Settlement CPT-4

DRY SEISMIC SETTLEMENT ANALYSIS AT CPT-01

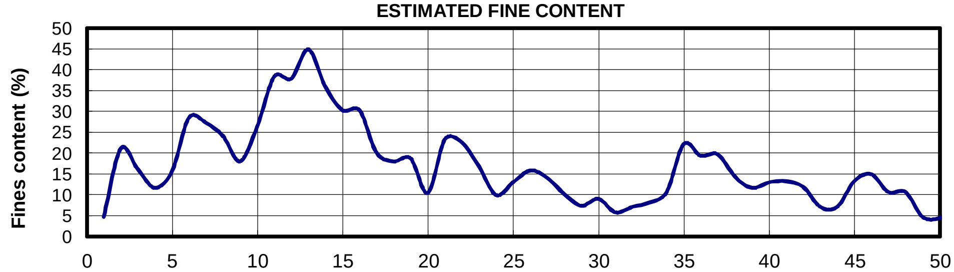
TIP RESISTANCE



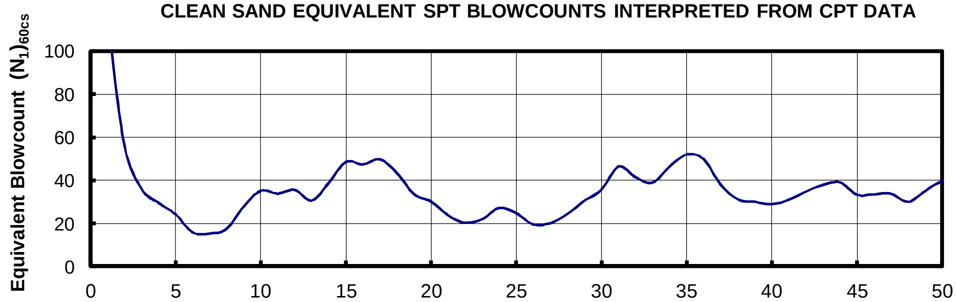
INTERPRETED SPT BLOWCOUNT (N1)60



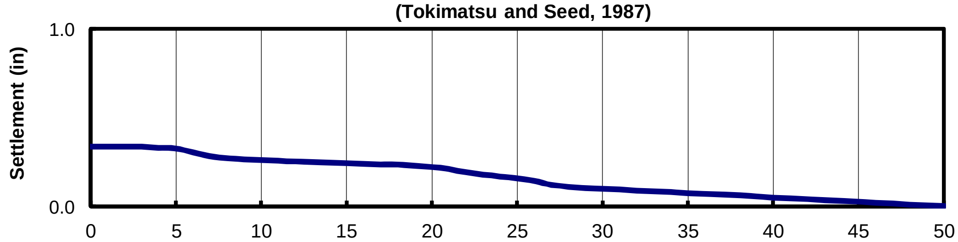
ESTIMATED FINE CONTENT



CLEAN SAND EQUIVALENT SPT BLOWCOUNTS INTERPRETED FROM CPT DATA



SEISMIC SETTLEMENT OF DRY SAND USING EQUIVALENT SPT
(Tokimatsu and Seed, 1987)



M_w	7.50
PHGA	0.657
MSF	1.00

P_{atm}	1.044271712
γ_{water}	62.42796058

From CPT DATA
Input
Results

Calculation of Clean sand equivalent $(N1)_{60cs}$ using CPT company's interpreted N_1

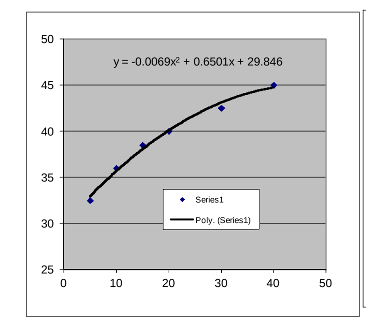
Depth (ft)	Provided CPT Readings q_c (avg) (tsf)	Provided CPT Readings f_s (avg) (tsf)	CPT-based SPT $(N_1)_{60}$ provided / interpreted by company	Measured / estimated fines content	Calculating $(N_1)60_{cs}$: α	Calculating $(N_1)60_{cs}$: β	Calculating $(N_1)60_{cs}$: $(N_1)60_{cs}$
0							
1	383.46	5.12	**116**	**5**	**0.00**	**1.00**	**116**
2	116.85	3.97	**48**	**21**	**3.79**	**1.09**	**56**
3	85.53	1.36	**32**	**16**	**2.80**	**1.05**	**36**
4	83.86	0.73	**27**	**12**	**1.43**	**1.03**	**30**
5	58.27	0.52	**21**	**16**	**2.64**	**1.05**	**24**
6	22.35	0.21	**10**	**28**	**4.60**	**1.14**	**16**
7	23.7	0.21	**10**	**27**	**4.50**	**1.13**	**15**
8	31.43	0.31	**12**	**24**	**4.18**	**1.11**	**17**
9	68.92	1.04	**23**	**18**	**3.23**	**1.07**	**28**
10	71.11	2.4	**27**	**26**	**4.43**	**1.13**	**35**
11	55.24	3.34	**24**	**38**	**5.00**	**1.20**	**34**
12	59.21	3.65	**25**	**38**	**5.00**	**1.20**	**36**
13	46.89	3.55	**21**	**45**	**5.00**	**1.20**	**31**
14	68.92	4.39	**28**	**36**	**5.00**	**1.20**	**39**
15	104.11	6.37	**38**	**30**	**4.72**	**1.16**	**49**
16	103.17	6.27	**37**	**30**	**4.72**	**1.16**	**47**
17	143.38	5.33	**43**	**20**	**3.62**	**1.08**	**50**
18	132.52	3.86	**38**	**18**	**3.23**	**1.07**	**43**
19	101.92	2.51	**28**	**19**	**3.37**	**1.07**	**34**
20	124.48	1.46	**28**	**10**	**1.00**	**1.02**	**30**
21	60.57	1.36	**18**	**23**	**4.09**	**1.10**	**24**
22	50.54	0.84	**15**	**23**	**4.00**	**1.10**	**20**
23	70.49	0.94	**18**	**17**	**2.95**	**1.06**	**22**
24	124.37	1.36	**26**	**10**	**0.84**	**1.02**	**27**
25	98.37	1.25	**22**	**13**	**1.87**	**1.04**	**25**
26	65.68	0.73	**16**	**16**	**2.72**	**1.05**	**20**
27	77.38	0.84	**17**	**14**	**2.22**	**1.04**	**20**
28	113.51	1.15	**23**	**10**	**0.92**	**1.02**	**24**
29	166.77	1.67	**30**	**7**	**0.16**	**1.01**	**31**
30	185.78	2.61	**35**	**9**	**0.56**	**1.02**	**36**
31	281.43	3.55	**46**	**6**	**0.02**	**1.00**	**46**
32	244.99	3.34	**41**	**7**	**0.13**	**1.01**	**41**
33	221.07	3.24	**38**	**8**	**0.33**	**1.01**	**39**
34	244.36	5.33	**44**	**11**	**1.08**	**1.02**	**46**
35	189.22	9.82	**44**	**22**	**3.95**	**1.09**	**52**
36	199.56	8.77	**43**	**19**	**3.50**	**1.07**	**50**
37	146.41	5.33	**32**	**20**	**3.56**	**1.08**	**38**
38	145.05	3.13	**28**	**14**	**2.31**	**1.04**	**31**
39	158.42	2.72	**28**	**12**	**1.43**	**1.03**	**30**
40	142.86	2.61	**26**	**13**	**1.87**	**1.04**	**29**
41	156.33	3.24	**28**	**13**	**1.96**	**1.04**	**31**
42	189.01	3.86	**32**	**12**	**1.52**	**1.03**	**35**
43	258.14	3.65	**37**	**7**	**0.13**	**1.01**	**38**
44	268.27	3.97	**39**	**7**	**0.13**	**1.01**	**39**
45	175.33	3.97	**30**	**13**	**1.96**	**1.04**	**33**
46	167.29	4.39	**30**	**15**	**2.48**	**1.05**	**34**
47	203.01	3.86	**32**	**11**	**1.08**	**1.02**	**34**
48	182.85	3.13	**28**	**11**	**1.08**	**1.02**	**30**
49	285.3	3.03	**35**	**5**	**0.00**	**1.00**	**35**
50	331.56	3.76	**40**	**4**	**0.00**	**1.00**	**40**
51	316.21	4.91	**41**	**7**	**0.08**	**1.01**	**41**

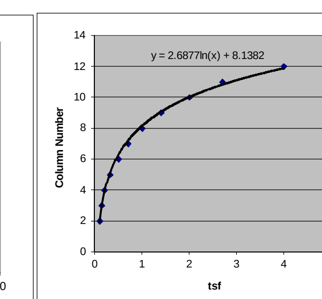
Dry sand seismic settlement calculations using the Tokimatsu and Seed (1987) method based on results

Depth of layer base(ft)	Layer thickness (ft)	Average layer depth (ft)	avg layer stress (tsf)	N1-60 from CPT company (col AQ of other spreadsheet)	**calculated** (N1-60)cs based on % fines (col BB of other spreadsheet)	(N1-60)cs used in calc <=60	K2-max	G-max	r-d	gama-e*(G-e/G-m)	gamma-e-calculated	epsln-c (M=7.5)-calculated
2	1.0	1.5	0.08	116	115.6	60.0	78	1004376	0.998472	7.00E-05	5.95E-05	2.36E-05
3	1.0	2.5	0.14	48	56.1	56.1	76	1268204.6	0.996146	9.22E-05	3.98E-05	1.38E-05
4	1.0	3.5	0.19	32	36.1	36.1	66	1295958.2	0.993765	1.26E-04	9.79E-05	3.62E-05
5	1.0	4.5	0.25	27	29.6	29.6	62	1374420.4	0.991399	1.52E-04	3.02E-04	1.50E-04
6	1.0	5.5	0.30	21	24.3	24.3	58	1422690.2	0.98907	1.80E-04	3.02E-04	2.11E-04
7	1.0	6.5	0.36	10	15.9	15.9	50	1343985.5	0.986786	2.24E-04	5.96E-04	8.52E-04
8	1.0	7.5	0.41	10	15.4	15.4	50	1427314.7	0.984549	2.43E-04	5.96E-04	8.99E-04
9	1.0	8.5	0.47	12	17.5	17.5	52	1585515.7	0.982355	2.47E-04	4.47E-04	5.50E-04
10	1.0	9.5	0.52	23	27.8	27.8	61	1957031.7	0.980196	2.24E-04	4.47E-04	2.46E-04
11	1.0	10.5	0.58	27	35.1	35.1	65	2223473.6	0.978066	2.17E-04	4.47E-04	1.71E-04
12	1.0	11.5	0.63	24	33.8	33.8	65	2297697.6	0.975954	2.29E-04	4.47E-04	1.80E-04
13	0.5	12.75	0.70	25	35.6	35.6	66	2460639.5	0.973323	2.37E-04	4.03E-04	1.52E-04
14	1.0	13.5	0.74	21	30.6	30.6	63	2408882.6	0.97174	2.56E-04	4.03E-04	1.89E-04
15	1.0	14.5	0.80	28	38.9	38.9	68	2703588.9	0.969612	2.44E-04	4.03E-04	1.37E-04
16	1.0	15.5	0.85	38	48.5	48.5	73	3008316.2	0.967451	2.34E-04	4.03E-04	1.23E-04
17	1.0	16.5	0.91	37	47.4	47.4	72	3078681.9	0.965241	2.43E-04	4.03E-04	1.23E-04
18	1.0	17.5	0.96	43	49.7	49.7	73	3222525.5	0.962965	2.46E-04	3.63E-04	1.12E-04
19	1.0	18.5	1.02	38	43.3	43.3	70	3164858.8	0.960604	2.64E-04	3.63E-04	1.13E-04
20	1.0	19.5	1.07	28	33.6	33.6	64	2984613.3	0.958139	2.94E-04	5.85E-04	2.39E-04
21	1.0	20.5	1.13	28	30.1	30.1	62	2951684.3	0.955551	3.12E-04	5.85E-04	2.82E-04
22	1.0	21.5	1.18	18	23.9	23.9	58	2796791.3	0.952817	3.44E-04	7.26E-04	5.23E-04
23	1.0	22.5	1.24	15	20.3	20.3	55	2712764.5	0.949915	3.70E-04	7.26E-04	6.92E-04
24	1.0	23.5	1.29	18	22.0	22.0	56	2845472.2	0.946823	3.67E-04	7.26E-04	6.04E-04
25	1.0	24.5	1.35	26	27.3	27.3	60	3121302.7	0.943516	3.48E-04	7.26E-04	4.14E-04
26	1.0	25.5	1.40	22	24.7	24.7	58	3082711	0.939973	3.65E-04	6.03E-04	4.08E-04
27	1.0	26.5	1.46	16	19.6	19.6	54	2906311.5	0.936169	4.01E-04	6.03E-04	6.15E-04
28	1.0	27.5	1.51	17	20.2	20.2	54	2992242.2	0.932081	4.02E-04	9.54E-04	9.20E-04
29	1.0	28.5	1.57	23	24.5	24.5	58	3247236.3	0.927687	3.82E-04	6.03E-04	4.16E-04
30	1.0	29.5	1.62	30	30.6	30.6	62	3557622.2	0.922967	3.60E-04	6.03E-04	2.84E-04
31	1.0	30.5	1.68	35	35.7	35.7	66	3810502.8	0.917901	3.45E-04	6.03E-04	2.26E-04
32	1.0	31.5	1.73	46	46.4	46.4	72	4224580.9	0.912474	3.20E-04	6.03E-04	1.84E-04
33	1.0	32.5	1.79	41	41.5	41.5	69	4133558.4	0.90667	3.35E-04	6.03E-04	1.94E-04
34	1.0	33.5	1.84	38	39.0	39.0	68	4110985.6	0.90048	3.45E-04	6.03E-04	2.04E-04
35	1.0	34.5	1.90	44	46.3	46.3	72	4418900.5	0.893898	3.28E-04	6.03E-04	1.84E-04
36	1.0	35.5	1.95	44	52.0	52.0	75	4658636.1	0.88692	3.17E-04	6.03E-04	1.91E-04
37	1.0	36.5	2.01	43	49.8	49.8	73	4655193.1	0.879549	3.24E-04	5.01E-04	1.55E-04
38	1.0	37.5	2.06	32	38.0	38.0	67	4313158.6	0.871792	3.56E-04	5.01E-04	1.75E-04
39	1.0	38.5	2.12	28	31.2	31.2	63	4092881.5	0.863661	3.82E-04	5.01E-04	2.29E-04
40	1.0	39.5	2.17	28	30.0	30.0	62	4092623.5	0.855174	3.88E-04	5.01E-04	2.43E-04
41	1.0	40.5	2.23	26	28.9	28.9	61	4093866	0.846353	3.93E-04	5.01E-04	2.58E-04
42	1.0	41.5	2.28	28	31.1	31.1	63	4242373.9	0.837224	3.85E-04	5.01E-04	2.30E-04
43	1.0	42.5	2.34	32	34.8	34.8	65	4459635.6	0.827819	3.71E-04	5.01E-04	1.94E-04
44	1.0	43.5	2.39	37	38.0	38.0	67	4643761.9	0.818172	3.60E-04	5.01E-04	1.75E-04
45	1.0	44.5	2.45	39	39.0	39.0	68	4740207.7	0.808323	3.56E-04	5.01E-04	1.70E-04
46	1.0	45.5	2.50	30	33.3	33.3	64	4545450.4	0.798311	3.75E-04	5.01E-04	2.07E-04
47	1.0	46.5	2.56	30	33.5	33.5	64	4607707.8	0.788178	3.74E-04	5.01E-04	2.05E-04
48	1.0	47.5	2.61	32	33.7	33.7	65	4666001.2	0.777967	3.72E-04	5.01E-04	2.03E-04
49	1.0	48.5	2.67	28	30.0	30.0	62	4534511.7	0.767722	3.86E-04	5.01E-04	2.43E-04
50	1.0	49.5	2.72	35	35.1	35.1	65	4827135	0.757484	3.65E-04	5.01E-04	1.92E-04
51	1.0	50.5	2.78	40	39.7	39.7	68	5079952.5	0.747293	3.49E-04	5.01E-04	1.67E-04
52	1.0	51.5	2.83	41	41.1	41.1	69	5188681.4	0.737189	3.44E-04	5.01E-04	1.62E-04

Depth of layer base(ft)	epsln-c (Mw)	2*epsln-c (Mw)	settlement (in)	**Cumulative settlement (inch)**	**Depth (ft) of top of layer**	**sig-v tsf**	**column Number**	**x**	**y**	gamma-e-calculated	gamma-e-read from curve	epsilon-c-calculated	epsilon-c-read off cur
2	2.36E-05	4.73E-05	0.00	0.34	**1.0**	**0.08**	**1**	**8.5**	**7.7**	**5.9E-05**	**-1.1E+01**	**2.4E-05**	
3	1.38E-05	2.77E-05	0.00	0.34	**2.0**	**0.14**	**3**	**9.6**	**6.0**	**4.0E-05**	**-1.0E+01**	**1.4E-05**	
4	3.62E-05	7.23E-05	0.00	0.34	**3.0**	**0.19**	**1**	**11.0**	**9.9**	**9.8E-05**	**-9.0E+00**	**3.6E-05**	
5	1.50E-04	3.01E-04	0.00	0.33	**4.0**	**0.25**	**4**	**11.8**	**14.8**	**3.0E-04**	**-8.0E+00**	**1.5E-04**	
6	2.11E-04	4.22E-04	0.01	0.33	**5.0**	**0.30**	**5**	**12.5**	**14.8**	**3.0E-04**	**-7.0E+00**	**2.1E-04**	
7	8.53E-04	1.71E-03	0.02	0.33	**6.0**	**0.36**	**5**	**13.5**	**17.8**	**6.0E-04**	**-6.0E+00**	**8.5E-04**	
8	9.00E-04	1.80E-03	0.02	0.31	**7.0**	**0.41**	**6**	**13.9**	**17.8**	**6.0E-04**	**-5.0E+00**	**9.0E-04**	
9	5.50E-04	1.10E-03	0.01	0.28	**8.0**	**0.47**	**6**	**13.9**	**16.5**	**4.5E-04**	**-4.0E+00**	**5.5E-04**	
10	2.46E-04	4.92E-04	0.01	0.27	**9.0**	**0.52**	**6**	**13.5**	**16.5**	**4.5E-04**	**-3.0E+00**	**2.5E-04**	
11	1.71E-04	3.43E-04	0.00	0.26	**10.0**	**0.58**	**7**	**13.4**	**16.5**	**4.5E-04**	**-2.0E+00**	**1.7E-04**	
12	1.80E-04	3.61E-04	0.00	0.26	**11.0**	**0.63**	**7**	**13.6**	**16.5**	**4.5E-04**	**-1.0E+00**	**1.8E-04**	
13	1.52E-04	3.04E-04	0.00	0.26	**12.5**	**0.70**	**7**	**13.7**	**16.1**	**4.0E-04**	**4.0E-03**	**1.5E-04**	
14	1.89E-04	3.78E-04	0.00	0.25	**13.0**	**0.74**	**7**	**14.1**	**16.1**	**4.0E-04**	**7.0E-04**	**1.9E-04**	
15	1.37E-04	2.74E-04	0.00	0.25	**14.0**	**0.80**	**8**	**13.9**	**16.1**	**4.0E-04**	**1.9E-03**	**1.4E-04**	
16	1.23E-04	2.47E-04	0.00	0.25	**15.0**	**0.85**	**8**	**13.7**	**16.1**	**4.0E-04**	**6.0E-03**	**1.2E-04**	
17	1.23E-04	2.46E-04	0.00	0.24	**16.0**	**0.91**	**8**	**13.9**	**16.1**	**4.0E-04**	**2.2E-03**	**1.2E-04**	
18	1.12E-04	2.24E-04	0.00	0.24	**17.0**	**0.96**	**8**	**13.9**	**15.6**	**3.6E-04**	**1.0E+00**	**1.1E-04**	
19	1.13E-04	2.27E-04	0.00	0.24	**18.0**	**1.02**	**8**	**14.2**	**15.6**	**3.6E-04**	**2.0E+00**	**1.1E-04**	
20	2.39E-04	4.78E-04	0.01	0.23	**19.0**	**1.07**	**8**	**14.7**	**17.7**	**5.8E-04**	**3.0E+00**	**2.4E-04**	
21	2.82E-04	5.65E-04	0.01	0.23	**20.0**	**1.13**	**8**	**14.9**	**17.7**	**5.8E-04**	**4.0E+00**	**2.8E-04**	
22	5.23E-04	1.05E-03	0.01	0.22	**21.0**	**1.18**	**9**	**15.4**	**18.6**	**7.3E-04**	**5.0E+00**	**5.2E-04**	
23	6.92E-04	1.38E-03	0.02	0.21	**22.0**	**1.24**	**9**	**15.7**	**18.6**	**7.3E-04**	**6.0E+00**	**6.9E-04**	
24	6.04E-04	1.21E-03	0.01	0.19	**23.0**	**1.29**	**9**	**15.7**	**18.6**	**7.3E-04**	**7.0E+00**	**6.0E-04**	
25	4.14E-04	8.28E-04	0.01	0.18	**24.0**	**1.35**	**9**	**15.4**	**18.6**	**7.3E-04**	**8.0E+00**	**4.1E-04**	
26	4.08E-04	8.16E-04	0.01	0.17	**25.0**	**1.40**	**9**	**15.6**	**17.8**	**6.0E-04**	**9.0E+00**	**4.1E-04**	
27	6.15E-04	1.23E-03	0.01	0.16	**26.0**	**1.46**	**9**	**16.0**	**17.8**	**6.0E-04**	**1.0E+01**	**6.1E-04**	
28	9.21E-04	1.84E-03	0.02	0.14	**27.0**	**1.51**	**9**	**16.0**	**19.8**	**9.5E-04**	**1.1E+01**	**9.2E-04**	
29	4.16E-04	8.31E-04	0.01	0.12	**28.0**	**1.57**	**9**	**15.8**	**17.8**	**6.0E-04**	**1.2E+01**	**4.2E-04**	
30	2.85E-04	5.69E-04	0.01	0.11	**29.0**	**1.62**	**9**	**15.6**	**17.8**	**6.0E-04**	**1.3E+01**	**2.8E-04**	
31	2.26E-04	4.52E-04	0.01	0.11	**30.0**	**1.68**	**10**	**15.4**	**17.8**	**6.0E-04**	**1.4E+01**	**2.3E-04**	
32	1.84E-04	3.69E-04	0.00	0.10	**31.0**	**1.73**	**10**	**15.0**	**17.8**	**6.0E-04**	**1.5E+01**	**1.8E-04**	
33	1.94E-04	3.87E-04	0.00	0.10	**32.0**	**1.79**	**10**	**15.2**	**17.8**	**6.0E-04**	**1.6E+01**	**1.9E-04**	
34	2.05E-04	4.09E-04	0.00	0.09	**33.0**	**1.84**	**10**	**15.4**	**17.8**	**6.0E-04**	**1.7E+01**	**2.0E-04**	
35	1.84E-04	3.69E-04	0.00	0.09	**34.0**	**1.90**	**10**	**15.2**	**17.8**	**6.0E-04**	**1.8E+01**	**1.8E-04**	
36	1.91E-04	3.83E-04	0.00	0.08	**35.0**	**1.95**	**10**	**15.0**	**17.8**	**6.0E-04**	**1.9E+01**	**1.9E-04**	
37	1.55E-04	3.10E-04	0.00	0.08	**36.0**	**2.01**	**10**	**15.1**	**17.0**	**5.0E-04**	**2.0E+01**	**1.5E-04**	
38	1.75E-04	3.49E-04	0.00	0.07	**37.0**	**2.06**	**10**	**15.5**	**17.0**	**5.0E-04**	**2.1E+01**	**1.7E-04**	
39	2.29E-04	4.57E-04	0.01	0.07	**38.0**	**2.12**	**10**	**15.8**	**17.0**	**5.0E-04**	**2.2E+01**	**2.3E-04**	
40	2.43E-04	4.87E-04	0.01	0.06	**39.0**	**2.17**	**10**	**15.9**	**17.0**	**5.0E-04**	**2.3E+01**	**2.4E-04**	
41	2.58E-04	5.17E-04	0.01	0.06	**40.0**	**2.23**	**10**	**15.9**	**17.0**	**5.0E-04**	**2.4E+01**	**2.6E-04**	
42	2.30E-04	4.61E-04	0.01	0.05	**41.0**	**2.28**	**10**	**15.9**	**17.0**	**5.0E-04**	**2.5E+01**	**2.3E-04**	
43	1.95E-04	3.89E-04	0.00	0.05	**42.0**	**2.34**	**10**	**15.7**	**17.0**	**5.0E-04**	**2.6E+01**	**1.9E-04**	
44	1.75E-04	3.50E-04	0.00	0.04	**43.0**	**2.39**	**10**	**15.6**	**17.0**	**5.0E-04**	**2.7E+01**	**1.7E-04**	
45	1.70E-04	3.40E-04	0.00	0.04	**44.0**	**2.45**	**11**	**15.5**	**17.0**	**5.0E-04**	**2.8E+01**	**1.7E-04**	
46	2.07E-04	4.15E-04	0.00	0.03	**45.0**	**2.50**	**11**	**15.7**	**17.0**	**5.0E-04**	**2.9E+01**	**2.1E-04**	
47	2.05E-04	4.10E-04	0.00	0.03	**46.0**	**2.56**	**11**	**15.7**	**17.0**	**5.0E-04**	**3.0E+01**	**2.0E-04**	
48	2.03E-04	4.06E-04	0.00	0.02	**47.0**	**2.61**	**11**	**15.7**	**17.0**	**5.0E-04**	**3.1E+01**	**2.0E-04**	
49	2.43E-04	4.87E-04	0.01	0.02	**48.0**	**2.67**	**11**	**15.9**	**17.0**	**5.0E-04**	**3.2E+01**	**2.4E-04**	
50	1.92E-04	3.84E-04	0.00	0.01	**49.0**	**2.72**	**11**	**15.6**	**17.0**	**5.0E-04**	**3.3E+01**	**1.9E-04**	
51	1.67E-04	3.34E-04	0.00	0.01	**50.0**	**2.78**	**11**	**15.4**	**17.0**	**5.0E-04**	**3.4E+01**	**1.7E-04**	
52	1.62E-04	3.24E-04	0.00	0.00	**51.0**	**2.83**	**11**	**15.4**	**17.0**	**5.0E-04**	**3.5E+01**	**1.6E-04**	

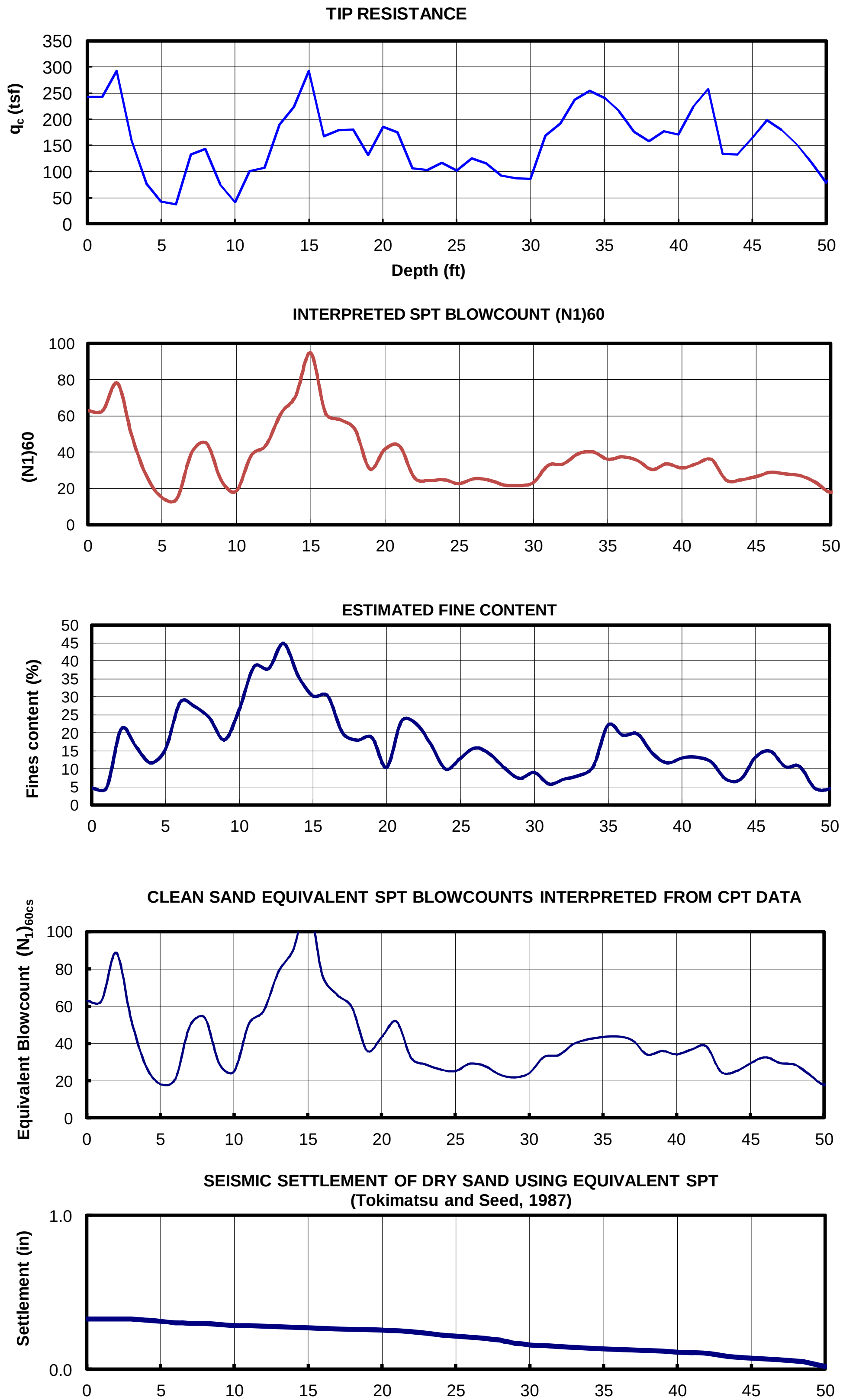
seismic Settlement (inch) col No	1	2	3	4	5	6	7	8	9	10	11	12
overburden		0.1 tsf		0.2 tsf		0.5 tsf		1 tsf		2 tsf		4 tsf
		0.1	0.14	0.2	0.32	0.5	0.7	1	1.4	2	2.7	4
	-0.06	-0.07	-0.07125	-0.07	-0.07125	-0.07	-0.03562	0.00	3.41E-15	0.00	3.41E-15	0.00
	2.97	3.17	3.1725	3.17	3.21	3.25	3.21	3.17	3.245625	3.32	3.31875	3.32
	4.71	5.12	5.1225	5.12	5.158125	5.19	5.158125	5.12	5.158125	5.19	5.19375	5.19
	6.01	6.85	6.78	6.71	6.70875	6.71	6.6	6.49	6.49125	6.49	6.49125	6.49
	6.94	8.15	8.004375	7.86	7.824375	7.79	7.68	7.57	7.60875	7.65	7.64625	7.65
	7.74	9.23	9.0525	8.87	8.835	8.80	8.653125	8.51	8.473125	8.44	8.4375	8.44
	9.00	11.25	10.875	10.50	10.3	10.10	10.05	10.00	10	10.00	10	10.00
	9.91	12.91	12.3675	11.83	11.53875	11.25	11.10563	10.96	10.81688	10.67	10.6725	10.67
	11.50	17.00	15.9	14.80	14.1425	13.49	13.2425	13.00	12.75	12.50	12.5	12.50
	13.00	24.59	21.49125	18.39	17.16375	15.94	15.36	14.78	14.53125	14.28	14.27899	14.28
	13.30	29.86	24.42875	19.00	17.75	16.50	16.05	15.60	15.3	15.00	14.996	14.99
	14.68		33	26.18	22.93313	19.69	18.67875	17.67	17.20125	16.73	16.65205	16.57
	15.00			29.78	25.45688	21.13	19.86938	18.61	17.80375	17.00	16.87793	16.76
	16.04				28	24.59	22.57313	20.55	19.79625	19.04	18.69297	18.35
	16.97					29.86	26.68313	23.51	22.28438	21.06	20.36335	19.67
	17.65						28	26.32	24.555	22.79	21.7236	20.66
	18.39							30.00	27.51188	25.02	23.42701	21.83
	18.95								30	27.12	25.02245	22.93
	19.57									29.93	27.17323	24.42
	21											30





DRY SEISMIC SETTLEMENT ANALYSIS AT CPT-4

FIGURE D-2



GEOBASE,INC.

M_w	7.50
PHGA	0.657
MSF	1.00

P_{atm}	1.044271712
γ_{water}	62.42796058

From CPT DATA
Input
Results

Calculation of Clean sand equivalent $(N1)_{60cs}$ using CPT company's interpreted N_1

Depth (ft)	Provided CPT Readings: q_c (avg) (tsf)	f_s (avg) (tsf)	CPT-based SPT $(N_1)_{60}$ provided / interpreted by company	Measured / estimated fines content	Calculating $(N_1)60_{cs}$: α	β	$(N_1)60_{cs}$
0	242.58		63	5			63
1	242.58	1.25	63	5	0.00	1.00	63
2	292.92	1.88	78	21	3.79	1.09	89
3	159.15	1.46	48	16	2.80	1.05	53
4	76.55	0.84	26	12	1.43	1.03	28
5	42.4	0.31	15	16	2.64	1.05	18
6	37.8	0.31	14	28	4.60	1.14	21
7	132.52	1.57	40	27	4.50	1.13	50
8	142.96	2.72	45	24	4.18	1.11	54
9	75.08	1.04	24	18	3.23	1.07	29
10	41.35	1.67	18	26	4.43	1.13	25
11	100.67	4.7	38	38	5.00	1.20	51
12	107.77	7.62	44	38	5.00	1.20	58
13	189.85	7.94	61	45	5.00	1.20	78
14	224	11.17	71	36	5.00	1.20	90
15	292.61	19.63	95	30	4.72	1.16	114
16	167.29	15.46	62	30	4.72	1.16	76
17	179.72	11.07	58	20	3.62	1.08	66
18	180.76	7.62	52	18	3.23	1.07	59
19	131.58	1.78	31	19	3.37	1.07	36
20	185.36	3.13	42	10	1.00	1.02	44
21	174.71	4.59	43	23	4.09	1.10	52
22	106.2	1.88	26	23	4.00	1.10	32
23	102.86	1.57	24	17	2.95	1.06	29
24	117.06	1.15	25	10	0.84	1.02	26
25	102.55	1.25	23	13	1.87	1.04	25
26	124.79	1.46	25	16	2.72	1.05	29
27	115.81	1.67	25	14	2.22	1.04	28
28	92.21	1.88	22	10	0.92	1.02	23
29	86.78	2.19	21	7	0.16	1.01	22
30	85.84	3.45	23	9	0.56	1.02	24
31	168.44	3.03	33	6	0.02	1.00	33
32	191.52	2.61	33	7	0.13	1.01	34
33	237.05	3.03	39	8	0.33	1.01	40
34	254.49	3.24	40	11	1.08	1.02	42
35	240.18	2.51	36	22	3.95	1.09	43
36	215.22	4.07	37	19	3.50	1.07	44
37	176.48	5.64	35	20	3.56	1.08	42
38	158	3.97	30	14	2.31	1.04	34
39	176.8	4.8	34	12	1.43	1.03	36
40	171.16	3.97	31	13	1.87	1.04	34
41	223.68	2.92	34	13	1.96	1.04	37
42	257.83	3.13	36	12	1.52	1.03	39
43	133.77	2.82	24	7	0.13	1.01	25
44	132.62	3.24	25	7	0.13	1.01	25
45	164.16	2.82	26	13	1.96	1.04	29
46	197.89	2.51	29	15	2.48	1.05	33
47	179.3	3.03	28	11	1.08	1.02	30
48	152.05	3.86	27	11	1.08	1.02	29
49	118.42	3.86	23	5	0.00	1.00	23
50	78.32	3.86	18	4	0.00	1.00	18
51	111.53	3.65	22	7	0.08	1.01	22

Dry sand seismic settlement calculations using the Tokimatsu and Seed (1987) method based on results

Depth of layer base(ft)	Layer thickness (ft)	Average layer depth (ft)	avg layer stress (tsf)	N1-60 from CPT company (col AQ of other spreadsheet)	calculated (N1-60)cs based on % fines (col BB of other spreadsheet)	(N1-60)cs used in calc <=60	K2-max	G-max	r-d	gama-e*(G-e/G-m)	gamma-e-calculated
2	1.0	1.5	0.08	63	62.9	60.0	78	1004376	0.998472	7.00E-05	5.95E-05
3	1.0	2.5	0.14	78	88.6	60.0	78	1296643.9	0.996146	9.02E-05	3.98E-05
4	1.0	3.5	0.19	48	53.2	53.2	75	1473660.2	0.993765	1.11E-04	9.79E-05
5	1.0	4.5	0.25	26	28.2	28.2	61	1352613.6	0.991399	1.55E-04	3.02E-04
6	1.0	5.5	0.30	15	18.2	18.2	53	1292407.2	0.98907	1.98E-04	3.02E-04
7	1.0	6.5	0.36	14	20.9	20.9	55	1470811	0.986786	2.05E-04	5.20E-04
8	1.0	7.5	0.41	40	49.6	49.6	73	2107636.2	0.984549	1.65E-04	2.60E-04
9	1.0	8.5	0.47	45	53.8	53.8	75	2305976.1	0.982355	1.70E-04	2.23E-04
10	1.0	9.5	0.52	24	29.0	29.0	61	1985247.7	0.980196	2.20E-04	4.47E-04
11	1.0	10.5	0.58	18	25.2	25.2	59	1991887.2	0.978066	2.42E-04	4.47E-04
12	1.0	11.5	0.63	38	50.6	50.6	74	2628210.4	0.975954	2.01E-04	3.92E-04
13	0.5	12.75	0.70	44	57.6	57.6	77	2888051.7	0.973323	2.02E-04	3.44E-04
14	1.0	13.5	0.74	61	78.5	60.0	78	3013128.1	0.97174	2.05E-04	3.44E-04
15	1.0	14.5	0.80	71	90.3	60.0	78	3122731.9	0.969612	2.11E-04	3.44E-04
16	1.0	15.5	0.85	95	114.4	60.0	78	3228617.2	0.967451	2.18E-04	4.03E-04
17	1.0	16.5	0.91	62	76.0	60.0	78	3331138.4	0.965241	2.25E-04	4.03E-04
18	1.0	17.5	0.96	58	66.1	60.0	78	3430597.2	0.962965	2.31E-04	3.63E-04
19	1.0	18.5	1.02	52	59.1	59.1	78	3509011.1	0.960604	2.38E-04	3.63E-04
20	1.0	19.5	1.07	31	36.2	36.2	66	3060166.5	0.958139	2.87E-04	3.63E-04
21	1.0	20.5	1.13	42	43.5	43.5	70	3336790.6	0.955551	2.76E-04	3.63E-04
22	1.0	21.5	1.18	43	51.8	51.8	74	3619956.6	0.952817	2.66E-04	3.63E-04
23	1.0	22.5	1.24	26	32.3	32.3	64	3164327.2	0.949915	3.17E-04	7.26E-04
24	1.0	23.5	1.29	24	28.7	28.7	61	3111591.4	0.946823	3.36E-04	7.26E-04
25	1.0	24.5	1.35	25	26.0	26.0	59	3074139	0.943516	3.53E-04	7.26E-04
26	1.0	25.5	1.40	23	25.2	25.2	59	3103911.8	0.939973	3.63E-04	6.03E-04
27	1.0	26.5	1.46	25	29.3	29.3	62	3324743.8	0.936169	3.51E-04	6.03E-04
28	1.0	27.5	1.51	25	27.9	27.9	61	3333286.3	0.932081	3.61E-04	6.03E-04
29	1.0	28.5	1.57	22	23.3	23.3	57	3195188.3	0.927687	3.89E-04	6.03E-04
30	1.0	29.5	1.62	21	21.8	21.8	56	3181355.7	0.922967	4.02E-04	9.54E-04
31	1.0	30.5	1.68	23	24.1	24.1	58	3343453.7	0.917901	3.93E-04	6.03E-04
32	1.0	31.5	1.73	33	32.8	32.8	64	3765249.9	0.912474	3.59E-04	6.03E-04
33	1.0	32.5	1.79	33	33.8	33.8	65	3862377.4	0.90667	3.58E-04	6.03E-04
34	1.0	33.5	1.84	39	39.8	39.8	68	4139238.4	0.90048	3.42E-04	6.03E-04
35	1.0	34.5	1.90	40	42.2	42.2	70	4285015.6	0.893898	3.38E-04	6.03E-04
36	1.0	35.5	1.95	36	43.4	43.4	70	4387114.7	0.88692	3.37E-04	6.03E-04
37	1.0	36.5	2.01	37	43.7	43.7	70	4458029.3	0.879549	3.38E-04	5.01E-04
38	1.0	37.5	2.06	35	41.7	41.7	69	4448994.1	0.871792	3.45E-04	5.01E-04
39	1.0	38.5	2.12	30	34.1	34.1	65	4213721.5	0.863661	3.71E-04	5.01E-04
40	1.0	39.5	2.17	34	36.0	36.0	66	4347926.6	0.855174	3.65E-04	5.01E-04
41	1.0	40.5	2.23	31	34.2	34.2	65	4327073.4	0.846353	3.72E-04	5.01E-04
42	1.0	41.5	2.28	34	36.8	36.8	66	4488799.9	0.837224	3.64E-04	5.01E-04
43	1.0	42.5	2.34	36	38.6	38.6	67	4614743.7	0.827819	3.58E-04	5.01E-04
44	1.0	43.5	2.39	24	24.8	24.8	58	4032533	0.818172	4.15E-04	8.01E-04
45	1.0	44.5	2.45	25	25.0	25.0	58	4089743.8	0.808323	4.13E-04	8.01E-04
46	1.0	45.5	2.50	26	29.5	29.5	62	4364881.5	0.798311	3.91E-04	5.01E-04
47	1.0	46.5	2.56	29	32.7	32.7	64	4567834.2	0.788178	3.77E-04	5.01E-04
48	1.0	47.5	2.61	28	29.6	29.6	62	4466831.6	0.777967	3.89E-04	5.01E-04
49	1.0	48.5	2.67	27	28.7	28.7	61	4465677.2	0.767722	3.92E-04	5.01E-04
50	1.0	49.5	2.72	23	23.4	23.4	57	4215144.8	0.757484	4.18E-04	8.01E-04
51	1.0	50.5	2.78	18	17.9	17.9	52	3895288.7	0.747293	4.55E-04	8.01E-04
52	1.0	51.5	2.83	22	21.9	21.9	56	4209730.3	0.737189	4.24E-04	8.01E-04

Depth of layer base(ft)	epsln-c (M=7.5)-calculated	epsln-c (Mw)	2*epsln-c (Mw)	settlement (in)	Cumulative settlement (inch)	Depth (ft) of top of layer	sig-v tsf	column Number	x	y	gamma-e-calculated	gamma-e-read from curve	epsilon-c-calculated	epsilon-c-read off cur
						0								
2	2.36E-05	2.36E-05	4.73E-05	0.00	0.33	1.0	0.08	1	8.5	7.7	5.9E-05	-1.1E+01	2.4E-05	
3	1.58E-05	1.58E-05	3.17E-05	0.00	0.33	2.0	0.14	3	9.6	6.0	4.0E-05	-1.0E+01	1.6E-05	
4	3.17E-05	3.17E-05	6.34E-05	0.00	0.33	3.0	0.19	1	10.4	9.9	9.8E-05	-9.0E+00	3.2E-05	
5	1.63E-04	1.63E-04	3.26E-04	0.00	0.32	4.0	0.25	4	11.9	14.8	3.0E-04	-8.0E+00	1.6E-04	
6	3.48E-04	3.48E-04	6.96E-04	0.01	0.32	5.0	0.30	5	13.0	14.8	3.0E-04	-7.0E+00	3.5E-04	
7	4.74E-04	4.74E-04	9.49E-04	0.01	0.31	6.0	0.36	5	13.1	17.2	5.2E-04	-6.0E+00	4.7E-04	
8	8.00E-05	8.00E-05	1.60E-04	0.00	0.30	7.0	0.41	6	12.2	14.1	2.6E-04	-5.0E+00	8.0E-05	
9	7.32E-05	7.32E-05	1.46E-04	0.00	0.30	8.0	0.47	6	12.3	13.5	2.2E-04	-4.0E+00	7.3E-05	
10	2.29E-04	2.29E-04	4.58E-04	0.01	0.30	9.0	0.52	6	13.4	16.5	4.5E-04	-3.0E+00	2.3E-04	
11	2.91E-04	2.91E-04	5.83E-04	0.01	0.29	10.0	0.58	7	13.8	16.5	4.5E-04	-2.0E+00	2.9E-04	
12	1.22E-04	1.22E-04	2.44E-04	0.00	0.28	11.0	0.63	7	13.0	15.9	3.9E-04	-1.0E+00	1.2E-04	
13	1.25E-04	1.25E-04	2.50E-04	0.00	0.28	12.5	0.70	7	13.1	15.4	3.4E-04	4.0E-03	1.2E-04	
14	1.37E-04	1.37E-04	2.73E-04	0.00	0.28	13.0	0.74	7	13.1	15.4	3.4E-04	7.0E-04	1.4E-04	
15	1.37E-04	1.37E-04	2.73E-04	0.00	0.28	14.0	0.80	8	13.3	15.4	3.4E-04	1.9E-03	1.4E-04	
16	1.60E-04	1.60E-04	3.20E-04	0.00	0.27	15.0	0.85	8	13.4	16.1	4.0E-04	6.0E-03	1.6E-04	
17	1.60E-04	1.60E-04	3.20E-04	0.00	0.27	16.0	0.91	8	13.5	16.1	4.0E-04	2.2E-03	1.6E-04	
18	1.44E-04	1.44E-04	2.89E-04	0.00	0.27	17.0	0.96	8	13.6	15.6	3.6E-04	1.0E+00	1.4E-04	
19	1.39E-04	1.39E-04	2.78E-04	0.00	0.26	18.0	1.02	8	13.8	15.6	3.6E-04	2.0E+00	1.4E-04	
20	1.34E-04	1.34E-04	2.68E-04	0.00	0.26	19.0	1.07	8	14.6	15.6	3.6E-04	3.0E+00	1.3E-04	
21	1.13E-04	1.13E-04	2.26E-04	0.00	0.26	20.0	1.13	8	14.4	15.6	3.6E-04	4.0E+00	1.1E-04	
22	1.15E-04	1.15E-04	2.29E-04	0.00	0.25	21.0	1.18	9	14.2	15.6	3.6E-04	5.0E+00	1.1E-04	
23	3.14E-04	3.14E-04	6.28E-04	0.01	0.25	22.0	1.24	9	15.0	18.6	7.3E-04	6.0E+00	3.1E-04	
24	3.78E-04	3.78E-04	7.57E-04	0.01	0.24	23.0	1.29	9	15.3	18.6	7.3E-04	7.0E+00	3.8E-04	
25	4.48E-04	4.48E-04	8.97E-04	0.01	0.23	24.0	1.35	9	15.5	18.6	7.3E-04	8.0E+00	4.5E-04	
26	3.93E-04	3.93E-04	7.87E-04	0.01	0.22	25.0	1.40	9	15.6	17.8	6.0E-04	9.0E+00	3.9E-04	
27	3.05E-04	3.05E-04	6.10E-04	0.01	0.21	26.0	1.46	9	15.4	17.8	6.0E-04	1.0E+01	3.0E-04	
28	3.30E-04	3.30E-04	6.61E-04	0.01	0.21	27.0	1.51	9	15.6	17.8	6.0E-04	1.1E+01	3.3E-04	
29	4.53E-04	4.53E-04	9.06E-04	0.01	0.20	28.0	1.57	9	15.9	17.8	6.0E-04	1.2E+01	4.5E-04	
30	8.03E-04	8.03E-04	1.61E-03	0.02	0.19	29.0	1.62	9	16.0	19.8	9.5E-04	1.3E+01	8.0E-04	
31	4.26E-04	4.26E-04	8.52E-04	0.01	0.17	30.0	1.68	10	15.9	17.8	6.0E-04	1.4E+01	4.3E-04	
32	2.54E-04	2.55E-04	5.09E-04	0.01	0.16	31.0	1.73	10	15.5	17.8	6.0E-04	1.5E+01	2.5E-04	
33	2.44E-04	2.44E-04	4.88E-04	0.01	0.15	32.0	1.79	10	15.5	17.8	6.0E-04	1.6E+01	2.4E-04	
34	2.00E-04	2.01E-04	4.01E-04	0.00	0.15	33.0	1.84	10	15.3	17.8	6.0E-04	1.7E+01	2.0E-04	
35	1.91E-04	1.91E-04	3.82E-04	0.00	0.14	34.0	1.90	10	15.3	17.8	6.0E-04	1.8E+01	1.9E-04	
36	1.88E-04	1.88E-04	3.76E-04	0.00	0.14	35.0	1.95	10	15.3	17.8	6.0E-04	1.9E+01	1.9E-04	
37	1.56E-04	1.56E-04	3.12E-04	0.00	0.13	36.0	2.01	10	15.3	17.0	5.0E-04	2.0E+01	1.6E-04	
38	1.60E-04	1.60E-04	3.21E-04	0.00	0.13	37.0	2.06	10	15.4	17.0	5.0E-04	2.1E+01	1.6E-04	
39	2.00E-04	2.01E-04	4.01E-04	0.00	0.12	38.0	2.12	10	15.7	17.0	5.0E-04	2.2E+01	2.0E-04	
40	1.86E-04	1.86E-04	3.72E-04	0.00	0.12	39.0	2.17	10	15.6	17.0	5.0E-04	2.3E+01	1.9E-04	
41	1.99E-04	2.00E-04	3.99E-04	0.00	0.12	40.0	2.23	10	15.7	17.0	5.0E-04	2.4E+01	2.0E-04	
42	1.81E-04	1.81E-04	3.62E-04	0.00	0.11	41.0	2.28	10	15.6	17.0	5.0E-04	2.5E+01	1.8E-04	
43	1.72E-04	1.72E-04	3.44E-04	0.00	0.11	42.0	2.34	10	15.5	17.0	5.0E-04	2.6E+01	1.7E-04	
44	5.37E-04	5.38E-04	1.08E-03	0.01	0.10	43.0	2.39	10	16.2	19.0	8.0E-04	2.7E+01	5.4E-04	
45	5.30E-04	5.30E-04	1.06E-03	0.01	0.09	44.0	2.45	11	16.2	19.0	8.0E-04	2.8E+01	5.3E-04	
46	2.51E-04	2.51E-04	5.02E-04	0.01	0.08	45.0	2.50	11	15.9	17.0	5.0E-04	2.9E+01	2.5E-04	
47	2.13E-04	2.13E-04	4.26E-04	0.01	0.07	46.0	2.56	11	15.8	17.0	5.0E-04	3.0E+01	2.1E-04	
48	2.49E-04	2.49E-04	4.98E-04	0.01	0.07	47.0	2.61	11	15.9	17.0	5.0E-04	3.1E+01	2.5E-04	
49	2.63E-04	2.63E-04	5.25E-04	0.01	0.06	48.0	2.67	11	15.9	17.0	5.0E-04	3.2E+01	2.6E-04	
50	5.98E-04	5.99E-04	1.20E-03	0.01	0.05	49.0	2.72	11	16.2	19.0	8.0E-04	3.3E+01	6.0E-04	
51	9.48E-04	9.49E-04	1.90E-03	0.02	0.04	50.0	2.78	11	16.6	19.0	8.0E-04	3.4E+01	9.5E-04	
52	6.69E-04	6.69E-04	1.34E-03	0.02	0.02	51.0	2.83	11	16.3	19.0	8.0E-04	3.5E+01	6.7E-04	

seismic Settlement (inch)

col No	1	2	3	4	5	6	7	8	9	10	11	12
overburden		0.1 tsf		0.2 tsf		0.5 tsf		1 tsf		2 tsf		4 tsf
		0.1	0.14	0.2	0.32	0.5	0.7	1	1.4	2	2.7	4
	-0.06	-0.07	-0.07125	-0.07	-0.07125	-0.07	-0.03562	0.00	3.41E-15	0.00	3.41E-15	0.00
	2.97	3.17	3.1725	3.17	3.21	3.25	3.21	3.17	3.245625	3.32	3.31875	3.32
	4.71	5.12	5.1225	5.12	5.158125	5.19	5.158125	5.12	5.158125	5.19	5.19375	5.19
	6.01	6.85	6.78	6.71	6.70875	6.71	6.6	6.49	6.49125	6.49	6.49125	6.49
	6.94	8.15	8.004375	7.86	7.824375	7.79	7.68	7.57	7.60875	7.65	7.64625	7.65
	7.74	9.23	9.0525	8.87	8.835	8.80	8.653125	8.51	8.473125	8.44	8.4375	8.44
	9.00	11.25	10.875	10.50	10.3	10.10	10.05	10.00	10	10.00	10	10.00
	9.91	12.91	12.3675	11.83	11.53875	11.25	11.10563	10.96	10.81688	10.67	10.6725	10.67
	11.50	17.00	15.9	14.80	14.1425	13.49	13.2425	13.00	12.75	12.50	12.5	12.50
	13.00	24.59	21.49125	18.39	17.16375	15.94	15.36	14.78	14.53125	14.28	14.27899	14.28
	13.30	29.86	24.42875	19.00	17.75	16.50	16.05	15.60	15.3	15.00	14.996	14.99
	14.68		33	26.18	22.93313	19.69	18.67875	17.67	17.20125	16.73	16.65205	16.57
	15.00			29.78	25.45688	21.13	19.86938	18.61	17.80375	17.00	16.87793	16.76
	16.04				28	24.59	22.57313	20.55	19.79625	19.04	18.69297	18.35
	16.97					29.86	26.68313	23.51	22.28438	21.06	20.36335	19.67
	17.65						28	26.32	24.555	22.79	21.7236	20.66
	18.39							30.00	27.51188	25.02	23.42701	21.83
	18.95								30	27.12	25.02245	22.93
	19.57									29.93	27.17323	24.42
	21											30