APPENDIX E
PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOHAZARD INVESTIGATION

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MOORE TWINING ASSOCIATES, INC.

PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOHAZARD INVESTIGATION
PROPOSED P1 PARKING STRUCTURE
OXNARD HARBOR DISTRICT
PORT OF HUENEME
VENTURA COUNTY, CALIFORNIA

Project Number: G93908.01-1

For:

Michael Baker International
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July 8, 2026

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MOORE TWINING ASSOCIATES, INC.

July 8, 2026 G93908.01-1

Michael Baker International, Inc.
9635 Granite Ridge Drive, Suite 300
San Diego, CA 92123

Attention: Ms. Melissa Whittemore

Subject: Preliminary Geotechnical Engineering and Geohazard Investigation
Proposed P1 Parking Structure
Oxnard Harbor District, Port of Hueneme
Ventura County, California

Dear Ms. Whittemore:

We are pleased to submit this preliminary geotechnical engineering and geohazard investigation report conducted for the planning phase of the P1 parking structure to be located at the Port of Hueneme, Ventura County, California. The contents of this preliminary report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluations, conclusions and preliminary recommendations.

We appreciate the opportunity to be of service to Michael Baker International, Inc. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
MOORE TWINING ASSOCIATES, INC.

Shaun Reich, EIT
Staff Engineer
Geotechnical Engineering Division

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EXECUTIVE SUMMARY (Page 1)

Moore Twining conducted this preliminary geotechnical engineering investigation for the proposed P1 Parking Structure Project planned within the Port of Oxnard, Port of Hueneme Harbor Facility.

It is our understanding the project consists of the preliminary design of up to six-level parking structure with a total floor area of approximately 456,000 square feet, and a maximum height of 75 feet. It is further reported that the project will be constructed in two (2) phases. The project will also include an adjacent 346,000 square-foot concrete pad for on grade parking of oversize vehicles or specialized equipment. Project construction will include demolition of existing improvements such as pavements, concrete flatwork, underground utilities, and a warehouse structure.

The field exploration consisted of a site reconnaissance, (seismic) cone penetration testing (CPT), drilling test borings, and soil sampling. Five (5) CPT soundings were advanced to depths of 100 feet below site grade (BSG). In addition, two (2) test borings were drilled using rotary auger methods within the proposed project site limits to depths ranging from about 76½ to 82½ feet BSG.

In general, below the asphalt concrete pavements, the auger explorations generally encountered a silty sand fill and poorly graded sand fill soils extending up to a depth ranging from 3 to about 7 feet below site grade (BSG). Underlying the fill soils and where fill was not observed, the native surface soils generally consisted of alternating layers of silty sand and poorly graded sand to depths of about 5 to 7½ feet BSG, with exception that silty clay was encountered at a depth of about 1-foot BSG in the hand auger boring excavated at SCPT-05. The silty sands and poorly graded sands were further underlain by alternating layers of medium stiff to stiff lean clay, silty clay, and very soft fat clay, extending to a depth of 15 to 16 feet BSG. These clay soils were further underlain by alternating layers of medium dense to dense poorly graded sand, medium dense to very dense silty sand, dense clayey sand, hard lean clay, dense poorly graded sand with silt, soft to stiff fat clay and hard silt extending to the maximum depth of the rotary auger borings, 81½ feet BSG.

The Cone Penetration Tests (CPTs) conducted for this investigation encountered soil "behavior" types that generally correlate with the soil borings. In general, the CPT data generally indicated dense to very dense granular soils with interbedded zones of medium dense granular soils and interbeds of soft to medium stiff clay and silt of varying thickness throughout the depth explored in the CPTs, to about 100 feet BSG, the maximum depth explored. Below a depth of about 80 to 85 feet BSG, the CPTs indicated very dense granular soils.

Due to shallow groundwater conditions, dewatering and wet soils are potential impacts for site preparation and construction of the project. Groundwater was initially encountered at depths of ranging from 9 to 10.5 feet BSG in the borings and CPTs excavated for this study. Based on the "Seismic Hazard Zone Report for the Oxnard 7.5-Minute Quadrangle, Ventura County, California," dated 2002, the historic high groundwater at the site is shown to be 5 feet.

The subject site is located in a Seismic Hazard Zone for liquefaction (California Geological Survey, Seismic Hazard Zone Map of the Oxnard Quadrangle). The subject site is also located in a liquefaction hazard zone identified by the Ventura County 2040 General Plan Draft Environmental Impact Report, published January 13, 2020. The results of the liquefaction analysis indicated total seismic settlements ranging from about 3 inches to 6¾ inches.

Due to the potential impacts from liquefaction, seismic settlement and lateral spreading, structural improvements and other improvements that are intended to be protected from the effects of liquefaction should be addressed by use of deep ground modification and/or by using pile foundations which are designed considering downdrag loading due to liquefaction. The advantages and disadvantages of the foundation support options would need to be evaluated to determine the preferred approach for support of the structure. Refer to the discussion of potential ground modification methods in Section 7.4 of this report.

Due to the shallow groundwater conditions, and the presence of near surface, fine-grained soils, wet, unstable soil conditions should be anticipated during site preparation. Use of bridging material such as large, crushed cobble with geotextile fabric should be anticipated to bridge soft areas. The moisture contents of the near surface soils are considered excessive for compaction. Thus, drying of wet soils, or replacement with suitable, drier fill materials should be anticipated to achieve moisture contents that allow for compaction of the soils to a stable condition.

The results of soil sample analyses indicate that the near surface soils exhibit a "corrosive" corrosion potential to buried metal objects.

Based on Table 19.3.1.1 - Exposure Categories and Classes from Chapter 19 of ACI 318, the sulfate concentrations from chemical testing of soil samples corresponds to an S0 classification (less than 0.10 percent by weight) for concrete based on two (2) samples tested. Thus, for a sulfate exposure category S0 should be anticipated for the site soils.

This executive summary should not be used for design and should be reviewed in conjunction with the attached preliminary report.

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 PURPOSE AND SCOPE OF INVESTIGATION 1

2.1 Purpose 1
2.2 Scope 2

3.0 BACKGROUND INFORMATION 3

3.1 Site History and Previous Studies 3
3.2 Site Description 3
3.3 Anticipated Construction 4

4.0 INVESTIGATIVE PROCEDURES 4

4.1 Field Exploration 4
4.1.1 Site Reconnaissance 4
4.1.2 Cone Penetration Test (CPT) Soundings 4
4.1.3 Drilling Test Borings 5
4.1.4 Soil Sampling 5
4.2 Laboratory Testing 5

5.0 FINDINGS AND RESULTS 6

5.1 Surface Conditions and Existing Asphalt Concrete Pavement Sections 6
5.2 Soil Profile 6
5.3 Soil Engineering Properties 7
5.4 Groundwater Conditions 9

6.0 GEOLOGIC AND SEISMIC HAZARDS EVALUATION 10

6.1 Geomorphic and Geologic Setting 10
6.2 Tectonics and Seismicity 11
6.3 Geologic Hazards 13
6.3.1 Landslides 13
6.3.2 Conditional Geologic Hazards 13
6.3.2.1 Volcanic Activity 13
6.3.2.2 Flooding and Dam Inundation 14
6.3.2.3 Seiches and Tsunamis 14
6.3.2.4 Radon Gas 15
6.3.2.5 Naturally Occurring Asbestos 15
6.3.2.6 Hydrocollapse 15
6.3.2.7 Regional Subsidence 15
6.3.2.8 Clays and Cyclic Softening 15
6.4 Seismic Hazards 16
6.4.1 Active Faulting and Surface Fault Rupture 16
6.4.2 Groundshaking 16
6.4.2.1 Historic Seismic Activity 17
6.4.2.2 Design Seismic Ground Motion Parameters and Site Class 17
6.4.3 Liquefaction and Seismic Settlement 18

TABLE OF CONTENTS (Cont'd)

7.0 GEOTECHNICAL ENGINEERING EVALUATIONS 20

7.1 Existing Site Features and Surface Conditions 20
7.2 Wet Soils and Shallow Groundwater 20
7.3 Expansive Soils 20
7.4 Mitigation Approaches and Foundation Support due to Liquefaction 21
7.5 Soil Corrosion 21
7.6 Sulfate Attack of Concrete 22

8.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS 23

9.0 DESIGN CONSULTATION 26

10.0 CONSTRUCTION MONITORING 26

11.0 NOTIFICATION AND LIMITATIONS 27

12.0 CLOSING 29

TABLE OF CONTENTS

APPENDICES

REFERENCES

APPENDIX A - Drawings A-1

Drawing No. 1 - Site Location Map
Drawing No. 2 - Test Boring and CPT Location Map
Drawing No. 3 - Regional Geologic Map
Drawing No. 4 - Map of Active and Potentially Active Faults Relative to Site
Drawing No. 5 - Historical Earthquake Epicenter Map
Drawing No. 6 - Site Geologic Map

APPENDIX B - Logs of Borings, CPTs, and Gregg Drilling CPT Report B-1

APPENDIX C - Results of Laboratory Tests C-1

APPENDIX D - Graphical Summaries of Liquefaction Analysis D-1

PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOHAZARD INVESTIGATION PROPOSED P1 PARKING STRUCTURE OXNARD HARBOR DISTRICT PORT OF HUENEME VENTURA COUNTY, CALIFORNIA

Project Number: G93908.01

1.0 INTRODUCTION

We are pleased to submit this preliminary geotechnical engineering and geohazard investigation report conducted for the proposed P1 Parking Structure project at the Oxnard Harbor District, Port of Hueneme facility in Ventura County, California. Moore Twining Associates, Inc. (Moore Twining) was authorized to conduct this investigation by Michael Baker International.

The contents of this report include the purpose of the investigation and the scope of services provided. The existing site features and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, this report provides a summary of evaluations, geomorphic and geologic setting, geologic hazards, seismic hazards, general conclusions and related recommendations. The report appendices contain the drawings (Appendix A); the logs of borings, CPT's and Gregg Drilling CPT Report (Appendix B); the results of laboratory tests (Appendix C), and graphical results of the liquefaction analysis (Appendix D),

The Geotechnical Engineering Division of Moore Twining, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of the investigation was to conduct a preliminary investigation to evaluate potential geotechnical engineering design constraints for the subject project, and to provide preliminary geotechnical engineering conclusions and recommendations for project planning and conceptual design of the subject project, including the following:

2.1.1 A summary of the soil and groundwater conditions encountered during the investigation;

2.1.2 A description of various geologic hazards that may impact the site including expansive soils, liquefaction, seismicity, flooding, and tsunami;

2.1.3 Descriptions of potential geologic/seismic hazards in accordance with the requirements of the 2025 California Building Code (CBC) and California Geological Survey – Note No. 48;

2.1.4 The seismic site class and mapped spectral acceleration parameters in accordance with the 2025 CBC;

2.1.5 Discussion of potential foundation types and methods of foundation support;

2.1.6 Discussions regarding soil corrosion potential; and

2.1.7 Test boring and Cone Penetration Test (CPT) logs and presentation of laboratory test results.

This report is provided specifically for the proposed P1 Parking Structure improvements described in the Anticipated Construction section of this report. This investigation is not intended to provide design level geotechnical engineering parameters for the project. Rather, this investigation is intended to assess potential geotechnical engineering constraints for project planning and to provide general recommendations for consideration in conceptual design. Future geotechnical engineering investigation will be required to develop final design recommendations for the project. This investigation did not include a site response analysis, site specific ground motion hazard analysis, in-place density tests, an environmental investigation, or an environmental audit.

2.2 Scope: Our proposal, dated January 13, 2025, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows:

2.2.1 The Request for Proposal for Contract Caltrans PPNO No. 6323, Advantage ID No. 724000320 No. "Preliminary Design and Environmental Services for P1 Parking Structure", dated 11/15/2024, prepared by The Port of Hueneme, Oxnard Harbor District, was reviewed.

2.2.2 The Site Plan prepared by Watry Design, Inc., identified as Port Hueneme Storage Garage, dated July 03, 2026, was reviewed.

2.2.3 A "Well Permit",GWP-09009 was obtained from Ventura County for the geotechnical exploration.

2.2.4 Various plans from the Port of Hueneme, Oxnard Harbor District, "Harbor Expansion Project", delta date December 1972, prepared by Moffatt & Nichol Engineers, were reviewed.

2.2.5 The "Notice of Preparation of a Draft Environmental Impact Report", dated February 12, 2026, was reviewed.

2.2.6 The Port Hueneme Harbor, Condition Survey, date April 7, 2025, prepared by the U.S. Army Corps of Engineers, Los Angeles District, was reviewed for general topographic elevations and grading information.

2.2.7 Various plans from the Port of Hueneme, "Berth 3 (Wharf 1) Deepening and Reconstruction Project",delta date November 01, 2024, prepared by KPFF, were reviewed.

2.2.8 Satellite images of the site between the years 1993 and 2022 from online

2.2.9 A site reconnaissance and subsurface exploration were conducted.

2.2.10 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.

2.2.11 Ms. Melissa Whittemore (Michael Baker International, Inc), Ms. Jennifer Boberg (The Port of Hueneme),and Mr Rodrigo Zaragoza, (Engineering Specialist for The Port of Hueneme), were consulted during the investigation.

2.2.12 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and engineering properties of the subsurface soils encountered.

2.2.13 This report was prepared.

3.0 BACKGROUND INFORMATION

The site history and previous studies, site description and the anticipated construction are summarized in the following subsections.

3.1 Site History and Previous Studies: At the time of the preparation of this investigation, the P1 Parking Structure was being planned to the east of the Port Hueneme entrance channel and to the north of At the time of our field investigation, this area was mostly occupied by an asphalt concrete parking lot used to stage vehicles being offloaded from

Based on review of readily available online aerial imagery dated from 1972, the subject area proposed to support the P1 Parking Structure had been occupied by three (3) relatively large above-ground storage tanks (AST's) located within a fenced area, parking areas, equipment storage, and a warehouse with a footprint of about 100,000 square-feet. An image of the site from 2006 indicates the three (3) AST's had been removed and the areas was being used for vehicle parking and storage, with the remaining areas unchanged. The improvements shown on the 2006 aerial image were similar to the conditions observed during our field investigation.

No previous reports of geotechnical engineering reports or environmental studies conducted within the subject site were provided for review during this investigation. If available, these reports should be provided for review and consideration for this project.

3.2 Site Description: The proposed P1 Parking Structure project is planned to be located within the northern portion of the Oxnard Harbor District, Port of Hueneme facility, located in Port Hueneme, California (see Drawing No. 1 in Appendix A). The subject site is located at about 34.150590 degrees latitude and -119.202656 degrees longitude.

The northern site area comprises approximately 35-acres of the about 85-acre parcel, with an address of 105 East Hueneme Road, in Ventura County, California.

At the time of our site observations, the subject site was occupied by an asphalt concrete parking lot and a warehouse building, as shown on Drawing No. 2 in Appendix A. In addition, pole mounted site lighting, stormwater inlet grates, and firewater hydrants were observed. In addition, the northern portion of the site was observed to contain interior chain-link cross fencing separating parking and storage areas for various equipment, and material storage. The overall site was observed to be primarily paved with asphalt concrete pavement with sporadic areas of Portland cement concrete pavements.

The project site is bounded to the north by Naval Base Ventura County(NBVC) Port Hueneme and single and multi-family residential development, to the east by single and multi-family residential development, to the south by the Port of Hueneme facility, with the commercial harbor channel, the southern portion of the Port of Hueneme facility, and the Pacific Ocean further south, and to the east by the NBVC Port Hueneme, and channel entrance to the west with the Pacific Ocean further west. Based on the "Notice of Preparation of a Draft Environmental Impact Report" indicating project limits, the wharf decking and harbor channels are located to the south and west of the site, respectively.

The existing asphalt concrete pavement and Portland cement concrete flatwork were observed to be in fair to poor condition.

The subject property is relatively flat and online imagery indicates elevations at the project site range feet above mean sea level (AMSL).

3.3 Anticipated Construction: The following project description is based on the request for proposal document entitled "Preliminary Design and Environmental Services for P1 Parking Structure", Caltrans PPNO No. 6323, Advantage ID No. 724000320, dated 11/15/2024, prepared by The Port of Hueneme, Oxnard Harbor District. This document is referred to herein as the RFP.

In addition, the following description is based on the referenced Site Plan prepared by Watry Design, Inc., dated July 03, 2026.

The Oxnard Harbor District is proposing the construction and operation of the P1 Hueneme Parking Structure Project at the Port of Hueneme (Port) in the City of Port Hueneme, at 333 Ponoma Street, in Ventura County (County).

The proposed project would include an approximately 456,000-square-foot, up to six-level parking structure to support the Port's ongoing vehicle processing operations. The parking garage is anticipated to have a maximum height of 75 feet. The parking structure would likely consist of reinforced concrete and would be designed to accommodate large quantities of vehicles and continuous operational use. Additionally, "high-and-heavy" vehicles, which may include oversized vehicles or specialized equipment, would be stored in a new approximately 346,000-square-foot at-grade concrete pad area located south of the proposed parking structure.

Project construction will include demolition of existing improvements such as pavements, concrete flatwork, underground utilities, and a warehouse structure.

Due to the planning level nature of the project, foundation loads have not been determined.

Appurtenances such as concrete flatwork, underground utilities, pavements, and other site improvements are anticipated as part of the project.

The extent of proposed grading was not known at the time of preparation of this report. However, considering the site is relatively flat, significant grading is not anticipated.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing programs conducted for this investigation are summarized in the following subsections. The field exploration was conducted in accordance with the scope of work included in our proposal, dated

4.1 Field Exploration: The field exploration included a site reconnaissance, cone penetration testing, drilling test borings, conducting standard penetration tests, and soil sampling. Prior to the field exploration, the exploration locations were marked and cleared by Underground Service Alert (USA) members. In addition, a private utility locator scanned the exploration locations for potential utility conflicts.

4.1.1 Site Reconnaissance: A site reconnaissance was conducted by a Moore Twining staff engineer during the field exploration work from December 15 through 19, 2025. The reconnaissance consisted of walking the areas proposed for the building and noting visible surface features. The features noted are described in the "Site Description" section of this report.

4.1.2 Cone Penetration Test (CPT) Soundings: On December 15th, and 16th, 2025, five (5) CPT soundings (CPT-03, CPT-04, SCPT-05, CPT-06, and SCPT-07) were advanced to depths of about 52½ feet BSG to 100 feet below site grade (BSG). The CPT locations were cleared of owner installed underground utility by advancing to a depths ranging from 3 to 7 feet BSG with a hand auger prior to advancing the CPT. The CPTs which did not extend to 100 feet BSG encountered refusal due to high tip resistance. The approximate locations of the CPT soundings are shown on Drawing No. 2 in Appendix A of this report.

The CPT soundings were performed by Gregg Drilling LLC. The CPT investigations were advanced using an electronic piezocone with a cone base area of 15 cm^2 and a sleeve surface area of 225 cm^2. The cone was hydraulically advanced using a 30-ton CPT rig in accordance with ASTM Test Method D 5778 (Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils). CPT measurements of cone bearing resistance, sleeve friction, and dynamic pore water pressure were recorded at about 0.05 meter (~0.16 foot) intervals during penetration to provide near continuous logs of the soil behavior types. In addition, shear wave velocity measurements were obtained in two (2) of the CPT soundings (SCPT-05 and SCPT-07) at ~10 foot intervals. The CPT hand auger excavation logs, CPT sounding logs and test data obtained, are presented in Appendix B of this report.

4.1.3 Drilling Test Borings: On December 17th and 18th, 2025, two (2) test borings were drilled for this investigation within the proposed building areas.

The test borings were drilled by Gregg Drilling LLC using a Mobile B54 Model D3 truck mounted drill rig to depth of about 76½ to 81½ feet below site grade (BSG). The drilling rig was equipped with 8-inch diameter hollow-stem augers which were used to advance the borings to a depth of about 10 feet, after which a 4-inch rotary wash bit and rotar
advance the borings to target depth. The approximate test boring locations are shown on the test boring location map provided as Drawing No. 2 in Appendix A.

The soils encountered in the test borings were logged during drilling by a representative of Moore Twining. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features. Soil samples were collected and returned to our laboratory for classification and soil mechanics testing. The presence and depth of free water, if any, in the test borings were noted and recorded during the drilling.

Test boring locations were determined by field measurement with reference to existing site features. Elevations of the borings were not surveyed as a part of the investigation. In accordance with the requirements of Ventura County Environmental Health Department, the boreholes were backfilled with neat cement.

4.1.4 Soil Sampling: During drilling of the borings, standard penetration tests were conducted, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler (SPT) has a 2 inch O.D. and a 1-⅜ inch inside diameter (I.D.). The sampler is driven by a 140 pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing shelby tubes and driving a California modified split barrel sampler into the soil. The soil was retained in stainless steel rings, 2.5 inches O.D. and 1 inch in height. The lower 6 inch portion of the samples were placed in close-fitting, plastic, air-tight containers which, in turn, were placed in cushioned boxes for transport to the laboratory.

Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the samples tested. The tests were conducted on disturbed and relatively undisturbed samples considered representative of the subsurface material encountered.

The results of laboratory tests are summarized in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

5.1 Surface Conditions and Existing Asphalt Concrete Pavement Sections: Our site observations are summarized in the Background Information section of this report. The pavement section thicknesses encountered at the borings and CPT locations are summarized in the Table No.1 below.

Table No. 1
Asphalt Concrete Pavement Section Thicknesses Encountered

Boring Number	Asphalt Concrete Thickness[1] (inches)	Aggregate Base Thickness[2] (inches)	Subgrade Soil
B-1	8.0	1½	Silty Sand Fill
B-2	7.0	3	Silty Sand
CPT-3	6½	5	Silty Sand Fill
CPT-4	7.0	5	Silty Sand Fill
SCPT-5	6.0	4	Silty Clay
CPT-6	7½	9	Silty Sand Fill
SCPT-7	7.0	5½	Silty Sand Fill

[1] - The asphalt concrete thickness was measured on four sides to the nearest ¼ inch and averaged to the nearest inch.
[2] - The aggregate base thickness was measured to the nearest ½ inch.

5.2 Soil Profile: In general, below the asphalt concrete pavements, the auger explorations generally encountered a silty sand fill and poorly graded sand fill soils extending up to a depth ranging from 3 to about 7 feet below site grade (BSG). Underlying the fill soils and where fill was not observed, the native surface soils generally consisted of alternating layers of silty sand and poorly graded sand to depths of about 5 to 7½ feet BSG, with exception that silty clay was encountered at a depth of about 1 foot BSG in the hand auger boring excavated at SCPT-05. The silty sands and poorly graded sands were further underlain by alternating layers of lean clay, silty clay, and fat clay, extending to a depth of 15 to 16 feet BSG.
graded sand, silty sand, clayey sand, lean clay, poorly graded sand with silt, fat clay and silt extending to the maximum depth of the rotary auger borings, 81½ feet BSG.

The Cone Penetration Tests (CPTs) conducted for this investigation encountered soil "behavior" types that generally correlate with the soil borings. In general, the CPT data generally indicated dense to very dense granular soils with interbedded zones of medium dense granular soils and interbeds of soft to medium stiff clay and silt of varying thickness throughout the depth explored in the CPTs, to about 100 feet BSG, the maximum depth explored. Below a depth of about 80 to 85 feet BSG, the CPTs indicated very dense granular soils.

5.3 Soil Engineering Properties: The engineering properties of the subsurface soils

Silty Sand and Poorly Graded Sand Fill Soils - A granular surficial layer of silty sand fill and poorly graded sand fill soils were generally encountered in a majority of the exploration locations throughout the site.
CPT-06, and SCPT-07, extending to d

Native Lean Clay with varying amounts of sand and silt: The native sandy lean clays encountered were described as me
ranging from 5 to 43 blows per foot, and SPT equivalent N-values (estimated by driving a California Modified split barrel sampler) of 5 to 7 blows per foot. The moisture content of the samples tested ranged from about 17.7 to 37.4 percent.
results of a direct shear test conducted
on a sample collected from boring B-2 at a depth of 5 to 6½ ft BSG indicated an internal angle of friction of 36 degrees with
conducted on a sample collected from boring B-1 at depths of 7½ to 9 feet BSG indicated a liquid limit of 32 and a plastic
boring B-2 at depths of 5 to 6½ feet BSG indicated a liquid limit of 22 and a plasticity index of 6. An Atterberg Limits test conducted on a sample colle
BSG indicated a liquid limit of 42 and a plasticity index of 25.
on a
21 and a plasticity index of 6.
from Boring B-1 at a depth of 10 to 11½ feet BSG indicated high compressibility characteristics (About 14
of 2 kips per square foot, the sample exhibited a of 0.4 percent. The result of one (1)
test conducted 9 feet BSG
indicated medium compressibil
16 kips per square foot). An expansion index test conducted on a soil sample collected from location
at depths ranging from 1 to 5-feet BSG indicated an index value of 6. An expansion index test conducted on a s
BSG indicated an index value

Native Clayey Sands and Clayey Sands with Gravel: The native clayey sands encountered were described as medium dense to dense as indicated by standard penetration resistance, SPT equivalent

foot.
undisturbed samples revealed dry densities of 91.4, 104.6, 110.2 and 117.1 pounds per cubic foot. An

BSG indicated a liquid
conducted on a sample collected from boring B-2 at a depth of 50 to 51½ ft BSG indicated an internal angle of friction of 29 degrees with a cohesion value of 820 pounds per square foot. Pocket penetrometer tests conducted during the fieldwork investigation recorded relative compressive strength of 0.75 tons per square feet.

Native Poorly Graded Sands and Poorly Graded Sands with Silt: The native poorly graded sands and poorly graded sands with silt encountered were described as medium dense to very dense as indicated standard penetration resistance, N-values, ranging from 32 to greater than 50 blows per foot and SPT equivalent N-values (estimated by driving a California Modified split barrel sampler) o
about 14.4 to 20.9 percent.
105.1 pounds per cubic foot. The results of a direct shear test conducted on a sample collected from
an internal angle of friction `41 degrees with
400 pounds per square foot.

Native Silty Sands with varying amounts of gravel: The native silty sands encountered were described as loose to very dense as indicated by a standard penetration resistance, N-value, ranging from 24 to 51 blows per foot, and an SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 7 to 56 blow per foot. The moisture contents of the samples tested ranged from about 8.5 to 22.8 percent. Six (6) relatively undisturbed sample revealed dry densities of about 85.3, 93.4, 103.2, 109.2, 110.0 and 110.3 pounds per cubic foot. The result of a consolidation test conducted on a sample collected from Boring B-1 at a depth of 5 to 6½ feet BSG indicated moderate compressibility characteristics (About 8 percent consolidation under a load of 16 kips per square foot) When wetted under a load of 2 kips per square foot, the sample exhibited a collapse of 0.1 percent. The result of a consolidation test conducted on a sample collected from Boring B-2 at a depth of 40 to 41½ feet BSG indicated moderate compressibility characteristics (About 5 ¾ percent consolidation under a load of 16 kips per square foot). The results of a direct shear test conducted on a sample collected from boring B-2 at a depth of 15 to 16½ ft BSG indicated an internal angle of friction of 25 degrees with a cohesion value of 490 pounds per square foot.

Native Fat Clay Soils: The native fat clay soils encountered were described as very soft to hard, as indicated by standard penetration resistance, N-value, of 0 to 4 blows per foot, and an SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 9 to greater than 50 blow per foot.
relatively undisturbed sample revealed a dry density of about 79.2 pounds per cubic foot. An Atterberg Limits test conducted on a sample collected from boring B-1 at depths of 65 to 66½ feet BSG indicated a liquid limit of 64 and a plasticity index of 40. An Atterberg Limits test conducted on a sample collected from boring B-2 at depths of 10 to 11½ feet BSG indicated a liquid limit of 53 and a plasticity index of 34. An Atterberg Limits test conducted on a sample collected from boring B-2 at depths of 75 to 76½ feet BSG indicated a liquid limit of 61 and a plasticity index of 39. Expansion Index testing conducted on a shelby tube sample collected from Boring B-2 at depths ranging from 12 to 14½ feet BSG indicated an expansion index of 88. Pocket penetrometer tests conducted during the fieldwork investigation recorded relative compressive strength ranging from 0 to 1.5 tons per square feet.

Native - Silt with varying amounts of Sand and Clay: The native sandy silt with varying amounts of sand and clay encountered was described as very stiff to hard as indicated by standard penetration resistance, SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 25 to 47 blow per foot. One (1) relatively undisturbed sample revealed a dry density of 92.5 pounds per cubic foot. The moisture contents of the samples tested ranged about 31 to 32.3 percent. An Atterberg Limits test conducted on a sample collected from boring B-2 at depths of 40 to 41 feet BSG indicated a non-viscous liquid limit and non-plastic plasticity index. Pocket penetrometer tests conducted during the fieldwork investigation recorded relative compressive strength ranging from 0.5 ton per square feet.

Expansion Index Tests: The results of an expansion index test conducted on a near surface lean clay with silt sample from sounding CPT-5 indicated an expansion index value of 6. The results of an expansion index test conducted on a sample collected from a depth of 12 to 14½ indicated an expansion index value of 88.

Resistance-value (R-value) Tests: One (1) R-value tests were performed for the new pavements proposed for the subject project. An R-value test conducted on a sample containing a mixture of silty sand fill and native silty sand clay sample collected from depths of 1 to 4 feet BSG from hand auger excavation collected from the location of sounding CPT-03 indicated an R-value of 69.

Chemical Analysis: Chemical tests were performed on two (2) soil samples collected within the upper 4 feet BSG to evaluate the corrosion potential. The results of the chemical testing performed are summarized in Table No. 2 below. The laboratory test sheets are included in Appendix C.

Table No. 2 - Corrosion Potential Test Results

Location / Parking Lot	Depth (Feet)	pH	Sulfate (Percent by Weight)	Chloride (Percent by Weight)	Minimum Resistivity (ohm-cm)
B-1	1.0-4.0	8.2	0.0034	0.0010	5,200
B-2	1.0-4.0	8.1	0.011	ND	3,300

ND - Not Detected (reporting limit of 0.0006 percent)

5.4 Groundwater Conditions: Due to the rotary wash methods used for drilling borings B-1 and B-2, groundwater depths were difficult to estimate in these borings. Groundwater was encountered within boring B-1 at a depth of about 15 feet BSG during drilling. However, a reliable measurement could not be made in boring B-2. Direct measurements with a water sounder conducted in the CPTs indicated groundwater depths ranging from about 9 to 10½ feet BSG.

Based on our review of Plate 1.2 Depth to historically highest groundwater and borehole locations, Oxnard 7.5-minute quadrangle, California, from the "Seismic Hazard Zone Report for the Oxnard 7.5-Minute Quadrangle, Ventura County, California," dated 2002, the historic high groundwater at the site

It should be recognized, however, that groundwater is dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. Actual groundwater levels during construction could be higher or lower than indicated in this report. The evaluation of such factors was beyond the scope of this investigation and report.

6.0 GEOLOGIC AND SEISMIC HAZARDS EVALUATION

6.1 GEOMORPHIC AND GEOLOGIC SETTING: The City of Port Hueneme is located on the southwest portion of the Oxnard Plain, adjacent to the Pacific Ocean, in Ventura County, California. In a broader sense, the Oxnard Plain is situated within the Transverse Ranges geomorphic province. The Oxnard Plain is bounded by the Santa Clara River Valley and the Santa Susana Mountains to the north and northeast with the Topatopa Mountains further beyond to the north; by the Santa Monica Mountains to the east; and by the Santa Barbara Channel of the Pacific Ocean to the south and west.

In regard to geologic structures, the following is summarized from Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan:

> *"Local physiography is dominated by the rugged slopes of the Transverse Ranges and reflects several water courses that run through the County. Three major riverine systems extend from the mountains to the ocean in the county: the Ventura River (watershed area is 227 square miles), the Santa Clara River (watershed area is 1,634 square miles, approximately 60 percent of which is in Ventura County), and the Calleguas Creek (watershed area is 343 square miles).*
>
> *Ventura County generally includes late Quaternary alluvial and fluvial sedimentary deposits, beach deposits and artificial fill. These deposits are composed mainly of volcanic, marine and non-marine sedimentary rocks overlying a basement complex of granitic and metamorphic rock. The Oxnard Plain is immediately underlain by thick alluvial sediments, which overlie the older sedimentary and volcanic rocks. Young Quaternary deposits cover about 13 percent of the Ventura County Quadrangle. Most of the exposed valley alluvium is Holocene (<11,700 years old), with older Quaternary deposits locally exposed on the lower slopes of surrounding hills. Most Holocene sediments exposed along the Ventura River valley are wash (Qw), alluvial (Qya), and marine (Qym) deposits. Superficially, the alluvial fan units are composed of material ranging in size from boulders to clay, with silt and clay being the major components."*

In regard to earthquake faults, the following is summarized from Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan:

> *"The Transverse Range's geomorphic province is characterized by west-trending folds, thrust faults, and fault-bounded valleys. The structural framework of the region is generally considered the result of regional compression caused by right-lateral strike-slip movement on the "Big Bend" segment of the San Andreas Fault. Major faults in the region are west-trending reverse faults.*

Based on the Geologic Map of the Oxnard 7.5-Minute Quadrangle, Ventura County, California, A Digital Database, dated 2003, prepared by Mr. Kevin B. Clahan, the area of the proposed parking structure is underlain by Artificial Fill (af) consisting of engineered and/or non engineered fill material.

According to the USGS 7½-Minute Series Topographic Map (Oxnard, California), produced by the United States Geological Survey (USGS), the area of the site where the parking structure is planned is at an elevation of about 10 feet above mean sea level (AMSL).

A regional geologic map is included as Drawing No. 3 in Appendix A of this report, and a site

6.2 TECTONICS AND SEISMICITY: An "active fault" is defined, for the purpose of this evaluation, as a fault that has had surface displacement within the Holocene Epoch (about the last 11,700 years). Potentially active faults also contribute to the seismicity of the site. A widely accepted definition of "potentially active" is a fault that does not show evidence of displacement during the Holocene, but shows evidence of displacement occurring less than 1.6 million years ago (after the beginning of the Pleistocene Epoch). Since the Quaternary Period includes the Holocene and Pleistocene Epochs, Quaternary Faults are those that have documented latest fault displacement within the Holocene or Pleistocene Epochs, undifferentiated. Faults showing evidence of displacement older than 1.6 million years are usually classified as "Pre-Quaternary" and "inactive." Pre-Quaternary faults do not impact the site seismicity.

The site is located in a region of high seismicity. The site location relative to known nearby fault systems within No. 5 (Map of Major Active and Potentially Active Faults Relative to Site) in Appendix A of this report.

The Fault Activity Map of California indicates that the nearest active fault to the site with surface rupture is the Springville Fault, which is located about 8 miles northeast of the site. The Springville Fault is a section of the Simi-Santa Rosa fault zone in the western Transverse Ranges and is a transpressional fault which combines horizontal strike-slip motion with vertical uplift, acting as part of a system that includes high-angle, transrotational faults.

The following summarizes the significant faults from Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan:

Malibu Coast Fault System

The Malibu Coast fault system includes the Malibu Coast, Santa Monica, and Hollywood faults. The system begins in the Hollywood area, extends along the southern base of the Santa Monica Mountains, and passes offshore a few miles west of Point Dume. The 1973 Point Mugu earthquake is believed to have originated on this fault system.

Oak Ridge Fault System

The Oak Ridge fault system is a steep (65 degrees) southerly dipping reverse fault that extends from the Santa Susana Mountains westward along the southerly side of the Santa Clara River Valley and into the Oxnard Plain. The system is more than 50 miles long on the mainland and may extend an equal or greater distance offshore. Several recorded earthquake epicenters on land and offshore may have been associated with the Oak Ridge fault system. Portions of the system are zoned by the state as active.

Pine Mountain Thrust Fault and Big Pine Fault

These two large faults occur in the mountainous portion of Ventura County, north of the Santa Ynez fault; the faults are located 9 and 16 miles north of the City of Ojai, respectively. The Pine Mountain thrust fault is reported to have ruptured the ground surface for a distance of 30 miles along its length during the northern Ventura County earthquake of November 1852.

San Andreas Fault

San Andreas is the longest and most significant fault in California. While it does not run through Ventura County, it is located just north of the county boundary, in some cases less than a mile away. Because of clearly established earthquake activity, this fault has been designated as active by the State of California. The last major

earthquake on the San Andreas Fault near Ventura County was the Fort Tejon earthquake of 1857, which was estimated at magnitude M 8.0 on the Mercalli Scale, causing the roof of Mission San Buenaventura to collapse and damaging its bell tower. It would have caused considerably more damage if there had been structures in the county at the time. There is a 59 percent chance that a magnitude M 6.7 quake or larger will occur on this fault within the next 30 years.

San Cayetano-Red Mountain-Santa Susana Fault System

This fault system consists of a major series of north-dipping reverse faults that extend over 150 miles from Santa Barbara County into Los Angeles County. Within this system, the San Cayetano fault is the greatest hazard to Ventura County; it is a major, north-dipping reverse fault that extends for 25 miles along the northern portion of the Ventura Basin. The San Fernando earthquake of 1971 was caused by activity along this fault.

Simi-Santa Rosa Fault System

This fault system extends from the Santa Susana Mountains westward along the northern margin of the Simi and Tierra Rejada valleys and along the southern slope and crest of the Las Posas Hills to its westerly termination.

Ventura-Pitas Point Fault

The western half of this fault is known as the Pitas Point fault, and the eastern half is known as the Ventura fault. The Pitas Point fault extends offshore into the Pacific Ocean and is roughly 14 miles long. The Ventura fault extends into the communities of Ventura and Sea Cliff and runs roughly parallel to portions of U.S. 101 and State Route 126. The fault is roughly 12 miles long. The Ventura-Pitas Point fault is a left-reverse fault."

Based on the results of disaggregation of the 2,475 year return period event (2 percent probability of exceedance in 50 years) for probabilistic analysis using the United States Geological Survey Earthquake Hazard Toolbox website (https://earthquake.usgs.gov/nshmp/) NSHM Conterminous U.S. 2018 model, the on-shore and off-shore segments of the Oak Ridge fault are the predominant contributing source of seismicity for the site area. The Malibu Coast (extension) fault, Sisar fault, Channel Islands fault, Anacapa - Dume fault, Simi-Santa Rosa fault, Red Mountain fault and Ventura-Pitas Point fault contribute successively lower percentages of the total seismicity and account for the remaining seismicity of the site area. Attributes of the primary faults and fault zones contributing to the site seismicity are provided in the following

Deterministic analysis using the Building Seismic Safety Council 2014 (BSSC2014) Scenario Catalog from the USGS website for the Earthquake Hazards Program (https://earthquake.usgs.gov/scenarios/catalog/bssc2014/) indicates that the Red Mountain fault has the highest predominant moment magnitude of all the faults listed in the table above with a magnitude of 7.4. The predominant moment magnitudes from the deterministic analysis are listed in the below table in parentheses.

Table No. 3
List of Quaternary Faults Contributing to the Seismicity at the Site

Fault (Rupture Segment)	Predominant Moment Magnitude(s)	Horizontal Distance to Site (Miles) from United States Geological Survey disaggregation website
Oak Ridge (offshore segment)	7.03 (6.9)	5.5
Oak Ridge (onshore segment)	7.34 (7.2)	7.2
Malibu Coast (extension)	7.47 (7.0)	5.5
Sisar	7.68 (7.0)	8.6
Channel Islands	6.84 (7.3)	7.2
Anacapa - Dume	7.55 (7.2)	9.1
Simi - Santa Rosa	7.12 (6.9)	7.8
Red Mountain	7.29 (7.4)	14.9
Ventura - Pitas Point	7.59 (7.1)	9.8

6.3 GEOLOGIC HAZARDS: The potential geologic hazards of landslides and conditional geologic hazards are described in the following subsections.

6.3.1 Landslides: In general, the site is relatively flat. However, based on the Construction Plans for "Berth 3 (Wharf 1) Deepening and Reconstruction Project", Bid Set dated 11/01/2024,prepared by KPFF, the adjacent commercial harbor channel extends about 35 to 40 feet below water surface, with earthen slopes constructed with a repose of about 2:1 (Horizontal:Vertical). Consequently, the subject project site is located about 250 feet north of existing descending slope of about 35 to 40 feet in height which extend below the water line in the adjacent channel. Due to the potential for liquefaction, the existing slopes are considered subject to seismic instability such as from lateral spreading, flow failure, etc. However, due to the distance between the parking structure and the existing slopes, and considering that the parking structure foundations will be either be supported on specialty deep ground modification or deep foundations, these measures will be designed to reduce potential impacts from the seismic slope instability.

6.3.2 CONDITIONAL GEOLOGIC HAZARDS: Conditional geologic hazards, as identified in California Geological Survey Note 48, with exception that this report does not include

6.3.2.1 Volcanic Activity: California includes six regions with a history of late Pleistocene volcanic eruptions that are subject to hazards from future eruptions (Miller, 1989). Of these six regions, none are located within 100 miles of the site. The volcanic region of the Rodman Mountains area within the Amber Crater-Lavic Lake Area, located about 152 miles northeast of the site, is the nearest volcanic region to the site.

Other than the aforementioned Amber-Crater-Lavic Lake Area, no additional known areas or areas of known volcanism are present within 100 miles of the site. Based on the distance between the site and known volcanic areas, potential impacts from basaltic lava flows or tephra hazards associated with basalt or basaltic-andesite vents is considered insignificant.

6.3.2.2 Flooding and Dam Inundation: The Flood Insurance Rate Map, Community Panel number 06111C0914F distributed by the Federal Emergency Management Agency, dated January 29, 2021, indicates the site is located in Zone X, which is defined as a "0.2 percent annual chance flood hazard, areas of 1 percent annual chance flood with average depth less than one foot or with drainage areas of less than one square mile."

Based on review of the City of Port Hueneme General Plan Update, Final Environmental Impact Report, dated September 2021, this document indicated, "Failure of any of four dams near the mouth of the Santa Clara River could potentially cause inundation in Port Hueneme. These dams include Pyramid Reservoir, Bouquet Canyon Reservoir, Castaic Reservoir, and Santa Felicia Dam (Port Hueneme 1998). Based on our review of the Division of Safety of Dams, California Dam Breach Inundation Maps (GIS website: https://fmds.water.ca.gov/maps/damim/), failure of the Castaic Dam is capable of inundating the entire site. Based on our review of Figure 11-6 Individual Dam Inundation Areas from Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan, the figure shows that the Castaic Dam and also Boquet Canyon Dam and Santa Felicia Dam are capable of inundating the entire site. Thus, dam inundation from the Castaic Dam, Boquet Canyon Dam, Santa Felicia Dam and also potentially from Pyramid Reservoir are considered to be a significant hazard for the site.

6.3.2.3 <u>Seiches and Tsunamis</u>: A seiche is a wave generated by the periodic oscillation of a body of water whose period is a function of the resonant characteristics of the containing basin as controlled by its physical dimensions. These periods generally range from a few minutes to an hour or more. Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan indicates, "There is no record of a seiche occurring in Ventura County. As such, the actual threat that is posed by seiches in Ventura County is small." However, based on the close proximity of the project site to the commercial harbor and channel entrance for Port Hueneme (site is located about 400 feet north and 600 feet east of portions of the Port of Hueneme), seiche is a potential hazard which could induce short term surface flooding near the parking structure.

Tsunamis are waves generated in oceans from seismic activity. The site is located about 400 feet north and 600 feet east of the port (harbor) and about 1,900 feet northeast of the Pacific Ocean. The City of Port Hueneme General Plan Update, Final Environmental Impact Report, dated September 2021, indicates, "Due to its proximity to the Pacific Ocean, areas
susceptible to tsunami inundation." Based on our review of Figure 4.9-2 Tsunami Inundation Areas from the City of Port Hueneme General Plan Update, Final Environmental Impact Report, dated September 2021, the limits of the site appear to be located just outside the mapped tsunami inundation area. However, based on the California Tsunami Hazard Area Maps and Data on the California Geological Survey (CGS) website (https://www.conservation.ca.gov/cgs/tsunami/maps), the entire site is shown to be mapped in a tsunami hazard inundation area. In addition, based on our review of Figure 11-9 Tsunami Evacuation Areas from Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan, the site is located in the area shown as the "Maximum Phase" for Tsunami Emergency Response Playbooks. The General Plan indicates, "Figure 11-9 shows the Phase 3 Evacuation and Maximum Evacuation Phase, based on models of maximum local and distance tsunamis coming from the Cascadia Subduction Zone. The Phase 3 Evacuation estimates a tsunami flood level of 1.7 to 5.0 feet above the high tide line, and a tsunami flood level of 7.7 to 11.0 feet above low tide conditions. The Maximum Evacuation Phase estimates a tsunami flood level of more than 5.0 feet above the high tide line, and a tsunami flood level of more than 11.0 feet above low tide conditions." Based on the close proximity of the site to the Pacific Ocean and the Port of Hueneme, and based on our review of the Tsunami Hazard Areas Map on the CGS website, tsunamis are a potential hazard for the site.

6.3.2.4 Radon Gas: Based on our review of Special Report 194 entitled, "Radon Potential in Ventura County, 2006 Update," dated August 2006, prepared by CGS, Figure 5 Radon Zones for Ventura County indicates that the City of Port Hueneme is located in a low radon potential zone which is defined as an area of geologic map units that likely have fewer than 6 percent of associated residences with radon levels that equal or exceed 4.0 picocuries per liter. The Radon Potential Zone Map for Ventura County, California, from Special Report 194, dated August 2006, also shows the City of Port Hueneme as being located in an area with "Low Potential for Indoor Radon Levels Above Four Picocuries Per Liter." In addition, since the parking structure will be open to the air and ventilated, indoor radon gas is not considered significant.

6.3.2.5 Naturally Occurring Asbestos: Asbestos occurs in soil and rock naturally in certain geologic settings in California. It has been documented that inhalation of asbestos fibers may cause negative health effects. Most commonly, naturally occurring asbestos is associated with serpentinite and partially serpentized ultramafic rocks. Ultramafic rocks are scattered throughout much of the Sierra Nevada mountain and the Coast Range regions. Open-File Report 2000-19, titled *A General Location Guide for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos,* prepared by the State of California Department of Conservation, Division of Mines and Geology, dated August, 2000, does not identify ultramafic rock in the site vicinity. The referenced report indicates the closest surficial occurrence of ultramafic rocks are located in Santa Barbara County, about 46 miles northwest of the site.

Ultramafic rocks were not encountered at the site during our investigation and are not common to the geologic environment of the site. Based on the cited literature and our site observations, it is our opinion that the potential to encounter near surface natura lly occurring asbestos containing rock at the site is very low.

6.3.2.6 Hydrocollapse: Collapsible soils typically consist of loose, dry, low-density soils that exhibit significant consolidation with the addition of water. Collapsible soils can be found in areas of young alluvial fans, debris flow sediments, and wind-blown deposits. Collapsible soils can possess significant strength in a dry state; however, the saturation of collapsible soils can break the clay/cement bonds holding the soil grains together and subsequent collapse of the soils can cause damage to overlying structures.

The results of laboratory consolidation tests conducted as part of this investigation indicated that the soils tested had insignificant collapse when wetted under a load of 2,000 pounds per square foot. In addition, considering the in-place dry densities of the near surface soils, the presence of shallow groundwater, and the results of the consolidation tests, collapsible soils are not anticipated to be a concern for the project.

6.3.2.7 Regional Subsidence: Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan indicates: "Several areas within Ventura County are experiencing subsidence due to groundwater extraction including the Oxnard Plain, the Las Posas Valley, and the Santa Clara River Valley. In addition, based on the interactive USGS mapping "Areas of Subsidence" (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html), the subject project site and entire City of Port Hueneme is located within a mapped area of subsidence due to groundwater pumping. Thus, the potential for regional subsidence should be recognized.

6.3.2.8 Clays and Cyclic Softening: Saturated, low plasticity clays were encountered in one of the borings drilled during this investigation. Accordingly, cyclic softening of the low plastic clays could occur due to earthquake shaking. However, the potential impacts from cyclic softening would be addressed by use of the mitigation measures described in this report due to liquefaction.

6.4 SEISMIC HAZARDS: The potential for fault hazard ground rupture, seismic groundshaking, seismic design ground motion parameters, and liquefaction and seismic settlement are described in the following subsections.

6.4.1 Active Faulting and Surface Fault Rupture: Earthquakes are caused by the sudden displacement of earth along faults with a consequent release of stored strain energy. The fault slippage can often extend to the ground surface where it manifests in abrupt relative ground displacement across a rupture surface. Damage resulting directly from fault rupture ground displacement occurs only where structures are located astride a rupture surface with relative displacement.

The project site is not located in an Alquist-Priolo Earthquake Fault Zone or a known local earthquake fault rupture hazard zone. The locations of active faults compared with the site were reviewed based on the 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6, prepared by Jennings and Bryant. A map depicting the major active faults in the vicinity of the site is included on Appendix A of this report.

The Fault Activity Map of California indicates the active fault with surface rupture nearest to the site is the Springville Fault, part of the Simi-Santa Rosa Fault Zone, which is located about 8 miles of the site. Considering the distance to the nearest known active fault with surface rupture, the potential for surface fault rupture due to a known active fault is considered low.

6.4.2 Groundshaking: For any given earthquake, the rock in the immediate vicinity will respond with a certain maximum acceleration and with a predominant period that depends on the nature of the rock and the source mechanism. Away from the focus of the earthquake, the ground motions begin to attenuate. The way in which the earthquake wave is altered depends to a great degree on source characteristics and to a lesser degree on the travel path.

Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan indicates the following about ground shaking and the effects of ground shaking in Ventura County:

> *"The effects of ground shaking in Ventura County depends on 1) conditions of the local geology influence events: solid bedrock is far less subject to intense shaking than loose sediment; 2) duration and intensity of the earthquake are subject generally to the size of the earthquake; and 3) distance (as the distance from the epicenter drops off so the intensity of the shaking decreases). The duration of strong ground motion is a function of magnitude, underlying geology, and distance from the fault. It is probably the single most important factor in producing excessive damage to structures. Long duration, reasonably high acceleration, and considerable amplitudes, such as would likely occur from a maximum seismic event on the Malibu Coast Fault system, are the combination that would cause the most damage to buildings in the County. A distant, maximum seismic event on the San Andreas Fault would produce less intensity of shaking; however, the duration of strong ground motion would be longer, resulting in a high potential for damage to high-rise flexible structures.*
>
> *Distance is another important factor affecting the severity of ground shaking. Ground shaking from distant seismic events(greater than 40 miles) will be different than events within 10 miles of the County. For more distant, large events (greater than 7.5M), such as those that occur on the San Andreas Fault, the ground shaking will reflect a predominance of long period waves. This will have minimal effects upon structures less than three stories in height, but will affect flexible structures (typically high-rise buildings, and other buildings taller than three stories), especially if the natural period of the building should coincide with that of the long period earthquake waves. The amplification of such motions could result in serious damage to high-rise*

structures. Short period waves, however, are generally very destructive near the epicenter of moderate- and large-magnitude seismic events, causing severe damage predominantly to low-rise rigid structures (less than three stories) not specifically designed to resist them.

The ground-shaking hazard exists throughout Ventura County, as it does throughout California. Certain areas may have increased ground shaking due to local geologic conditions, as well as, the location and orientation of the earthquake fault. The highest amplification of ground shaking occurs in areas with the greatest potential for long period wave shaking. Basically, this is the San Andreas Fault zone in the northern part of the county and the Oakridge Fault zone in the southern part of the county."

Chapter 11 Hazards and Safety from the Ventura County 2040 General Plan also includes Table 11-2 Percent of County Population Affected by Earthquake Related Ground Shaking (Ventura County, California) and indicates that 65.9 percent of Port Hueneme would be affected by extreme ground shaking. Table 11-2 does not provide percentages of Port Hueneme's population that could be affected by other categories of ground shaking, such as severe or violent ground shaking categories.

A summary of our review of historic seismic activity relative to the site is included below.

6.4.2.1 Historic Seismic Activity: The general site region has experienced recurring seismic activity. Based on historical earthquake data obtained from the U.S. Geological Survey's earthquake database system, approximately 251 historical earthquakes with magnitude 4.5 or greater have been recorded from 1900 through February 18, 2026, within about 100 miles of the site. A map showing the location of the project site with relation to the approximate historical earthquake epicenter locations and magnitude category is presented on Drawing No. 5 in Appendix A of this report.

The nearest earthquake event with magnitude 4.5 or greater was identified as a 4.8 magnitude earthquake which occurred on March 18, 1957, approximately ¼ mile south of the site (across the opposite side of the port harbour (between the port harbour and the Pacific Ocean). The most recent earthquake event 4.5 or greater was identified as a 4.7 magnitude earthquake which occurred on September 12, 2024 in the mountains north of Malibu and about 23¼ miles southeast of the site. The largest magnitude earthquake identified in the 100 mile search radius was the 7.5 magnitude 1952 Kern County, California Earthquake which occurred on July 21, 1952, approximately 56.9 miles north-northeast of the site. This event is estimated to have caused a peak ground acceleration at the subject site of about 0.07g based on the ground acceleration contours shown on the U.S.G.S. Shake Map. This analysis was performed using the USGS on-line United States Geological Survey, Search Earthquake Catalog, (https://earthquake.usgs.gov/earthquakes/search/).

6.4.2.2 Design Seismic Ground Motion Parameters: Seismic coefficients and spectral response acceleration values were developed in accordance with the 2025 California Building Code (CBC). The CBC methodology for determining design ground motion values is based on U.S Geological Survey seismic hazard maps.

This report assumes the seismic design standards for project design will be based on ASCE 7-22. Based on the data obtained during this investigation, the time-averaged shear wave velocity within the upper 100 feet (V_{S30}) at the two seismic CPT soundings indicated a V_{S30} of 640 feet per second and about 680 feet per second. Thus, based on ASCE 7-22, a Site Class DE was selected. It is recognized that due to the liquefaction susceptibility, a Site Class F category (and a site response analysis) would be applicable for design of structures with fundamental periods of 0.5 seconds or higher. However, in accordance with ASCE 7-22, the Site Class DE category would apply for determining the mapped acceleration coefficients for structures with fundamental periods less than 0.5 seconds, unless a more stringent requirement was applied to the project.

A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.67g was determined for the site using the Ground Motion Parameter Calculator available from the Ground Motion Parameter Calculator ASCE Hazard Tool online application (https://ascehazardtool.org). An earthquake magnitude of 7.52 was applied in the analysis based on the highest earthquake magnitude determined from probabilistic analysis (hazard disaggregation analysis for the United States Geological Survey Earthquake Hazard Toolbox website (https://earthquake.usgs.gov/nshmp/) NSHM Conterminous U.S. 2018 model, and the deterministic analysis using the Building Seismic Safety Council 2014 (BSSC2014) Scenario Catalog from the USGS website for the Earthquake Hazards Program (https://earthquake.usgs.gov/scenarios/catalog/bssc2014/).

6.4.3 <u>Liquefaction and Seismic Settlement</u>: Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing can result. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction.

Based upon the Earthquake Zones of Required Investigation, Oxnard Quadrangle, prepared by the California Geological Survey, dated December 20, 2002, the site is located in a liquefaction hazard zone.

Based on the "Seismic Hazard Zone Report for the Oxnard 7.5-Minute Quadrangle, Ventura County, California," dated 2002, the historic high groundwater at the site is shown to be 5 feet. Thus, a

Liquefaction and seismic settlement analyses were conducted based on the subsurface conditions encountered in the soil borings and cone penetration tests conducted for this investigation. The evaluations were conducted in accordance with the procedures of the Summary Report of Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER 97-0022, 1997; using the computer program LiquefyPro, developed by CivilTech Software. A groundwater depth of 5 feet, a peak horizontal ground acceleration of 0.67g, and an earthquake magnitude of 7.52 were used in the evaluation. The tip resistance and sleeve friction from the CPT data were used to determine the cyclic stress ratio needed to initiate liquefaction. The blow count data (N-values) from the Standard Penetration Testing conducted in the borings was also utilized to assess the liquefaction potential.

The borings and CPT soundings were analyzed for liquefaction. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soil layers encountered throughout the depths explored (see test boring logs, Appendix B).

The analyses indicate that liquefaction would occur in layers of varying thickness extending to depths of up to about 85 feet BSG at some of the CPT locations. Graphical results of the liquefaction analysis is included in Appendix D.

The total seismic settlements estimated at each boring and CPT location, rounded up to the nearest 1/4 inch, are listed in Table No. 4 below.

Table No. 4
Estimated Total Liquefaction Seismic Settlements from Borings and CPT Soundings

Boring/CPT	Analysis Depth	Estimated Liquefaction Seismic Settlement
B-1	76½ feet BSG	3 inches
B-2	81½ feet BSG	4¼ inches
CPT-3	85 feet BSG	6¾ inches
CPT-4	85 feet BSG	6 inches
SCPT-5	85 feet BSG	6½ inches
CPT-6	52½ feet BSG*	4¾ inches
SCPT-7	85 feet BSG	5½ inches

Notes: 1. Seismic settlement values are rounded up to the nearest 1/4 inch
2. BSG = Below Site Grade surface
* = CPT refusal depth

Due to the close proximity to the ground surface and thickness of liquefiable layers, there is potential for surface manifestation of liquefaction, such as sand boils and loss of bearing pressure for shallow foundations.

In addition, lateral spreading is a potential impact from liquefaction. Due to the existing slopes associated with the adjacent harbor channel which appear to extend to a depth of 35 to 40 feet below the water surface, these adjacent slopes and adjacent areas would be subject to potential seismic instability and lateral spreading in the event of liquefaction.

Due to the potential impacts associated with liquefaction, potential liquefaction mitigation measures are described in Section 7.4 of this report.

7.0 GEOTECHNICAL ENGINEERING EVALUATIONS: The data and methodology used to develop conclusions and recommendations are summarized in the following subsections. The evaluation was based upon the subsurface soil conditions encountered during this investigation and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in the Conclusions section of this report.

7.1 Existing Site Features and Surface Conditions: At the time of our field exploration, the area of the proposed P1 Parking Structure primarily contained asphalt concrete pavements used for vehicle/equipment staging and storage.
improvements including underground utilities, pole mounted lighting, and various stored materials were present. Thus, removal of these existing surface and subsurface features will be required as part of the proposed project. In addition, remnants of former structures such as the previous above ground tanks may also be present within the site, which could include pile foundations. As part of the site preparation, all soils which are disturbed as a result of the removal of the existing surface and subsurface improvements should be excavated to expose undisturbed native soils. In the event existing pile foundations are present, the tops of existing piles should be removed to a minimum depth of 5 feet below the finished grade. In addition, if existing piles are present, the locations would need to be reviewed with regard to the locations of future foundations for the parking structure to address any potential conflicts. All excavations should be backfilled with compacted engineered fill.

7.2 Wet Soils and Shallow Groundwater: As indicated in this report, due to the shallow groundwater conditions, and the presence of near surface, fine-grained soils, wet, unstable soil conditions should be anticipated and will require drying such as by chemical soil treatment prior to being used as engineered fill. Also, it should be anticipated that the base of excavations will require stabilization due to wet, unstable soil conditions. Use of bridging material such as large, crushed cobble with geotextile fabric should be anticipated to bridge soft areas.

Due to shallow groundwater conditions, dewatering and wet soils are potential impacts for site preparation and construction of the project. In addition, buoyancy effects and appropriate waterproofing measures should be considered for design and of subsurface improvements such as pits. For planning purposes, a historic high groundwater depth of about 5 feet is estimated at the site.

7.3 Expansive Soils: One of the potential geotechnical hazards evaluated at this site is the expansion potential of the near surface soils. Over time, expansive soils will experience cyclic drying and wetting as the dry and wet seasons pass. Expansive soils experience volumetric changes (shrink/swell) as the moisture content of the clayey soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade when not designed for the anticipated expansive soil pressures. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. The potential for damage to slabs-on-grade and foundations supported on expansive soils can be reduced by placing non-expansive fill below the slabs-on-grade.

In evaluation of the potential for expansive soils at the site, expansion index testing was performed on a representative sample of the near surface soils which are anticipated to be within the zone of influence of the planned improvements. The expansion index testing was performed in accordance with ASTM D4829 and the results are included in Appendix C of this report. The results of the expansion index (EI) testing indicated that the near surface clayey sample tested exhibited a very low expansion potential (EI = 6).

7.4 Mitigation Approaches and Foundation Support Due to Liquefaction: As indicated in Section 6.4.3 of this report, liquefaction resulting in seismic settlement and partial loss/reduction of foundation bearing support is a potential impact resulting from moderate to strong earthquake ground shaking. In addition, potential impacts from lateral spreading and seismic instability of the adjacent slopes at the channel will need to be considered to determine the extent of the liquefaction mitigation.

Specialty deep ground modification methods are recommended as one option to mitigate the potential impacts associated with liquefaction. The intent of the ground modification design would be to mitigate liquefaction and to allow for support of either a shallow foundation system, or for use of a deep foundation system that would not be impacted by the effects of liquefaction such as downdrag loading and lateral spreading. Due to the height of the structure, structural design for lateral force resistance may require deep foundations for efficiency. The method of deep ground modification which is the most cost effective and compatible with the subsurface soil conditions and adjacent improvements to address liquefaction impacts should be selected for the site. In general, these may include vibro-compaction (stone columns), rigid inclusions and deep cement soil mixing. Typically, a design-build deep ground modification contractor would determine the most appropriate approach based on the subsurface geotechnical engineering data, the specified settlement limits for the improvements, planned grading, the loads associated with proposed structures and improvements and other project constraints.

Deep ground modification methods that result in higher ground vibrations (such as stone columns) would need to be evaluated with regard to adjacent structures. Vibratory installation methods may adversely effect surrounding building structures and improvements by inducing differential settlement.

In addition, the site and surrounding subsurface environmental conditions will need to be considered with regard to selection of the type of deep ground modification. In order to use deep ground modification elements for support of the structure, the foundation design will need to be coordinated with the design of the deep ground modification system.

As an alternative to use of deep ground modification to mitigate the effects of liquefaction, pile foundations relying on deeper soils for support below the liquefiable soils could be considered without the use of deep ground modification. However, due to the significant downdrag loading from deep liquefiable soils as indicated from the CPT data, deep foundation systems would be challenging at this site to address liquefaction related hazards if deep ground modification is not performed. The advantages and disadvantages of the foundation support options would need to be evaluated to determine the preferred approach for support of the structure. It should be noted that pavement improvements which are supported on grade would be subject to the effects of liquefaction if deep ground modification is not conducted.

7.5 Soil Corrosion: The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soils tested are included in the Conclusions section of this report based on the corrosion severity ratings listed in Table No. 5, below.

Table No. 5

Soil Resistivity (ohm cm)	Corrosion Potential Rating
>20,000	Essentially non-corrosive
10,000 - 20,000	Mildly corrosive
5,000 - 10,000	Moderately corrosive
3,000 - 5,000	Corrosive
1,000 - 3,000	Highly corrosive
<1,000	Extremely corrosive

The results of soil sample analyses indicate for samples tested in the area of the proposed building that the near-surface soils exhibit a "corrosive" corrosion potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

The soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

7.6 Sulfate Attack of Concrete: Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil, groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with structural concrete is to perform laboratory testing to determine the concentrations of sulfates present in the soils. The test results are then compared with the exposure classes in Table 19.3.1.1 of ACI 318 to provide guidelines for concrete exposed to soils containing sulfates. It should be noted that other exposure conditions such as the presence of: seawater, groundwater with elevated concentrations of dissolved sulfates, or materials other than soils can result in sulfate exposure categories to concrete that are higher than the concentrations of sulfate in soil. The design engineer will need to determine whether other potential sources of sulfate exposure need to be considered other than exposure to sulfates in

soil. The sulfate exposure classes for soils from Table 19.3.1.1 are summarized in the below Table No.6.

Table No. 6
ACI Exposure Categories for Water Soluble Sulfate in Soils

Sulfate Exposure Class (per ACI 318)	Water Soluble Sulfate in Soil (Percent by Mass)
S0	Less than 0.10 Percent
S1	0.10 to Less than 0.20 Percent
S2	0.20 to Less than or Equal to 2.00 Percent
S3	Greater than 2.00 Percent

Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The laboratory test results for sulfates are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

8.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

Based on the data collected during the field exploration and laboratory testing, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, the following general conclusions and preliminary recommendations are presented.

8.1 In general, the site is considered feasible to construct the proposed multi-story parking structure, provided that the potential impacts of liquefaction are addressed in the site preparation and foundation design as described in this report.

8.2 Descriptions of the subsurface soils encountered in the borings and CPTs conducted for this investigation are included in Section 5.2 of this report. In general, below the asphalt concrete pavements, the auger explorations generally encountered a silty sand fill and poorly graded sand fill soils extending up to a depth ranging from 3 to about 7 feet below site grade (BSG). Underlying the fill soils and where fill was not observed, the native surface soils generally consisted of alternating layers of silty sand and poorly graded sand to depths of about 5 to 7½ feet BSG, with exception that silty

excavated at SCPT-05. The silty sands and poorly graded sands were further underlain by alternating layers of medium stiff to stiff lean clay, silty clay, and very soft fat clay, extending to a depth of 15 to 16 feet BSG. These clay soils were further underlain by alternating layers of medium dense to dense poorly graded sand, medium dense to very dense silty sand, dense clayey sand, hard lean clay, dense poorly graded sand with silt, soft to stiff fat clay and hard silt extending to the maximum depth of the rotary auger borings, 81½ feet BSG.

8.3 The Cone Penetration Tests (CPTs) conducted for this investigation encountered "soil behavior" types that generally correlate with the soil borings. In general, the CPT data generally indicated dense to very dense granular soils with interbedded zones of medium dense granular soils and interbeds of fine grained soft to medium stiff clay and silt of varying thickness throughout the depth explored in the CPTs, to about 100 feet

BSG, the maximum depth explored. Below a depth of about 80 to 85 feet BSG, the CPTs generally indicated very dense granular soils.

8.4 Groundwater was encountered during drilling of the soil borings and CPTs at depths ranging from 9 feet BSG to 10½ feet BSG. These groundwater depths do not represent stabilized groundwater conditions. Shallower groundwater conditions should be anticipated and can be dependant on many factors including changes from tidal influences. Based on the "Seismic Hazard Zone Report for the Oxnard 7.5-Minute Quadrangle, Ventura County, California," dated 2002, the historic high groundwater at the site is shown to be 5 feet.

8.5 Based on the laboratory testing conducted as part of this investigation, the near surface soils exhibit a very low expansion potential. Accordingly, special measures to address an expansive soils condition are not anticipated to be required for the project.

8.6 Due to the fine-grained nature of the near surface soils and the shallow groundwater conditions, storm water infiltration systems are not considered feasible from a geotechnical engineering standpoint.

8.7 The results of soil sample analyses indicate that the near surface soils exhibit a "corrosive" corrosion potential to buried metal objects. If piping or other materials subject to corrosion are planned below the historic high groundwater levels and groundwater levels encountered during our field exploration, special measures should address potential corrosion due to saltwater intrusion.

8.8 Based on Table 19.3.1.1 - Exposure Categories and Classes from Chapter 19 of ACI 318, the sulfate concentrations from chemical testing of soil samples tested as an S0 classification (less than 0.10 percent by weight) for concrete based on the two (2) samples tested. Thus, a sulfate exposure category S0 should be anticipated for the site soil conditions for consideration of concrete mix designs in contact with these soils. The salinity of the groundwater should also be considered in determination of the required concrete mixture characteristics.

8.9 Based on our review of the Federal Emergency Management Agency (FEMA) The Flood Insurance Rate Map, Community Panel number 06111C0914F, distributed by

is located in Zone X, which is defined as an "0.2 percent annual chance flood hazard, areas of 1 percent annual chance flood with average depth less than one foot or with drainage areas of less than one square mile."

8.10 The subject site is located in a region of high seismicity and is subject to strong ground shaking from potential earthquakes on a number of nearby faults. A discussion of local faults is included in 6.2 of this report.

8.11 Based on review of the City of Port Hueneme General Plan Update and Ventura County 2040 General Plan, dam inundation and tsunami are potential geologic hazards. In addition, based on the interactive USGS mapping, the subject project site and entire City of Port Hueneme is located within a mapped area of subsidence due to groundwater pumping. Thus, the potential for regional subsidence should be recognized.

8.12 Liquefiable soils were encountered in the CPTs to significant depths. Refer to Section 6.4.3 of this report. Total seismic settlement due to liquefaction was estimated to range from about 3 to 6 ¾ inches. Due to the potential impacts from liquefaction, structural improvements and other improvements that are intended to be protected from the effects of liquefaction should be addressed by use of deep ground modification and/or

by using pile foundations which are designed considering downdrag loading and effects of lateral spreading due to liquefaction. Special sleeving or other approaches may be necessary to reduce the downdrag loading. The advantages and disadvantages of a deep foundation system compared with deep ground modification would need to be evaluated to determine the preferred approach. Refer to the discussion of potential ground modification methods in Section 7.4 of this report. In the event pile foundations are used, on a preliminary basis, for seismic design purposes, the piles should derive bearing support in the very dense sands encountered in the CPTs below depths of about 80 to 85 feet BSG. Ground modification methods would need to be determined by a specialty geotechnical engineering design-build contractor.

8.13 Due to the potential for liquefaction, the existing channel slopes and adjacent ground are subject to seismic instability such as from lateral spreading, flow failure, etc. However, due to the distance between the parking structure and the existing slopes, and considering that the parking structure foundations will be either be supported on specialty deep ground modification or deep foundations, these measures will be designed to reduce potential impacts from the seismic slope instability. Where deep ground modification is conducted, the extent of deep ground modification may need to be extended adjacent to the parking structure to provide protection from lateral spreading associated with the adjacent slopes.

8.14 The average shear wave velocity data from the CPT soundings indicated a Site Class DE category based on ASCE 7-22. However, due to liquefaction, a Site Class F category should be applied for any applicable design conditions other than the mapped spectral acceleration parameters included in the table below, which were determined considering a Site Class DE and assuming the structure will have a fundamental period of vibration of less than 0.5 seconds, unless a more stringent requirement was applied to
structures with a fundamental period of 0.5 seconds or higher.

The following mapped seismic acceleration factors were developed for the site using the Ground Motion Parameter Calculator ASCE Hazard (https://ascehazardtool.org).

Table No. 7
Mapped Seismic Factors

Mapped Seismic Factors (ASCE 7-22)*	
Site Class	DE*
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.67
Spectral Response At Short Period (0.2 Second), Ss	1.82
Spectral Response At 1-Second Period, S_1	0.73
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	1.86
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	1.97
Five percent damped design spectral response accelerations for short period, S_{DS}	1.24
Five percent damped design spectral response accelerations at 1-second period, S_{D1}	1.31

* - Only applicable for determining the mapped seismic coefficients for structures with a fundamental period less than 0.5 seconds.

Notes: 1. Values in above table reflect the "mapped" seismic parameters.
2. The mapped seismic parameters are subject to the disclaimers listed in the ASCE Hazard Tool website
3. S_{DS} value determined in accordance with ASCE 7-22, Section 11.4.4.

9.0 DESIGN CONSULTATION

9.1 Moore Twining should be retained to review those portions of the contract drawings and specifications that pertain to earthwork and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.

9.2 If Moore Twining is not retained for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

10.0 CONSTRUCTION MONITORING

10.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

10.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in

accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

10.3 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

10.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

10.5 If Moore Twining is not retained to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance satisfactory to the owner and review this report. After their review, the firm should, in writing, state that they understand the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

10.6 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to prepare a final report summarizing the observations during site preparation activities relative to the recommendations of this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions.

11.2 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

11.3 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (more than 12

months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

11.4 Changed site conditions, or relocation of proposed structure(s), may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

11.5 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structure on this site not discussed herein is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

11.6 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

11.7 This report presents the results of a geotechnical engineering investigation and geologic/seismic hazards investigation only and should not be construed as an environmental audit or study.

11.8 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

11.9 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

12.0 CLOSING

We appreciate the opportunity to be of service. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

Shaun Reich, EIT
Project Engineer

Allen H. Harker, CEG
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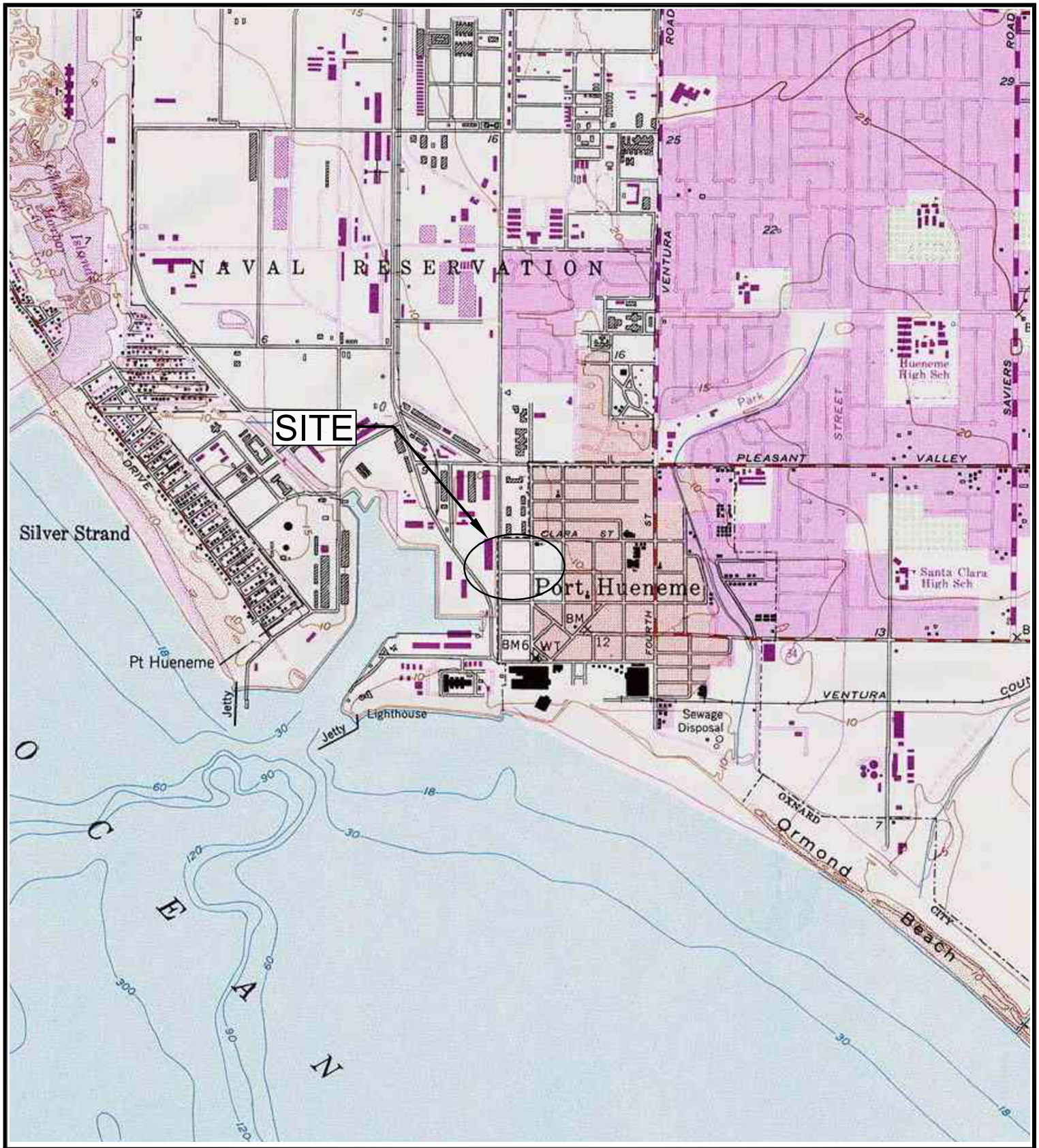
USGS website (https://earthquake.usgs.gov/earthquakes/search/), Search Earthquake Catalog for digitized California historical earthquake catalogs

United States Geological Survey Earthquake Hazard Toolbox website (https://earthquake.usgs.gov/nshmp/) NSHM Conterminous U.S. 2018 model

APPENDIX A

DRAWINGS

Drawing No. 1 - Site Location Map
Drawing No. 2 - Test Boring Location Map
Drawing No. 3 - Regional Geologic Map
Drawing No. 4 - Map of Major Active and Potentially Active Faults Relative to Site
Drawing No. 5 - Historical Earthquake Epicenter Map
Drawing No. 6 - Site Geologic Map

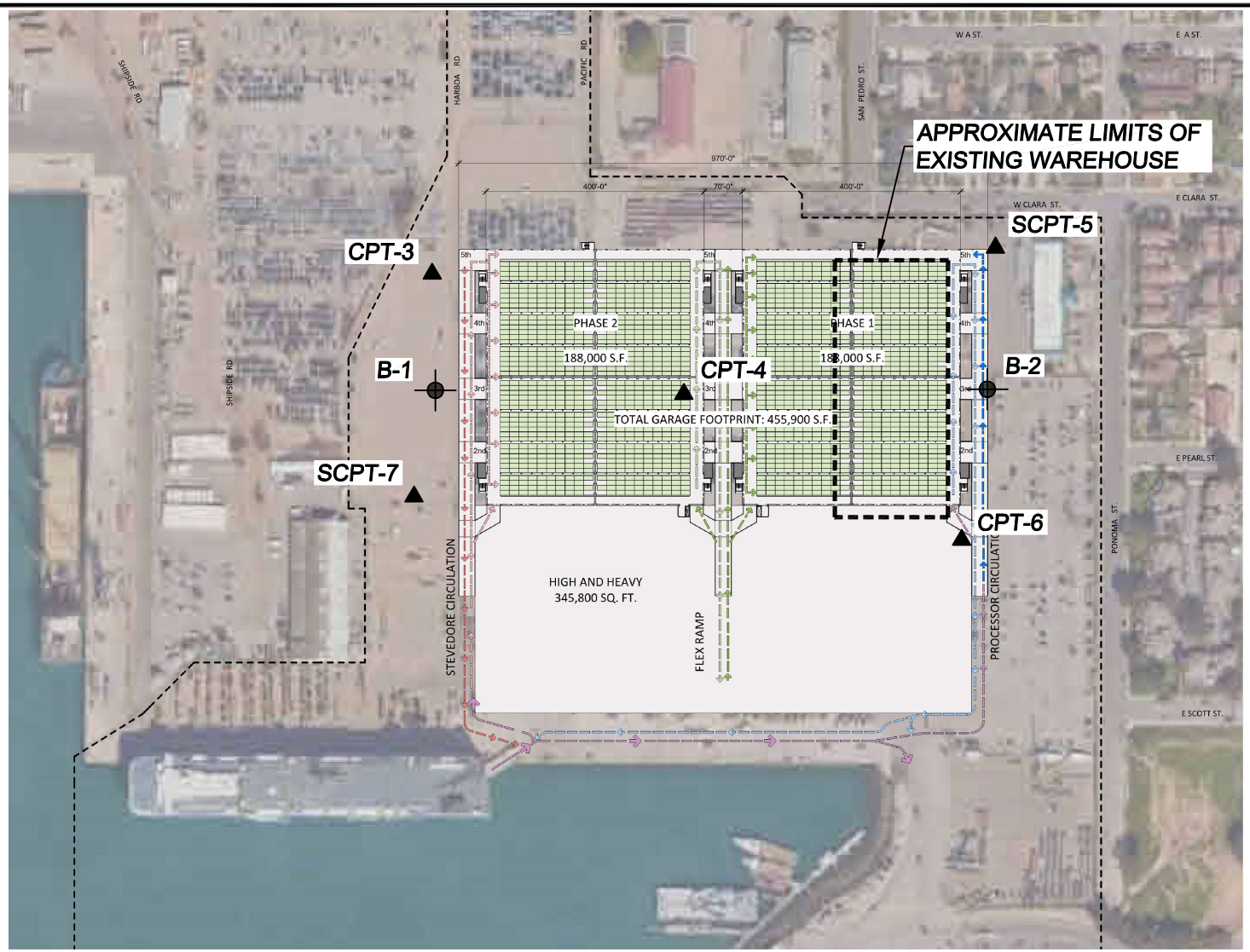


SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
OXNARD, CALIFORNIA QUADRANGLE 1967

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP P1 PARKING STRUCTURE PROJECT OXNARD HARBOUR DISTRICT PORT OF HUENEME VENTURA COUNTY, CALIFORNIA	FILE NO.: 93908-01-01 DRAWN BY: RM PROJECT NO. G93908.01	DATE: 02/25/2026 APPROVED BY: DRAWING NO. 1

MOORE TWINING ASSOCIATES, INC.



APPROXIMATE TEST BORING LOCATION

APPROXIMATE LOCATION OF CPT SOUNDING (CPT)
AND SEISMIC CPT SOUNDING (SCPT)

N

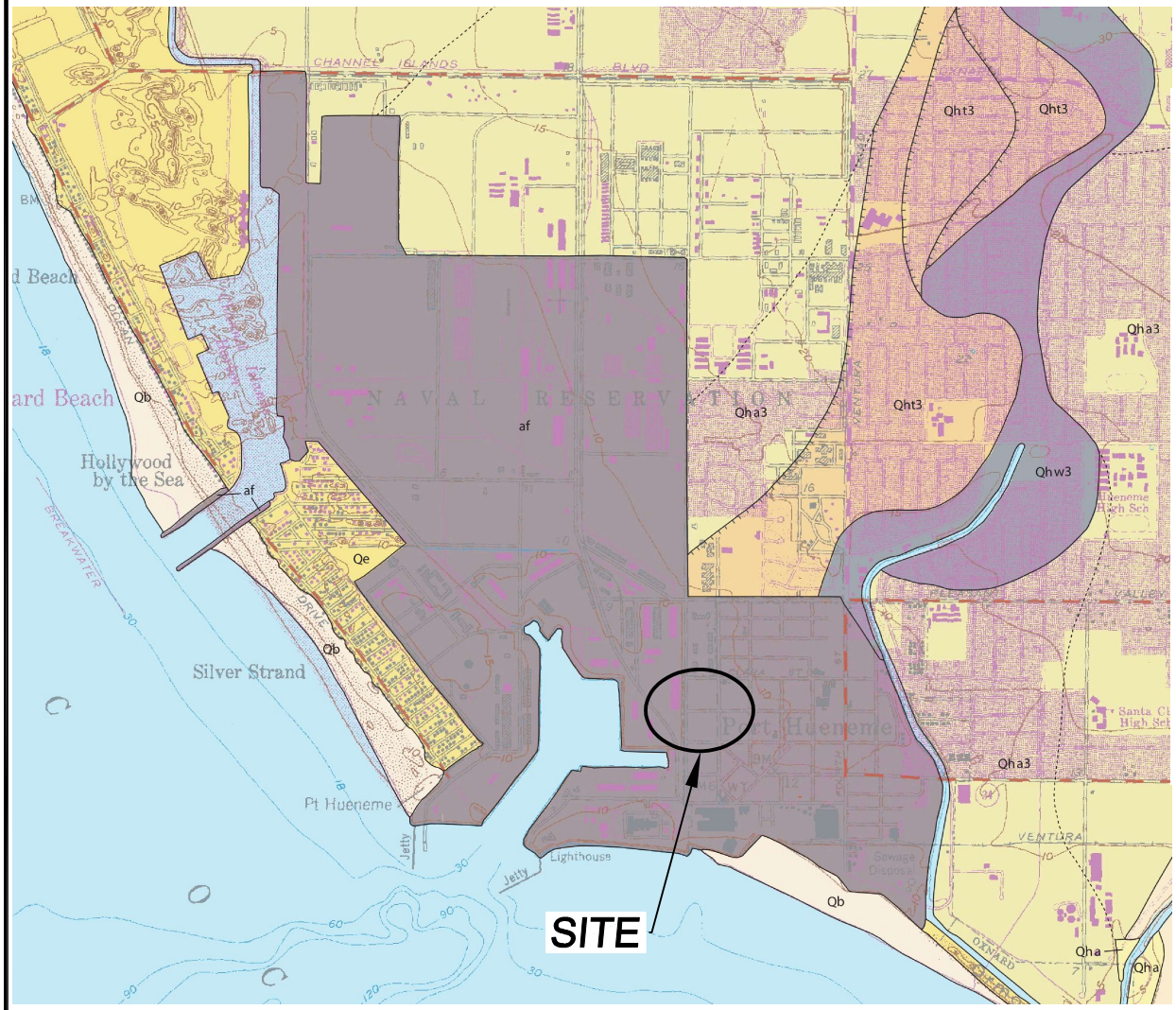
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APPROXIMATE SCALE
IN FEET

TEST BORING AND CPT LOCATION MAP
P1 PARKING STRUCTURE PROJECT
OXNARD HARBOUR DISTRICT
PORT OF HUENEME
VENTURA COUNTY, CALIFORNIA

FILE NO. 93908-01-02	DATE DRAWN: 07/08/2026
DRAWN BY: RM	APPROVED BY:
PROJECT NO. G93908.01	DRAWING NO. 2

MOORE TWINING ASSOCIATES, INC.
Since 1898



af	Artificial fill; may be engineered and/or non-engineered.
alf	Artificial levee fill; may be engineered and/or non-engineered.
Qw	Active wash deposits within major river channels; composed of unconsolidated silt, sand and gravel.
Qb	Active beach deposits; composed mainly of loose sand, well-sorted, fine-to coarse-grained.
Qe	Active coastal eolian (sand dune) deposits, consist of loose sand and silt.
Qes	Active coastal estuarine deposits; composed of submerged/saturated silty clay.
Qht	Latest Holocene stream terrace deposits, deposited in point bar and overbank settings associated with unit Qw; composed of unconsolidated, poorly sorted, clayey sand and sandy clay with gravel.
Qha	Latest Holocene alluvial deposits, deposited as overbank material associated with unit Qw, recognized by scour and incised channeling features; composed of unconsolidated, poorly sorted, clayey sand with some gravel. May include terrace deposits (Qht).
Qhw_3	Holocene wash deposit; composed of unconsolidated sand, silt and gravel.
Qht_3	Holocene stream terrace deposits, deposited in point bar and overbank settings associated with unit Qhw_3; composed of unconsolidated clayey sand and sandy clay with gravel.
Qha_3	Holocene alluvial deposits, deposited as overbank material associated with unit Qhw_3, recognized by scour and incised channeling features; composed of unconsolidated, poorly sorted, clayey sand with some gravel.

REFERENCE: GEOLOGIC MAP OF THE OXNARD 7.5-MINUTE QUADRANGLE, VENTURA COUNTY, CALIFORNIA, A DIGITAL DATABASE, DATED 2003, PREPARED BY KEVIN B. CLAHAN

N

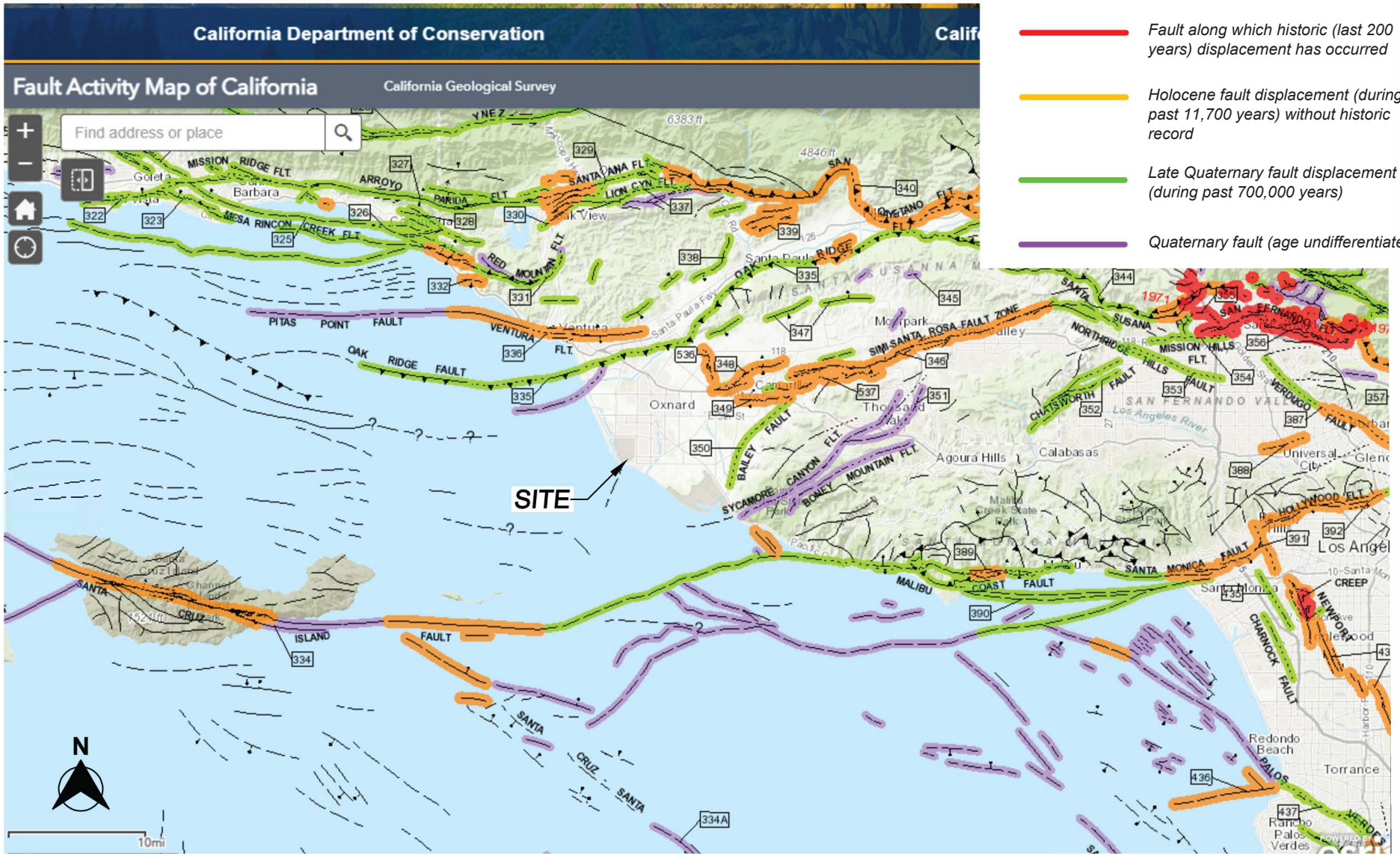
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APPROXIMATE SCALE IN MILES

REGIONAL GEOLOGIC MAP
P1 PARKING STRUCTURE PROJECT
OXNARD HARBOUR DISTRICT
PORT OF HUENEME
VENTURA COUNTY, CALIFORNIA

FILE NO. 93908-01-02	DATE DRAWN: 02/26/2026
DRAWN BY: RM	APPROVED BY:
PROJECT NO. G93908.01	DRAWING NO. 3

MOORE TWINING ASSOCIATES, INC.
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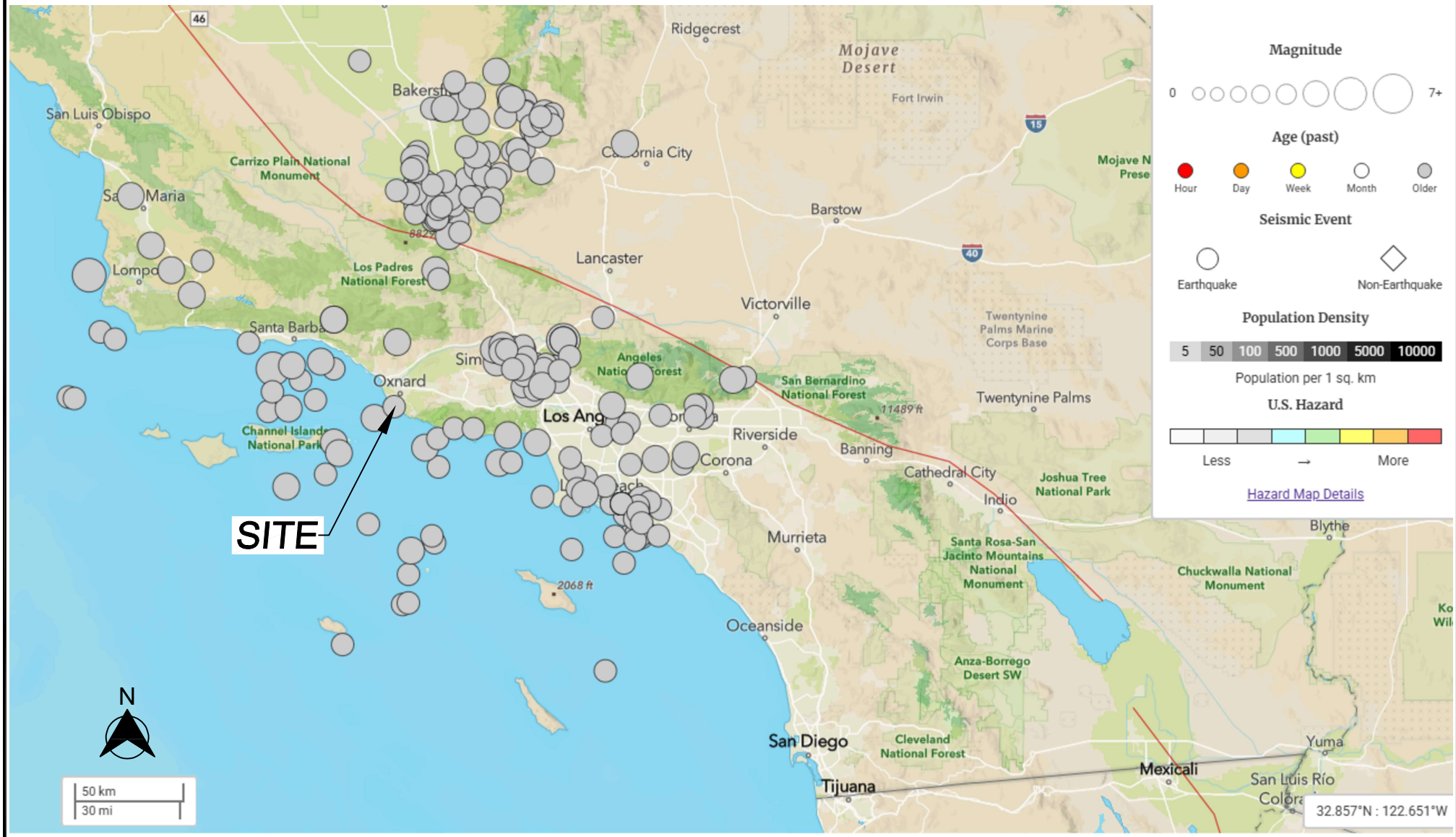
LEGEND

- *Fault along which historic (last 200 years) displacement has occurred*
- *Holocene fault displacement (during past 11,700 years) without historic record*
- *Late Quaternary fault displacement (during past 700,000 years)*
- *Quaternary fault (age undifferentiated)*

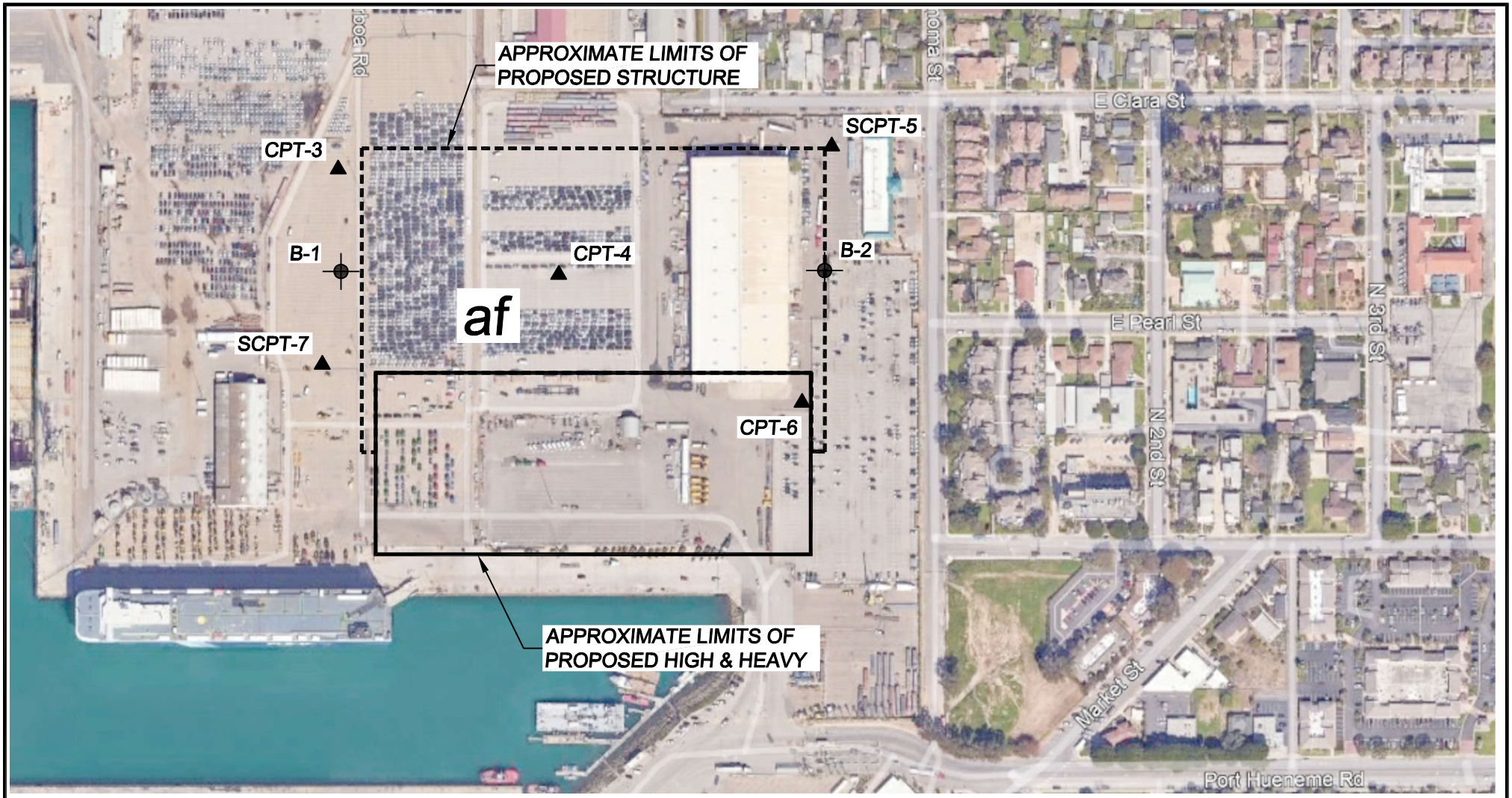
MAP OF ACTIVE AND POTENTIALLY ACTIVE FAULTS RELATIVE TO SITE
P1 PARKING STRUCTURE PROJECT
OXNARD HARBOUR DISTRICT
PORT OF HUENEME
VENTURA COUNTY, CALIFORNIA

FILE NO. 93908-01-02	DATE DRAWN: 02/26/2026
DRAWN BY: RM	APPROVED BY:
PROJECT NO. G93908.01	DRAWING NO. 4

MOORE TWINING ASSOCIATES, INC.
Since 1898



HISTORICAL EARTHQUAKE EPICENTER MAP P1 PARKING STRUCTURE PROJECT OXNARD HARBOUR DISTRICT PORT OF HUENEME VENTURA COUNTY, CALIFORNIA	FILE NO. 93908-01-02	DATE DRAWN: 02/26/2026	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. G93908.01	DRAWING NO. 5	



APPROXIMATE TEST BORING LOCATION

APPROXIMATE LOCATION OF CPT SOUNDING (CPT) AND SEISMIC CPT SOUNDING (SCPT

af : ARTIFICIAL FILL; MAY BE ENGINEERED AND/OR NON-ENGINEERED

N

0 80

APPROXIMATE SCALE IN FEET

REFERENCE: GEOLOGIC MAP OF THE OXNARD 7.5-MINUTE QUADRANGLE, VENTURA COUNTY, CALIFORNIA, A DIGITAL DATABASE, DATED 2003, PREPARED BY KEVIN B. CLAHAN

SITE GEOLOGIC MAP
P1 PARKING STRUCTURE PROJECT
OXNARD HARBOUR DISTRICT
PORT OF HUENEME
VENTURA COUNTY, CALIFORNIA

FILE NO. 93908-01-02	DATE DRAWN: 07/08/2026
DRAWN BY: RM	APPROVED BY:
PROJECT NO. G93908.01	DRAWING NO. 6

MOORE TWINING ASSOCIATES, INC.
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APPENDIX B

LOG OF ROTARY AND CPT TEST BORINGS AND GREGG DRILLING CPT REPORT

This appendix contains the final boring logs. The logs represents our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at the test boring locations. Also, the passage of time may result in changes in the soil conditions.

An explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Boring and CPT logs. The Gregg Drilling CPT report is provided at the end of Appendix B, providing the data gathered during the CPT fieldwork.

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

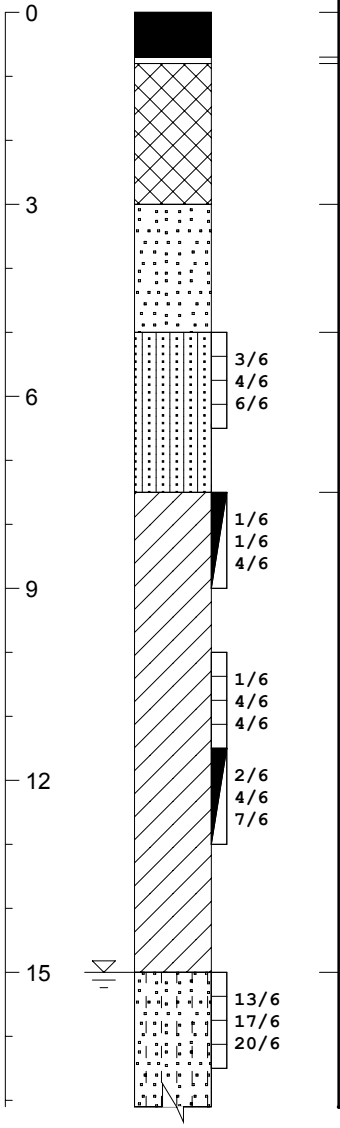
Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 17, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 15 feet



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
AC	ASPHALTIC CONCRETE = 8 inches			
AB	AGGREGATE BASE = 1.6 inches			
FILL	At 0.8 feet - FILL - SILTY SAND; moist, fine-to-medium grained, brown	From 1-4': pH = 8.2 SR = 5,200 ohm-cm Cl = 0.0010% SS = 0.0034%		
SP	At 3 feet - POORLY GRADED SAND; moist, fine-to-medium grained, brown			
SM	At 5 feet - SILTY SAND WITH GRAVEL; loose, moist, fine-to-coarse grained, brown, predominantly fine gravel, trace coarse gravel	From 5-6.5': DD = 109.2 pcf Gravel = 16.6% Sand = 65.7% -200 = 17.7%	10	8.5
CL	At 6 feet - Increased moisture, dark brown	From 7.5-9': PP = 0.5 tsf Sand = 15.2% -200 = 84.8% LL = 32 PI = 15	5	37.4
	At 7.5 feet - LEAN CLAY WITH SAND; medium stiff, high moisture, medium plasticity, brown			
	At 10 feet - Slight plasticity, increased fine grained sand content, trace silt	From 10-11.5' DD = 92.2 pcf PP = 0.75 tsf	8	31.0
	At 11.5 feet - Stiff, low plasticity	From 11.5-13' PP = 0.75 tsf	11	29.4
SP-SM	At 15 feet - POORLY GRADED SAND WITH SILT; medium dense, wet, fine to medium grained, brown, trace fine gravel	From 15-16.5': DD = 101.2 pcf Gravel = 2.6% Sand = 89.0%	37	20.9

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 17, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 15 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
18				-200 = 8.4% ø = 41° c = 400 psf		
21	9/6 15/6 17/6		At 20 feet - Dense, medium-to-coarse grained wiht fine grained sand		32	14.4
24						
27	15/6 16/6 18/6	SC	CLAYEY SAND WITH GRAVEL; medium dense, wet, fine-to-coarse grained sand, gray, gravel is fine gravel	From 25-26.5': DD = 117.1 pcf Gravel = 22.7% Sand = 65.0 -200 = 12.3% LL = 25 PI = 10	34	10.1
30	8/6 11/6 13/6	SM	SILTY SAND; medium dense, wet, medium-to-coarse grained, trace fine gravel		24	16.7
33						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 17, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 15 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
36	11/6 27/6 47/6	SC	CLAYEY SAND; dense, wet, fine-to-coarse grained sand, medium-to-high plasticity, gray	From 35-36.5': DD = 110.2 pcf Gravel = 27.8% Sand = 55.8% -200 = 16.4%	74	17.0
39						
42	5/6 22/6 26/6	SP-SM	POORLY GRADED SAND WITH SILT; dense, wet, medium-to-coarse grained with fine grained sand, with fine gravel, gray		48	17.7
45	14/6 33/6 51/6	SM	SILTY SAND; very dense, wet, fine-to-medium grained, some coarse grained sand, gray	DD = 110.0 pcf	84	16.0
48						
51	15/6 20/6 24/6		Dense		44	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 17, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 15 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
54	11/6 24/6 46/6	SC	CLAYEY SAND; dense, wet, fine-to-medium grained, low-to--medium plasticity, gray	DD = 91.4 pcf	70	35.2
57						
60	18/6 26/6 25/6	SM	SILTY SAND; very dense, wet, fine-to-medium grained, gray		51	
63						
66	6/6 8/6 12/6	CH	FAT CLAY; stiff, moist, very high plasticity, olive-brown	From 65-66.5': DD = 79.2 pcf PP = 1.5 tsf LL = 64 PI = 40	20	38.8

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 17, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 15 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
69						
	6/6 13/6 14/6	SM	SILTY SAND; medium dense, wet, fine-to-medium grained, gray-brown		27	
72						
75	25/6 37/6 34/6	ML	SANDY SILT; hard, moist, with fine grained sand, dark brown to black	DD = 92.5 pcf PP = 0.5 tsf	71	32.3
			Bottom of boring B-1 at 76.5 feet BSG.			
78						
81						
84						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

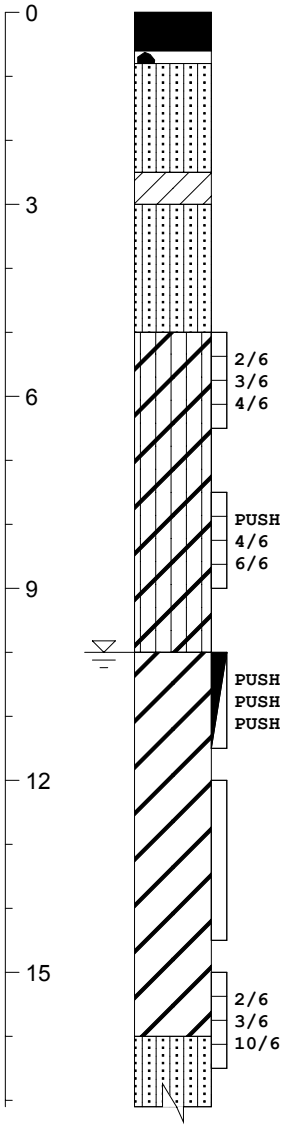
Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 18, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 10 Feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		AC	ASPHALTIC CONCRETE = 7 inches			
		AB	AGGREGATE BASE = 3.1 inches			
		SM	At 0.8 feet - SILTY SAND; moist, fine grained, dark brown, trace fine gravel	From 1-4': Gravel = 0.8% Sand = 71.5% -200 = 27.7% pH = 8.1 SR = 3,300 ohm-cm Cl < 0.0060% SS = 0.011%		
		CL	At 2.5 feet - LEAN CLAY; moist, low-to-medium plasticity, dark brown			
		SM	At 3 feet - SILTY SAND; moist, fine-to-medium grained, dark brown		7	17.7
		CL-ML	At 5 feet - SANDY SILTY CLAY; medium stiff, moist, low plasticity, dark brown	From 5-6.5': DD = 101.0 pcf PP = 0.5 tsf LL = 22 PI = 6 ø = 36° c = 70 psf	10	32.1
			At 7.5 feet - Increase in fine grained sand content	From 7.5-9': DD = 93.5 pcf PP = 0.5 tsf Sand = 38.6% -200 = 61.4%	0	44.9
		CH	At 10 feet - FAT CLAY; very soft, very moist, high plasticity, brown	From 10-11.5': PP = 0 tsf LL = 53 PI = 34		14.5
				From 12-14.5' 0 psi push pressure EI = 88		
				From 15-16.5 DD = 85.3 pcf	13	36.0
		SM	SILTY SAND; loose, moist, fine-to-medium grained, light brown, trace			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 18, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 10 Feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
18	5/6 13/6 14/6		fine gravel At 17.5 feet - Medium dense	From 17.5-19': Gravel = 0.1% Sand = 87.0% -200 = 12.9%	27	22.8
21	16/6 23/6 27/6		Dense	DD = 110.3 pcf	50	16.0
24	10/6 15/6 18/6				33	21.5
27						
30	17/6 28/6 42/6			DD = 103.2 pcf	70	20.2
33						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 18, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 10 Feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
36	13/6 17/6 24/6				41	
39		ML	SILT; wet, non-plastic, brown	PP = 1.25 tsf LL = Non-viscous PI = Non-plastic	37	31.0
42	7/6 13/6 24/6	SM	SILTY SAND; medium dense, wet, fine grained, trace medium grained, brown Alternating lenses of high plasticity clay	From 41-41.5': DD = 93.4 pcf		28.3
45	14/6 19/6 24/6	CL	LEAN CLAY; hard, wet, high plasticity, brown	From 45-45.5': LL = 42 PI = 25	43	
48						
51	9/6 14/6 47/6	SC	CLAYEY SAND; dense, wet, fine-to-medium grained sand, medium	DD = 104.6 pcf PP = 0.75 tsf	61	18.3

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 18, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 10 Feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
54			plasticity, brown			
57	21/6 47/6 32/6	SP-SM	POORLY GRADED SAND WITH SILT; very dense, wet, medium grained with fine grained sand, gray		79	
60	21/6 42/6 50/4		Thin (2-inch) lenses of olivish, medium-to-high plasticity clay, with seashell fragments	DD = 105.1 pcf	>50	19.1
63						
66	19/6 26/6 38/6		Alternating layers of fine grained silty sand and medium grained poorly graded sand with seashell debris		64	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Mobile B54 Model D3

Auger Type: Mud Rotary

Hammer Type: Auto-trip, 140lb. 30in. drop

Logged By: S.R.

Date: December 18, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: 10 Feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
69	32/6 50/5	SP	POORLY GRADED SAND; very dense, wet, medium-to-coarse grained, light gray		>50	16.3
72						
75	PUSH PUSH 4/6	CH	FAT CLAY; soft, wet, high plasticity, dark olive	From 75-76.5': PP = 0.75 tsf LL = 61 PI = 39	4	
78						
	30/6 46/6 50/5			PP = 0.5 tsf	>50	28.7
81		SP	POORLY GRADED SAND; very dense, wet, fine-to-medium grained, trace silt, light gray			20.3
			Bottom of boring B-2 at 81.4 feet BSG.			
84						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: CPT-03

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Logged By: S.R.

Date: December 16, 2025

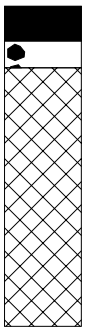
Drill Type: Hand Auger

Elevation:

Auger Type: 4-1/2 inch O.D. Hand Auger

Hammer Type: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	ASPHALTIC CONCRETE = 6.5 inches			
		AB	AGGREGATE BASE = 5.0 inches			
		FILL	@ 0.96 feet - FILL - SILTY SAND; moist, fine to medium grained sand with fine to medium gravel, dark brown	Gravel = 9.5% Sand = 78.1% -200 = 12.4%		
3		FILL	@ 3.0 feet - FILL - POORLY GRADED SAND; moist, fine to medium grained sand with trace fine gravel, light brown			
		FILL	@ 5.0 feet - FILL - SILTY SAND; increased moisture, fine to medium grained sand, dark brown, with petroleum odor			
6			Bottom of CPT-03 Hand Auger Excavation at 5.0 feet BSG to Verify Clearance from Owner-Installed Underground Utility. CPT-03 Sounding Advanced to a Target Depth of 100.07 feet BSG. Groundwater Tagged at time of CPT-03 Exploration at 10.3 Feet BSG.			
9						
12						
15						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: CPT-04

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Hand Auger

Auger Type: 4-1/2 inch O.D. Hand Auger

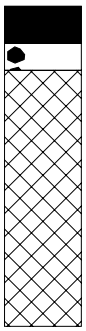
Hammer Type: N/A

Logged By: S.R.

Date: December 16, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	ASPHALTIC CONCRETE = 7.0 inches			
		AB	AGGREGATE BASE = 5.0 inches			
		FILL	@ 1.0 feet - FILL - SILTY SAND; moist, fine to medium grained sand, dark brown light brown			
3			@ 4.25 feet - FILL - SILTY SAND; increased moisture, fine to medium grained sand, dark brown, with petroleum odor			
6			Bottom of CPT-04 Hand Auger Excavation at 4.25 feet BSG to Verify Clearance from Owner-Installed Underground Utility. CPT-04 Sounding Advanced to a Refusal Depth of 90.22 feet BSG. Groundwater Tagged at time of CPT-04 Exploration at 9.0 Feet BSG.			
9						
12						
15						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: SCPT-05

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Hand Auger

Auger Type: 4-1/2 inch O.D. Hand Auger

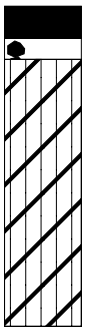
Hammer Type: N/A

Logged By: S.R.

Date: December 15, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	ASPHALTIC CONCRETE = 6.0 inches	PI = 6 LL = 21 EI = 6		
		AB	AGGREGATE BASE = 4.0 inches			
		CL-ML	@ 0.83 feet - SILTY CLAY; moist, fine with trace medium grained sand, dark brown, low plasticity			
3						
6			Bottom of CPT-05 Hand Auger Excavation at 5.0 feet BSG to Verify Clearance from Owner-Installed Underground Utility. CPT-03 Sounding Advanced to a Target Depth of 100.0 feet BSG.			
9						
12						
15						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: CPT-06

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Hand Auger

Auger Type: 4-1/2 inch O.D. Hand Auger

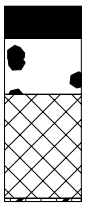
Hammer Type: N/A

Logged By: S.R.

Date: December 15, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	ASPHALTIC CONCRETE = 7.5 inches	Gravel = 39.9% Sand = 48.3% -200 = 11.8%		
		AB	AGGREGATE BASE = 9.0 inches			
		FILL	@ 1.375 feet - FILL - SILTY SAND; moist, fine to medium grained sand with fine gravel, brown			
3		CL-ML	@ 3.0 feet - LEAN CLAY; moist, fine with trace medium grained sand, grey- brown, low plasticity			
6			Bottom of CPT-06 Hand Auger Excavation at 3.0 feet BSG to Verify Clearance from Owner-Installed Underground Utility. CPT-06 Sounding Advanced to a Refusal Depth of 52.56 feet BSG. Groundwater Tagged at time of CPT-06 Exploration at 10.5 Feet BSG.			
9						
12						
15						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: SCPT-07

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Project Number: G93908.01

Drilled By: Gregg Drilling, LLC.

Drill Type: Hand Auger

Auger Type: 4-1/2 inch O.D. Hand Auger

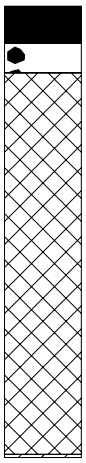
Hammer Type: N/A

Logged By: S.R.

Date: December 15, 2025

Elevation:

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	ASPHALTIC CONCRETE = 7.0 inches	Gravel = 44.1% Sand = 45.7% -200 = 10.2%		
		AB	AGGREGATE BASE = 5.5 inches			
3		FILL	@ 1.05 feet - FILL - SILTY SAND; moist, fine to medium grained sand with fine to medium gravel, dark brown			
		FILL	@ 2.0 feet - FILL - POORLY GRADED SAND; moist, fine to medium grained sand with trace fine gravel, light brown			
6		CL	@ 7.0 feet - SAND LEAN CLAY; increased moisture, with fine grained sand, dark brown, with low plasticity			
9			Bottom of CPT-07 Hand Auger Excavation at 7.0 feet BSG to Verify Clearance from Owner-Installed Underground Utility. CPT-07 Sounding Advanced to a Target Depth of 100.0 feet BSG. Groundwater Tagged at time of SCPT-07 Exploration at 10.3 Feet BSG.			
12						
15						

Notes: *CPT-07 Excavation Left Open Overnight, Groundwater Observed at a Depth of 10.4 feet BSG the Following Morning*

Figure Number

KEY TO SYMBOLS

Symbol	Description
	Strata symbols
	Asphalt concrete
	Aggregate base
	Fill
	Poorly Graded Sand
	Silty Sand

Symbol	Description
	Lean Clay
	Poorly Graded Sand with Silt
	Clayey Sand
	Fat clay
	Silt

Notes:

1. Exploratory borings were drilled on December 17 & 18, 2025 using a Mobile B54 Model D3 drill rig equipped with 8-inch O.D. hollow stem auger to a depth of about 10-feet, followed by a 4-inch rotary wash bit. CPT locations were hand augered to depths ranging from 3 to 7 feet for purposes of utility clearance.

2. Groundwater was encountered during drilling of some borings, see logs.

3. Boring locations were measured or paced from existing features.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. The "N-value" reported for the California Modified Split Barrel Sampler is the uncorrected field blow count. This value should not be interpreted as an SPT equivalent N-value.

6. Results of tests conducted on samples recovered are reported on the logs.

DD =	Natural dry density (pcf)	LL =	Liquid Limit (%)
+4 =	Percent retained on the No. 4 sieve(%)	PI =	Plasticity Index (%)
-200 =	Percent passing the No. 200 sieve (%)	EI =	Expansion Index
Sand =	Percent passing the No. 4 and retained on No. 200 sieves(%)	Gravel =	Percent passing 3-inch & retained on No. 4 sieve(%)
SR =	Soil resistivity (ohms-cm)	pH =	Soil pH
ø =	Internal Angle of Friction (degrees)	c =	Cohesion (psf)
pcf =	Pounds per cubic foot	psf =	Pounds per square foot
O.D. =	Outside diameter	AMSL =	Above mean sea level
N/A =	Not applicable	N/E =	Not encountered
BSG =	Below Site Grade Elevation	NV =	Non-Viscous
NP =	Non-Plastic	ND =	Not Detected (<0.0060%)
LOI =	Loss-On-Ignition (%)	PP =	Pocket Penetrometer (tsf)

KEY TO SYMBOLS

Symbol Description

Strata symbols



Sandy, Silty Clay

Misc. Symbols



Water table during drilling



Boring continues

Soil Samplers



California Modified split barrel ring sampler



Standard penetration test



Undisturbed thin wall Shelby tube

CONE PENETRATION TESTING (CPT) REPORT

Gregg Drilling LLC

Prepared for: Moore Twining
Project D1255059
December 17, 2025

Prepared by: Eleni Pateras
epateras@greggdrilling.com

Table of Contents

Cone Penetration Testing (CPT) Procedure 3
15cm^2 Standard Cone Specifications 4
Cone Penetration Test Data & Interpretation 5
Pore Pressure Dissipation Tests (PPDTs) 6
Seismic Cone Penetration Tests (SCPT) 7
Soil Sampling 8
Ground Water Sampling 9
References 10
Table 1: Cone Penetration Testing Summary 11

APPENDIX A: Cone Penetration Test Plots

APPENDIX B: Pore Pressure Dissipation Test Plots

APPENDIX C: Seismic Plots & Tables

December 17, 2025

Moore Twining
Attn: Shaun Reich

Subject: CPT Site Investigation
Port of Hueneme
Port of Hueneme, CA
GREGG Project Number: D1255059

Dear Shaun:

The following report presents the results of Gregg Drilling's Cone Penetration Test investigation for the above referenced site. The following testing services were performed.

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☒
4	Groundwater Samples	(GWS)	☐
5	Soil Samples	(SS)	☐
6	Vapor Samples	(VS)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to email me at reports@greggdrilling.com.

Sincerely,

CPT Reports Team
Gregg Drilling, LLC.

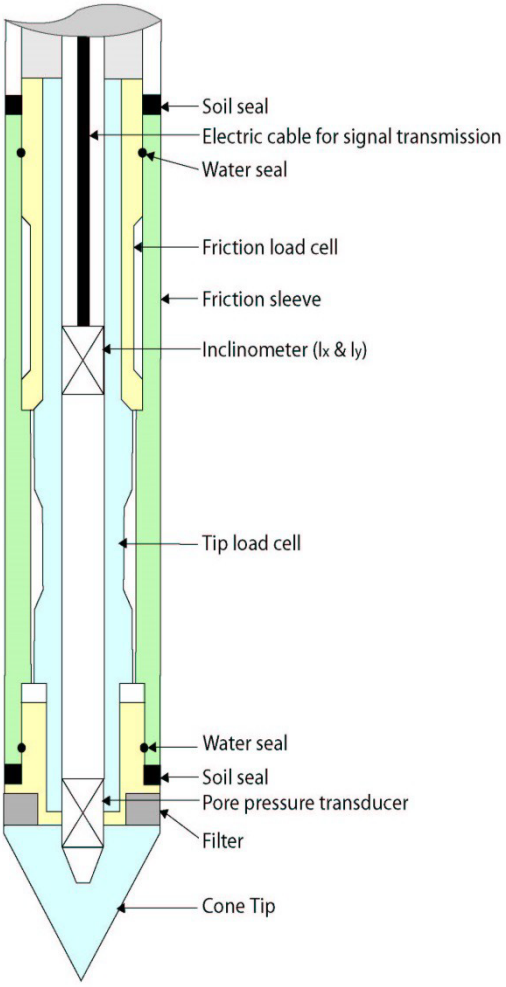
Cone Penetration Testing (CPT) Procedure

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, Figure CPT.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above-mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (PPDT). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.



15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.75
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (2010) (Figure SBT). Typical plots display SBT based on the non-normalized charts of Robertson (2010) or normalized data (2009 and 2016). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (2009 and 2016) which can be displayed as SBTn. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Robertson and Cabal (Guide to Cone Penetration Testing 7th Edition, 2022). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling LLC does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

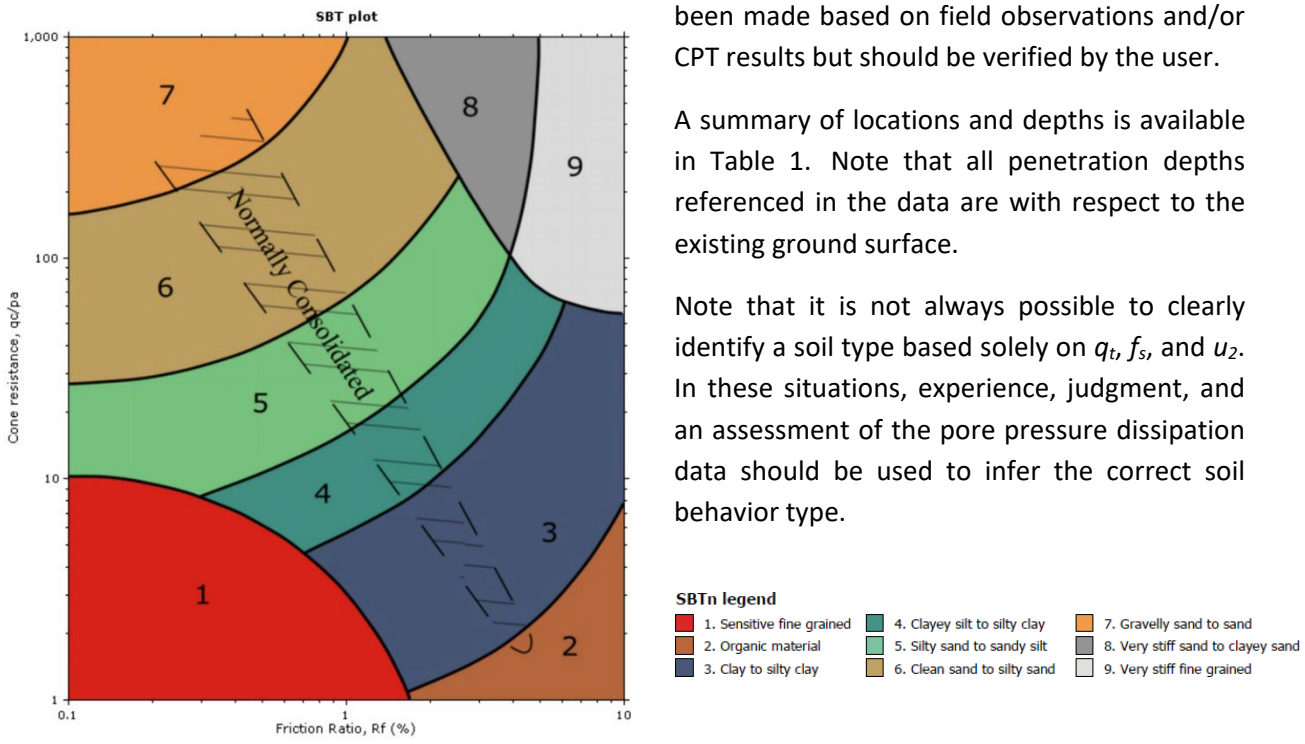


Figure SBT (After Robertson 2010) – Note: Colors may vary slightly compared to plots

Pore Pressure Dissipation Tests (PPDTs)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u)* with time is measured behind the tip of the cone and recorded.

Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

To correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests completed for this project is included in Table 1.

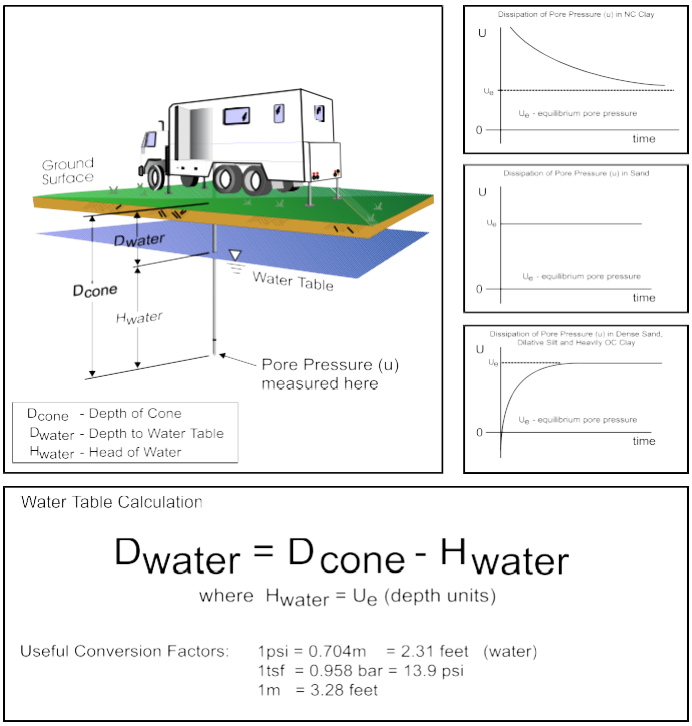


Figure PPDT

Seismic Cone Penetration Tests (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg Drilling's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: v = Δd/Δt

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary of the shear wave velocities, arrival times and wave traces are provided with the report.

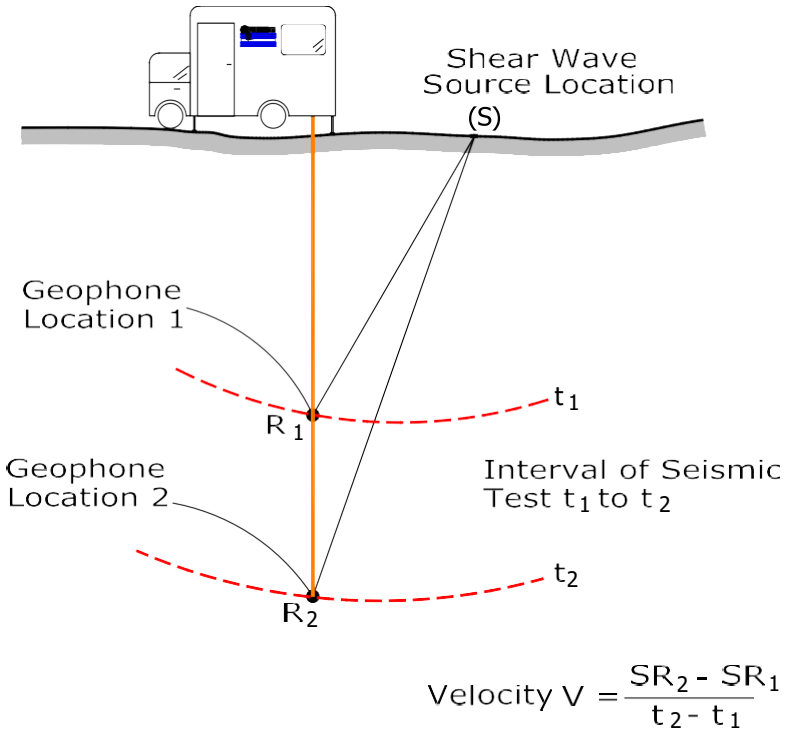


Figure SCPT

Soil Sampling

Gregg Drilling uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

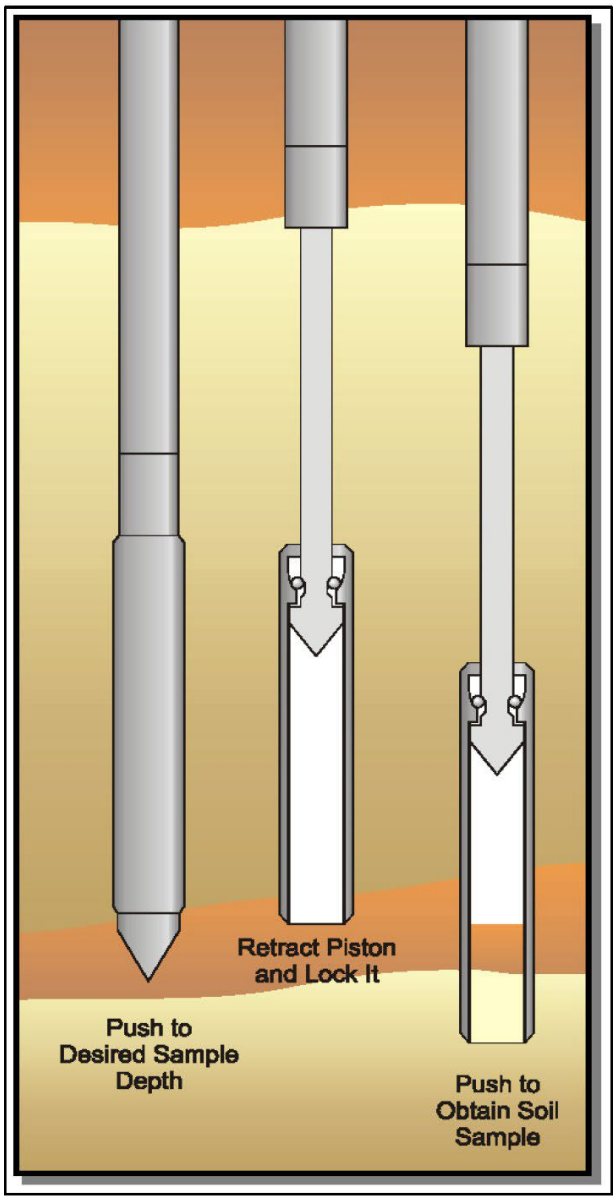


Figure SS

Ground Water Sampling

Gregg Drilling conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

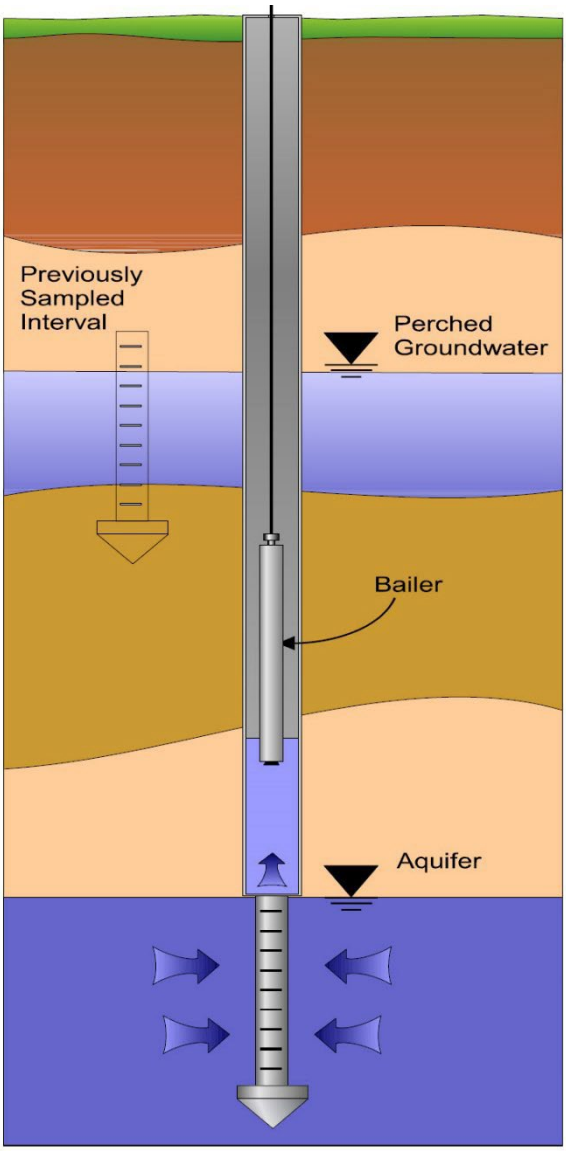


Figure GWS

References

Lunne, T., Robertson, P.K. and Powell, J.J.M., “Cone Penetration Testing in Geotechnical Practice” E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., “Soil Classification using the Cone Penetration Test”, Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

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Robertson, P.K., 2010, “Soil Behavior type from the CPT: an update”, 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, Vol.2. pp 575-583

Robertson, P.K., 2016. “CPT-based Soil Behaviour Type (SBT) Classification System – an update”. Canadian Geotechnical Journal, 53(12); pp 1910-1927

Robertson, P.K. and Cabal, K.L., “Guide to Cone Penetration Testing for Geotechnical Engineering”, 6th Edition, 2015, 145 p. Free online, http://www.greggdrilling.com/technical-guides.

Mayne, P.W., “NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization”, available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, “Geo-Environmental Application of Penetration Testing”, Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Copies of ASTM Standards are available through www.astm.org

Table 1: Cone Penetration Testing Summary

CPT Sounding Identification	Date	Termination Depth (ft)	Depth of Soil Samples (ft)	Depth of Groundwater Samples (ft)	Depth of Pore Pressure Dissipation Tests (ft)
CPT-3	12/16/2025	100.07	-	-	100.07
CPT-4	12/16/2025	90.22	-	-	90.22
SCPT-5	12/15/2025	100	-	-	-
CPT-6	12/15/2025	52.56	-	-	20.21, 52.56
SCPT-7	12/15/2025	100	-	-	-

APPENDIX A:

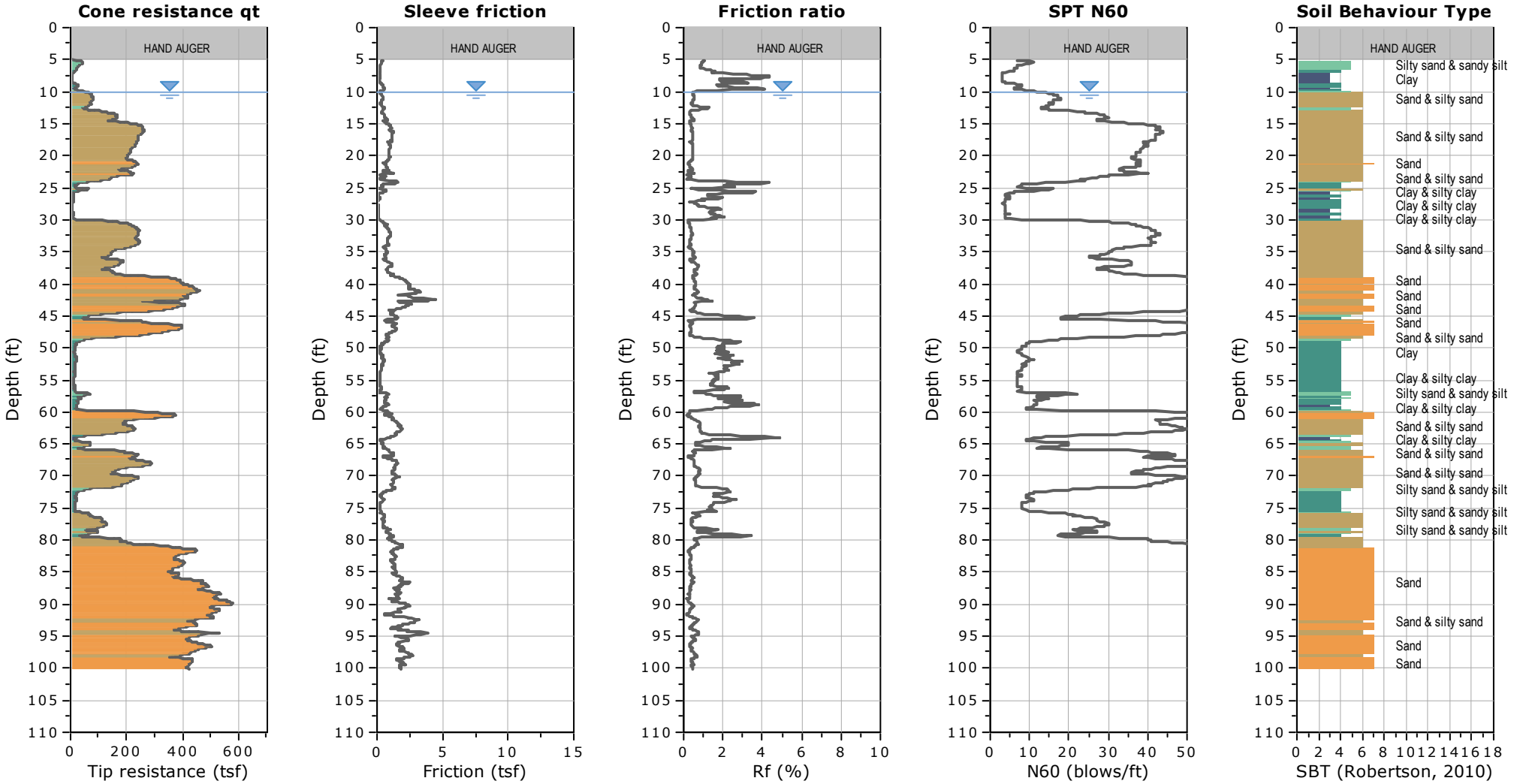
CPT PLOTS

GREGG DRILLING, LLC
https://www.greggdrilling.com/

CPT: CPT-3

CLIENT: MOORE TWINING
SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227
Total depth: 100.07 ft, Date: 12/16/2025



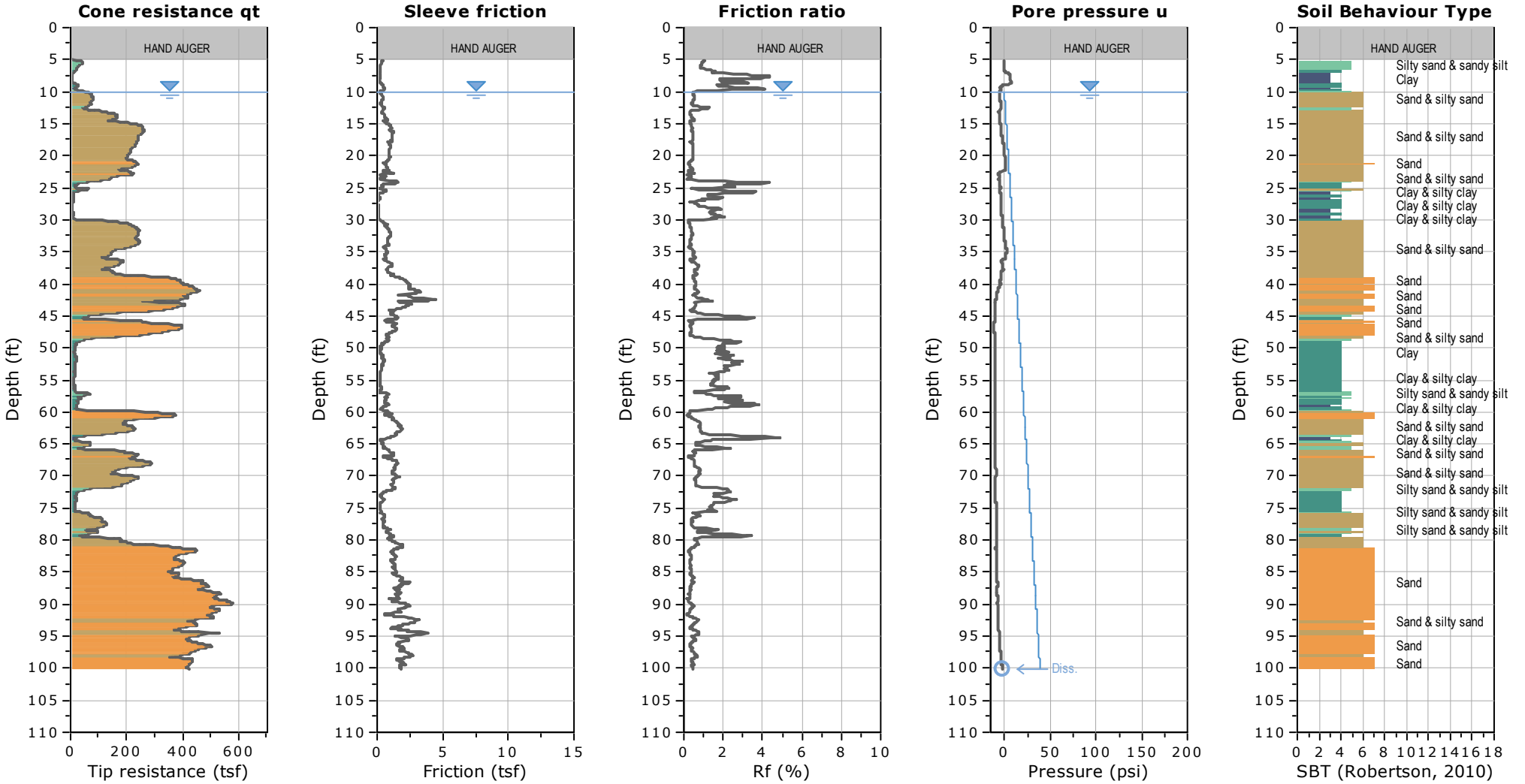
GREGG DRILLING, LLC
https://www.greggdrilling.com/

CPT: CPT-3

CLIENT: MOORE TWINING
SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 100.07 ft, Date: 12/16/2025



GREGG DRILLING, LLC
https://www.greggdrilling.com/

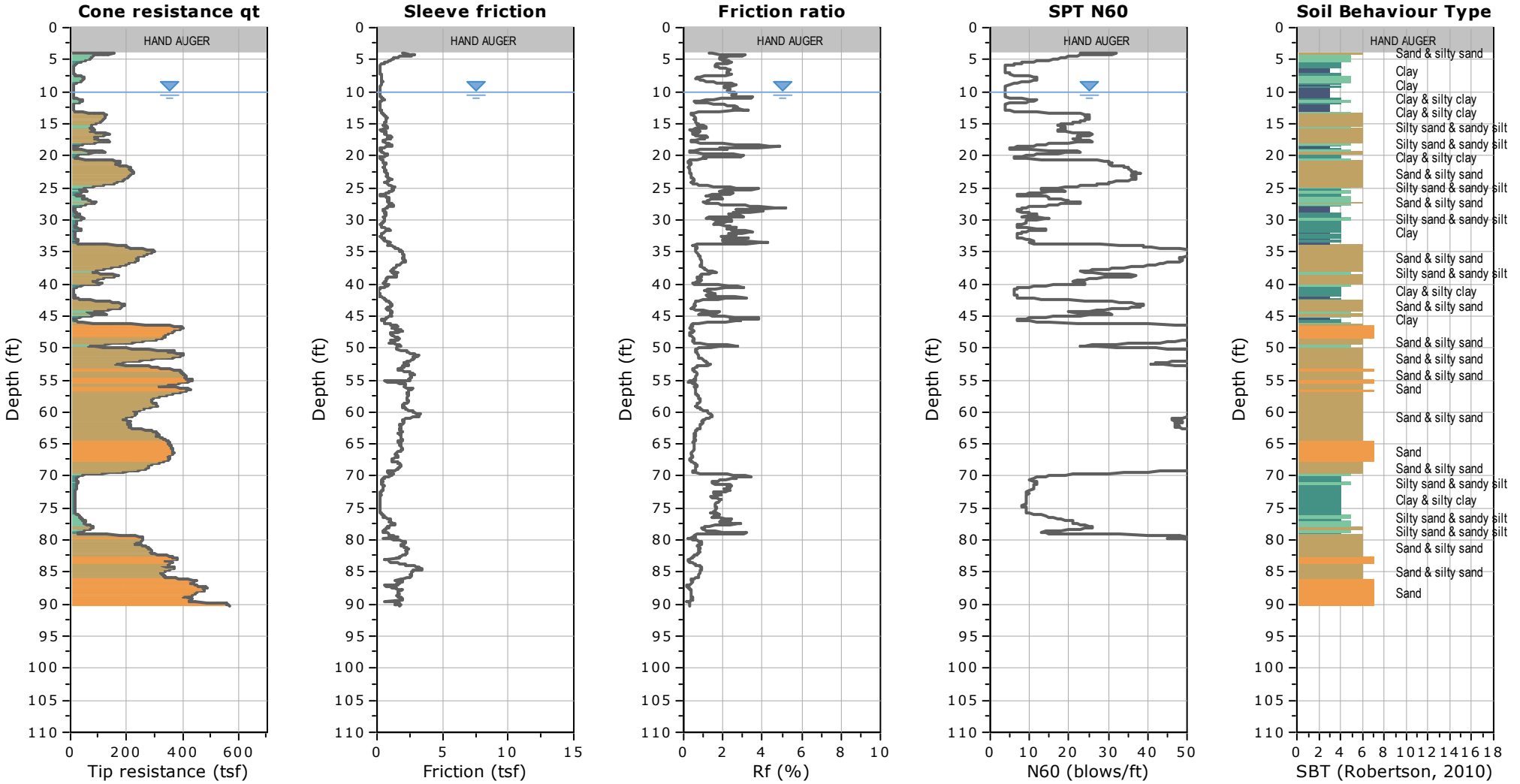
CPT: CPT-4

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 90.22 ft, Date: 12/16/2025



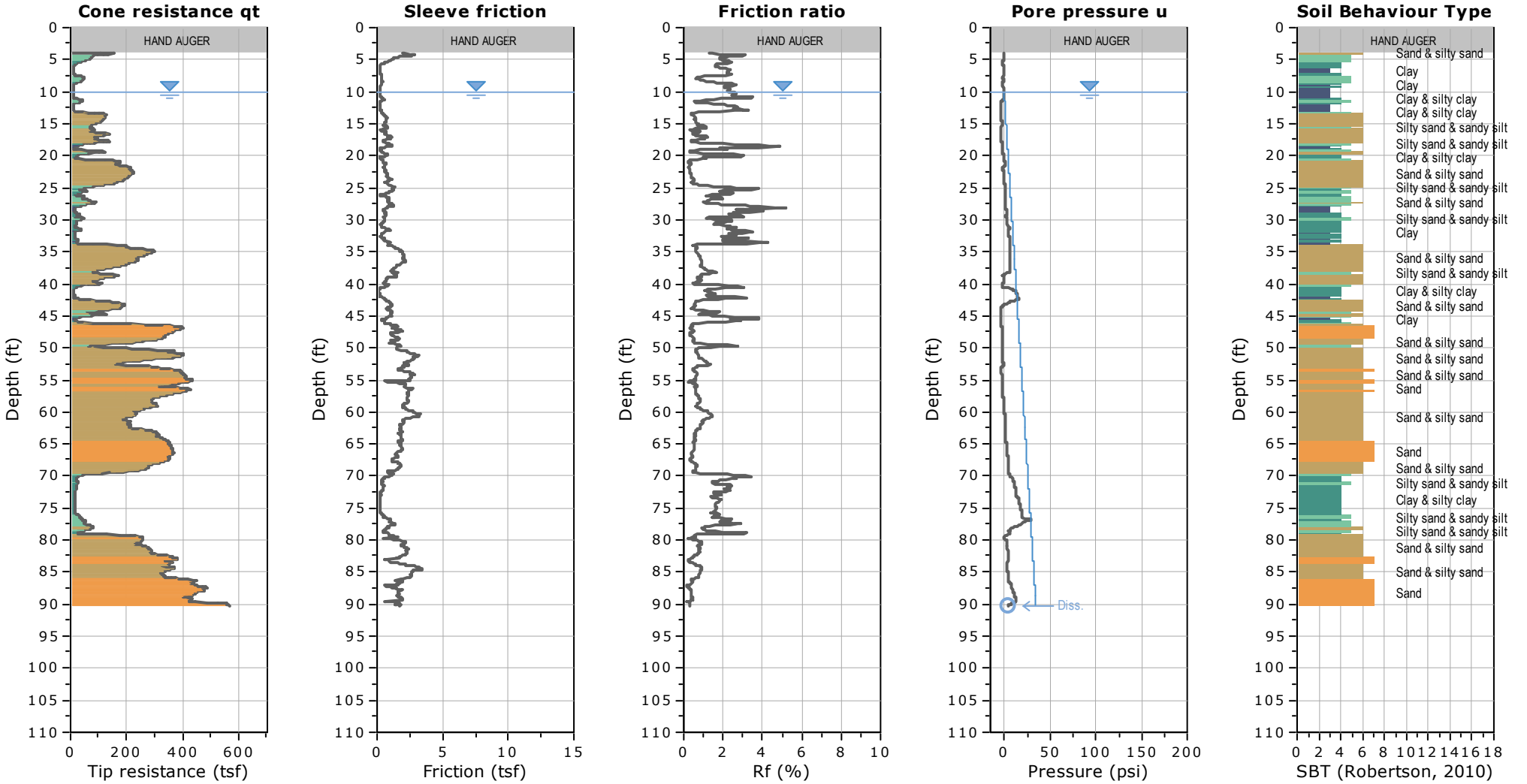
CPT: CPT-4

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 90.22 ft, Date: 12/16/2025



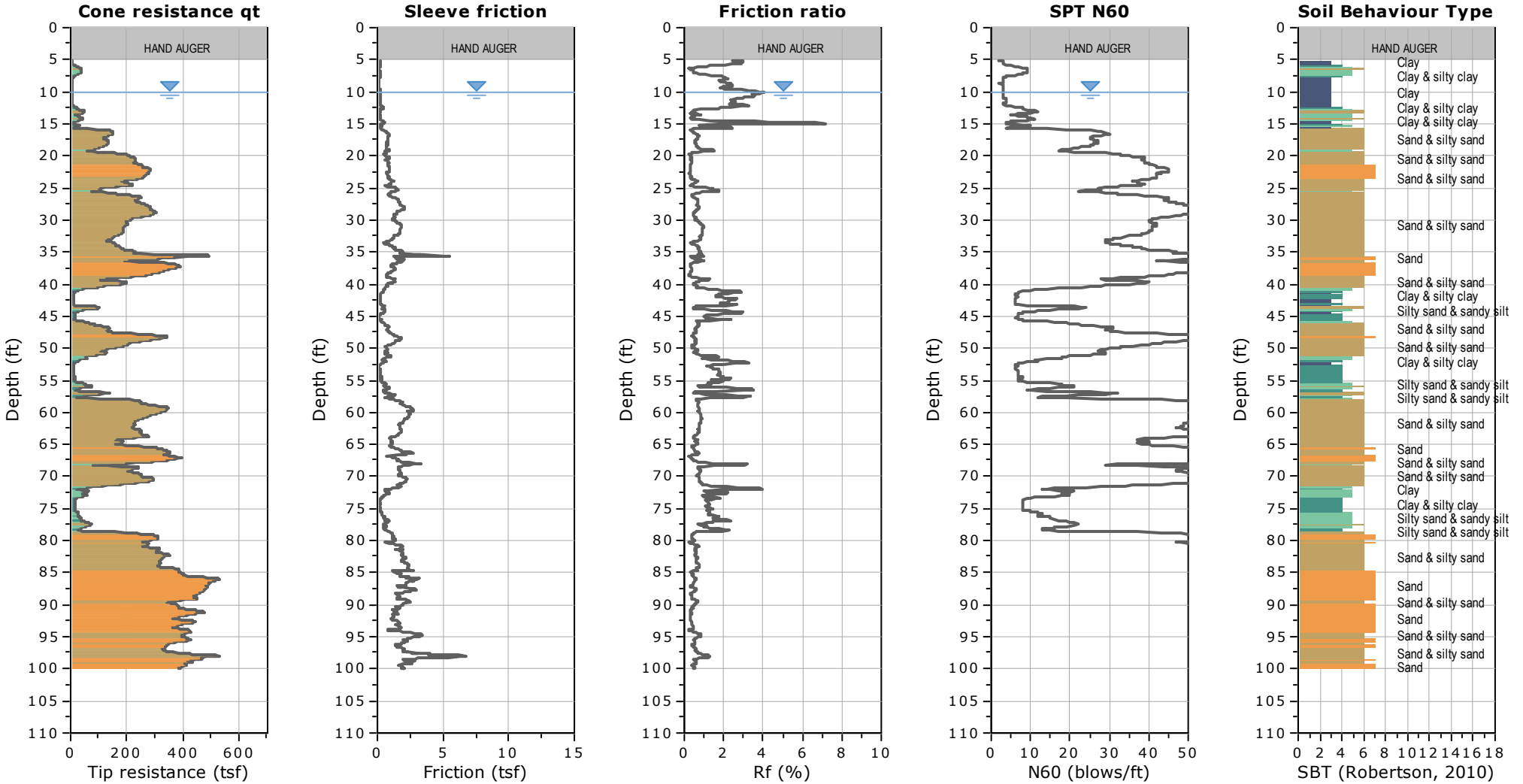
CPT: SCPT-5

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 100.00 ft, Date: 12/15/2025



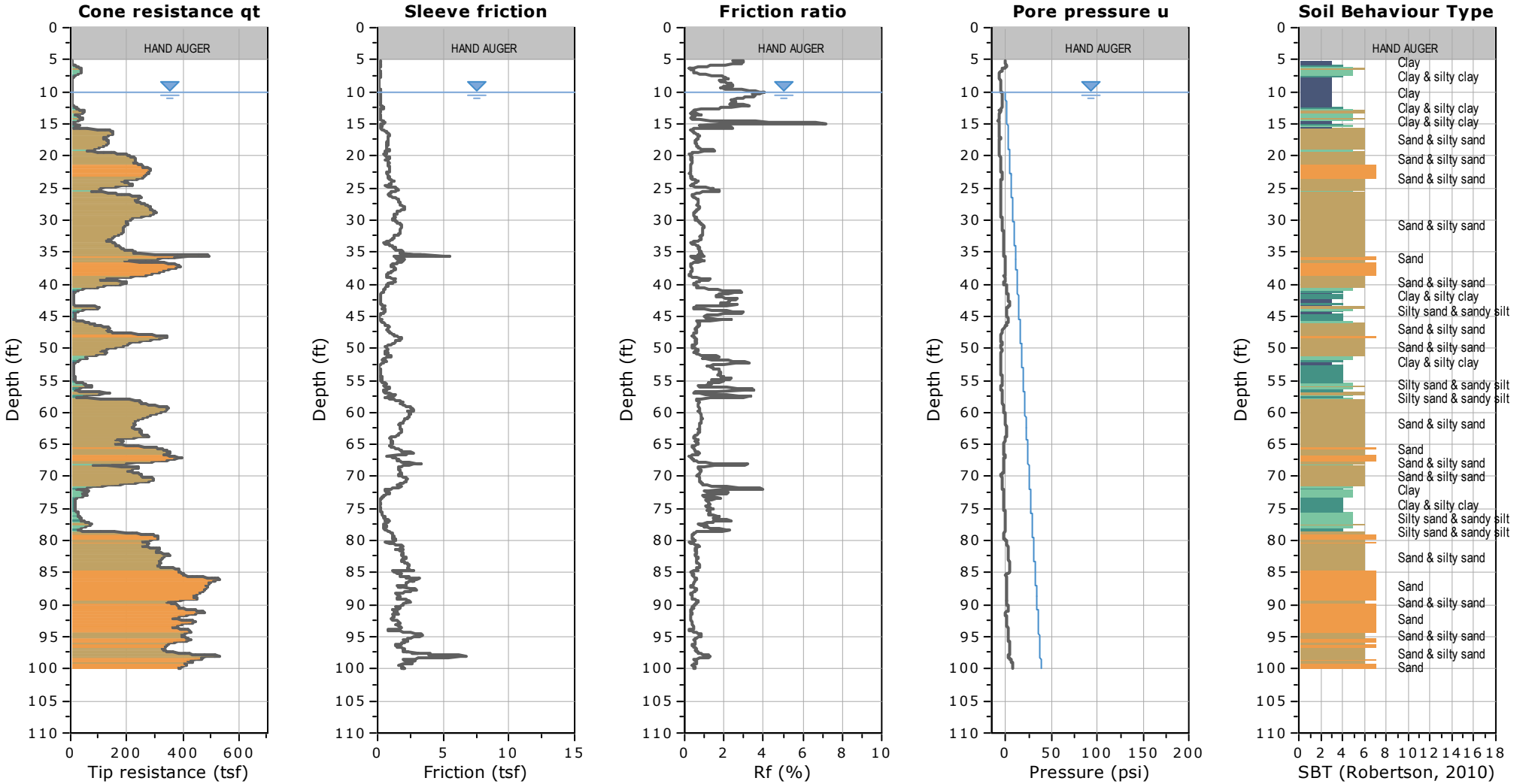
CPT: SCPT-5

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 100.00 ft, Date: 12/15/2025

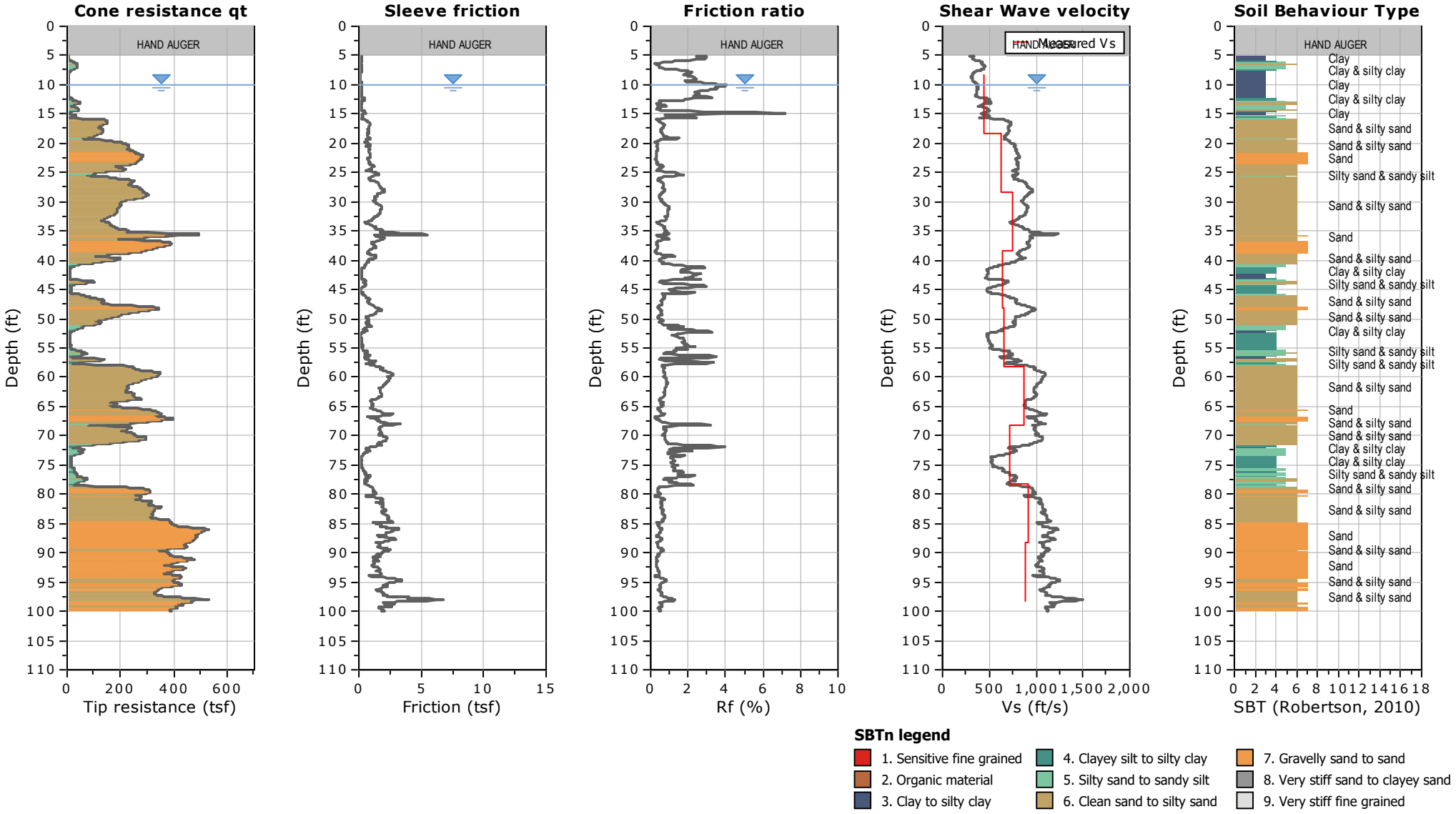


GREGG DRILLING, LLC
https://www.greggdrilling.com/

CPT: SCPT-5

CLIENT: MOORE TWINING
SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227
Total depth: 100.00 ft, Date: 12/15/2025



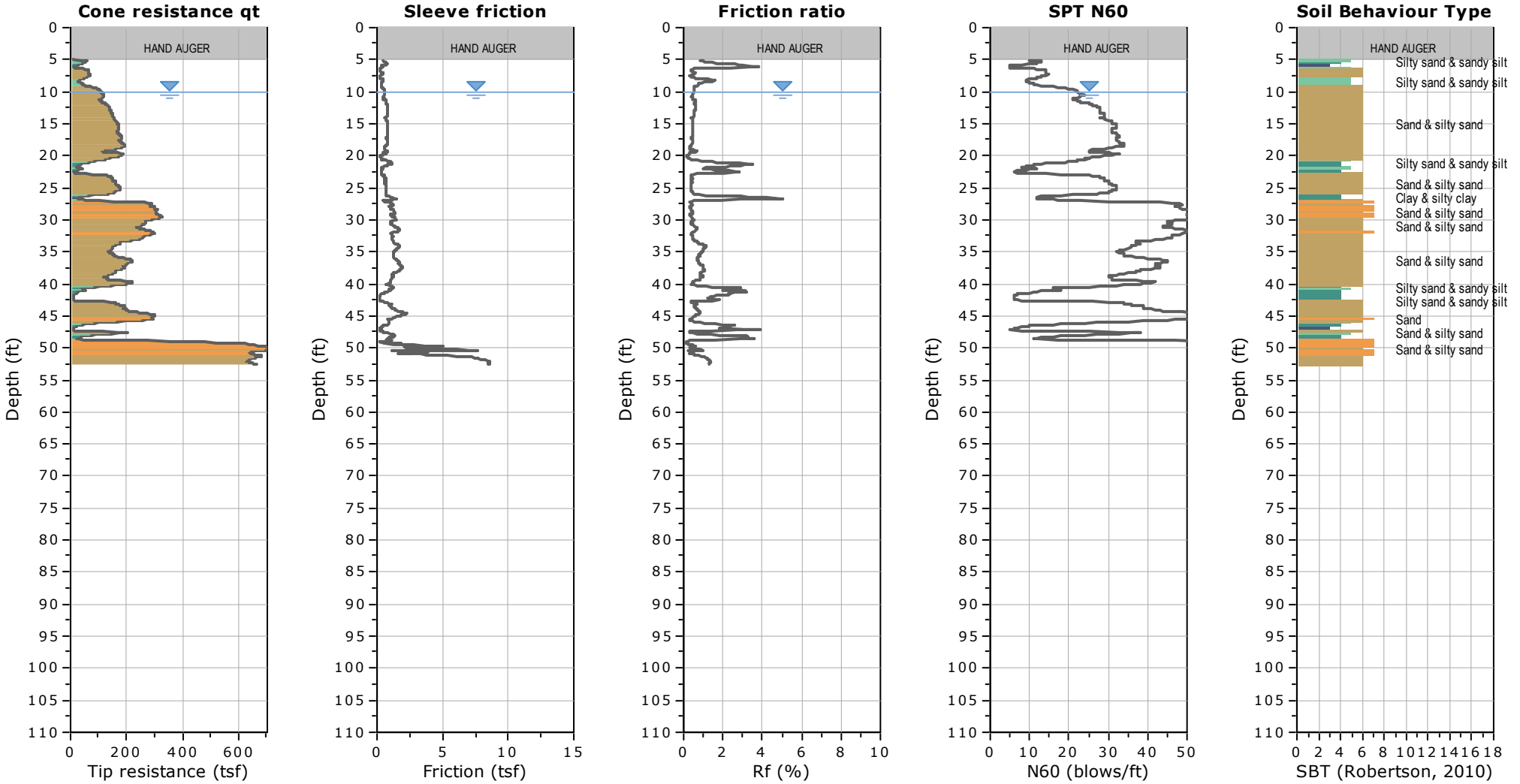
CPT: CPT-6

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 52.56 ft, Date: 12/15/2025



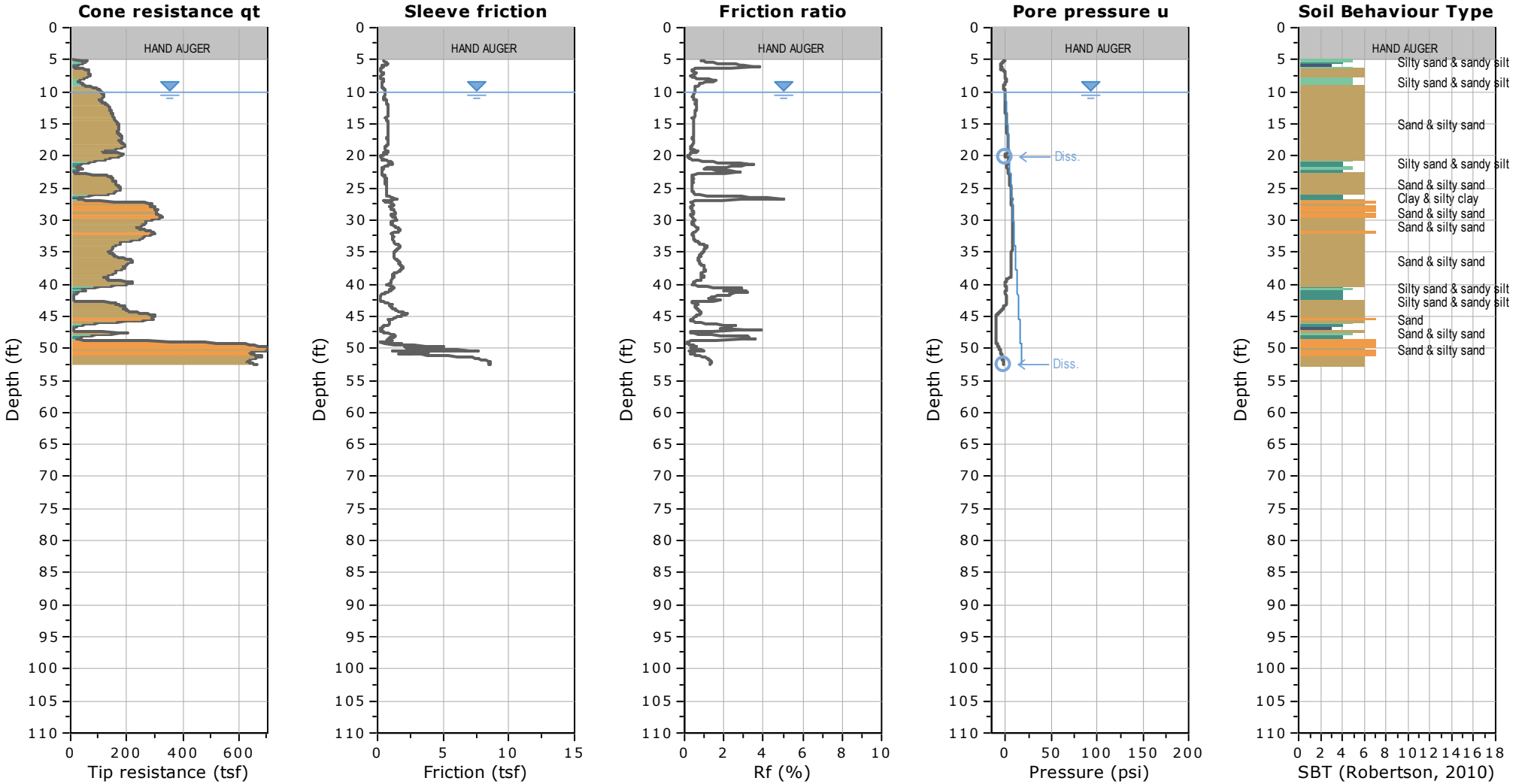
CPT: CPT-6

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 52.56 ft, Date: 12/15/2025



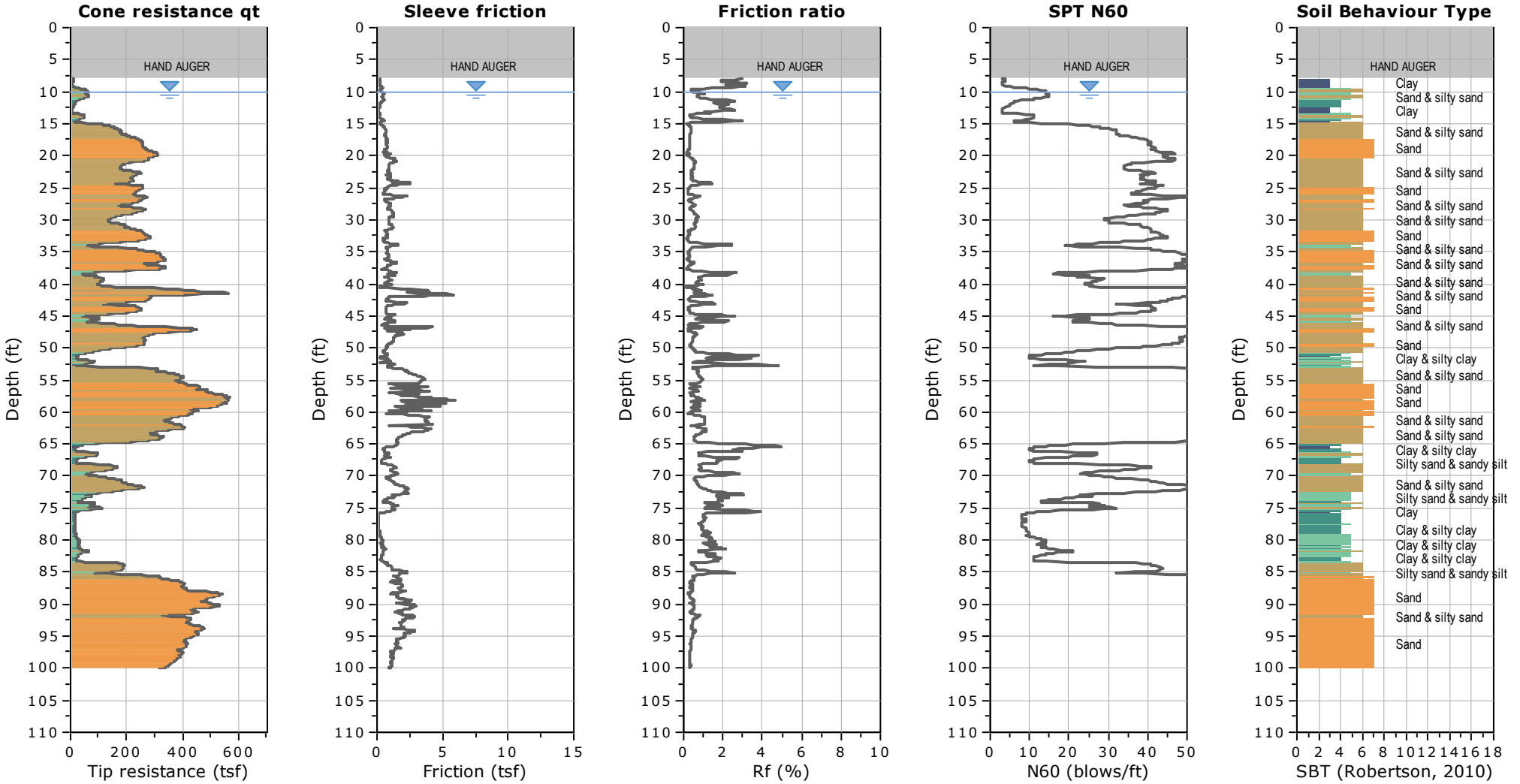
CPT: SCPT-7

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 100.00 ft, Date: 12/15/2025



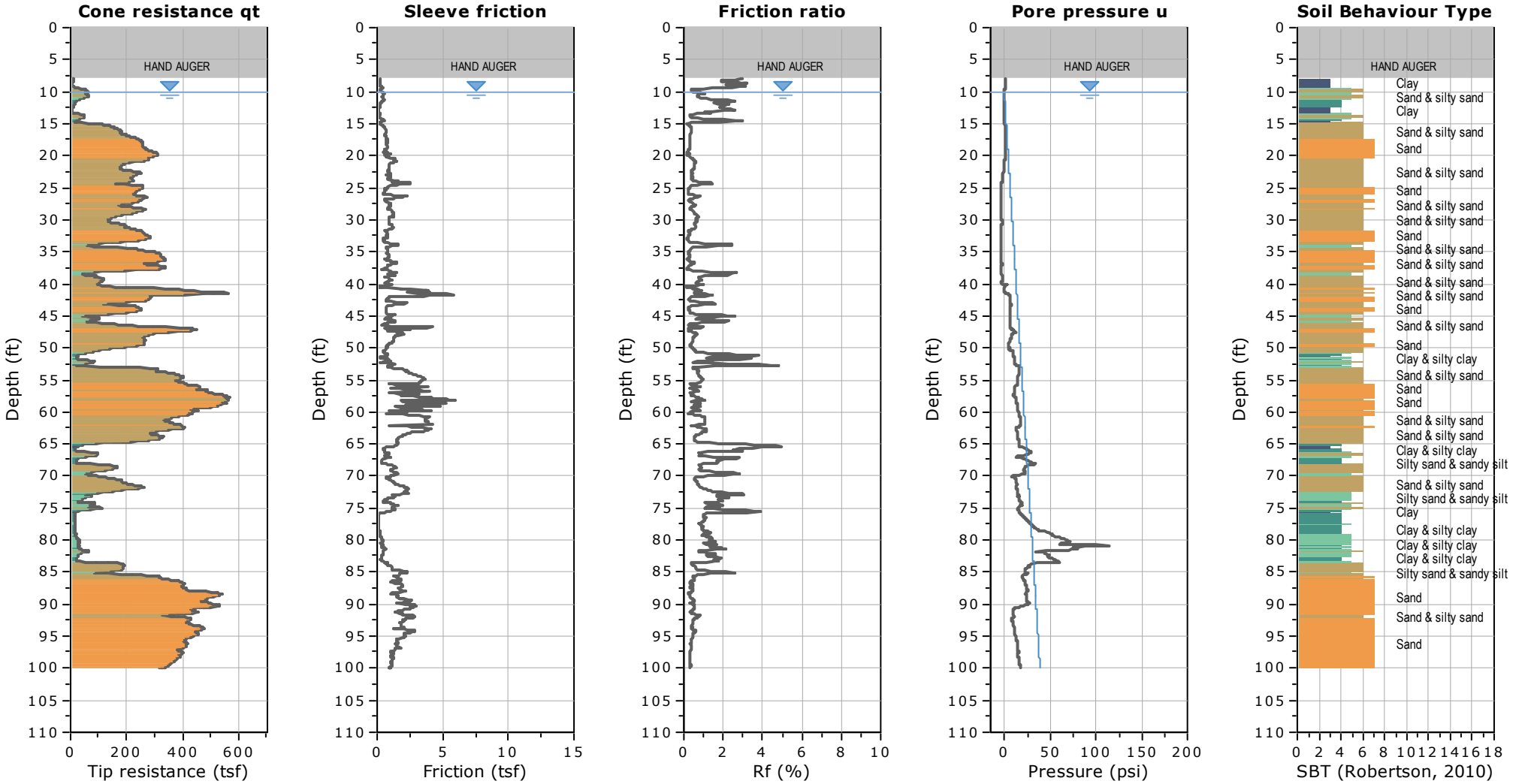
CPT: SCPT-7

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 100.00 ft, Date: 12/15/2025



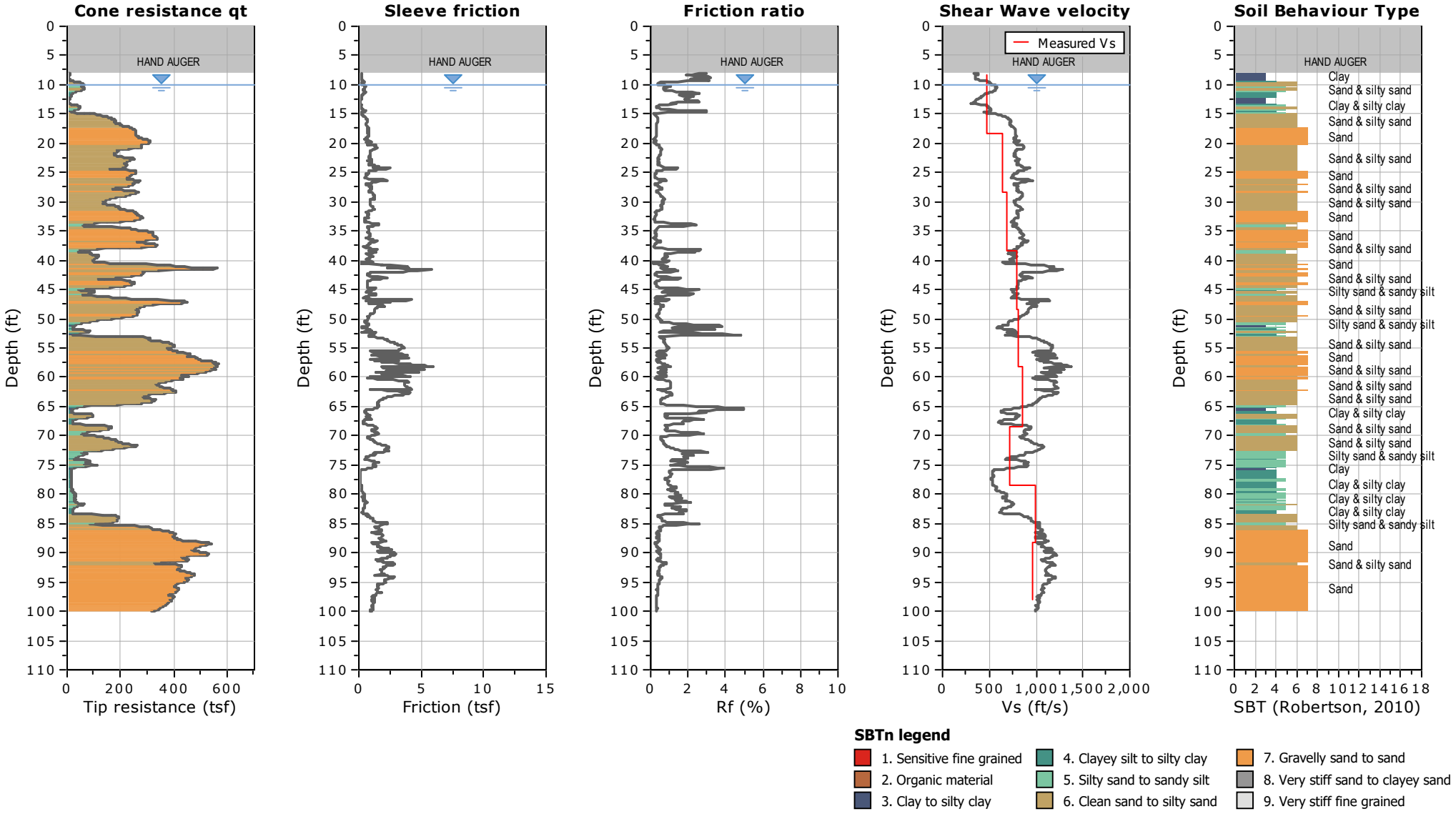
CPT: SCPT-7

CLIENT: MOORE TWINING

SITE: PORT OF HUENEME, PORT HUENEME, CA

FIELD REP: SHAUN R.
Cone ID: 231227

Total depth: 100.00 ft, Date: 12/15/2025



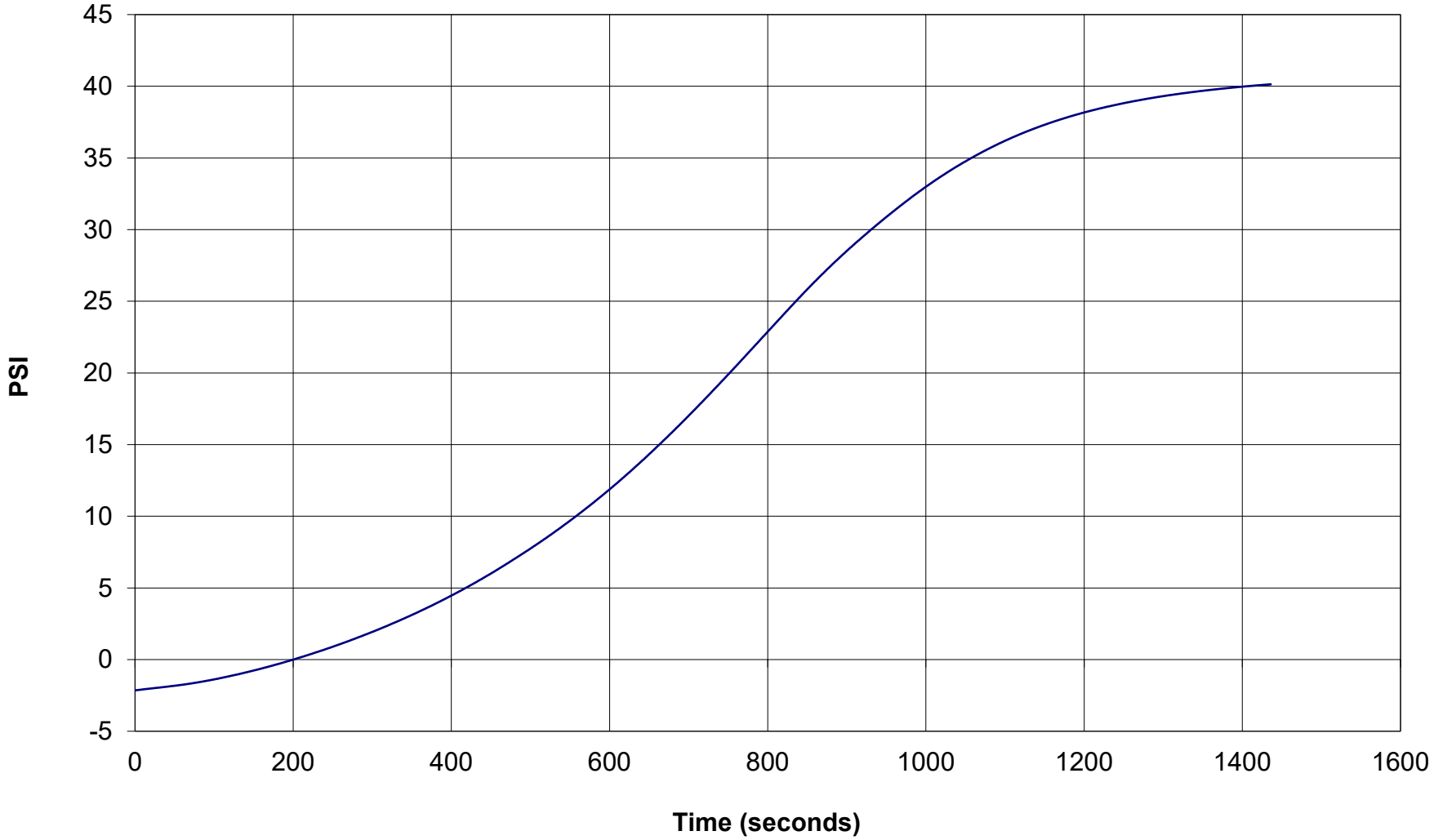
APPENDIX B:

PORE PRESSURE DISSIPATION TEST PLOTS

GREGG DRILLING, LLC

Pore Pressure Dissipation Test

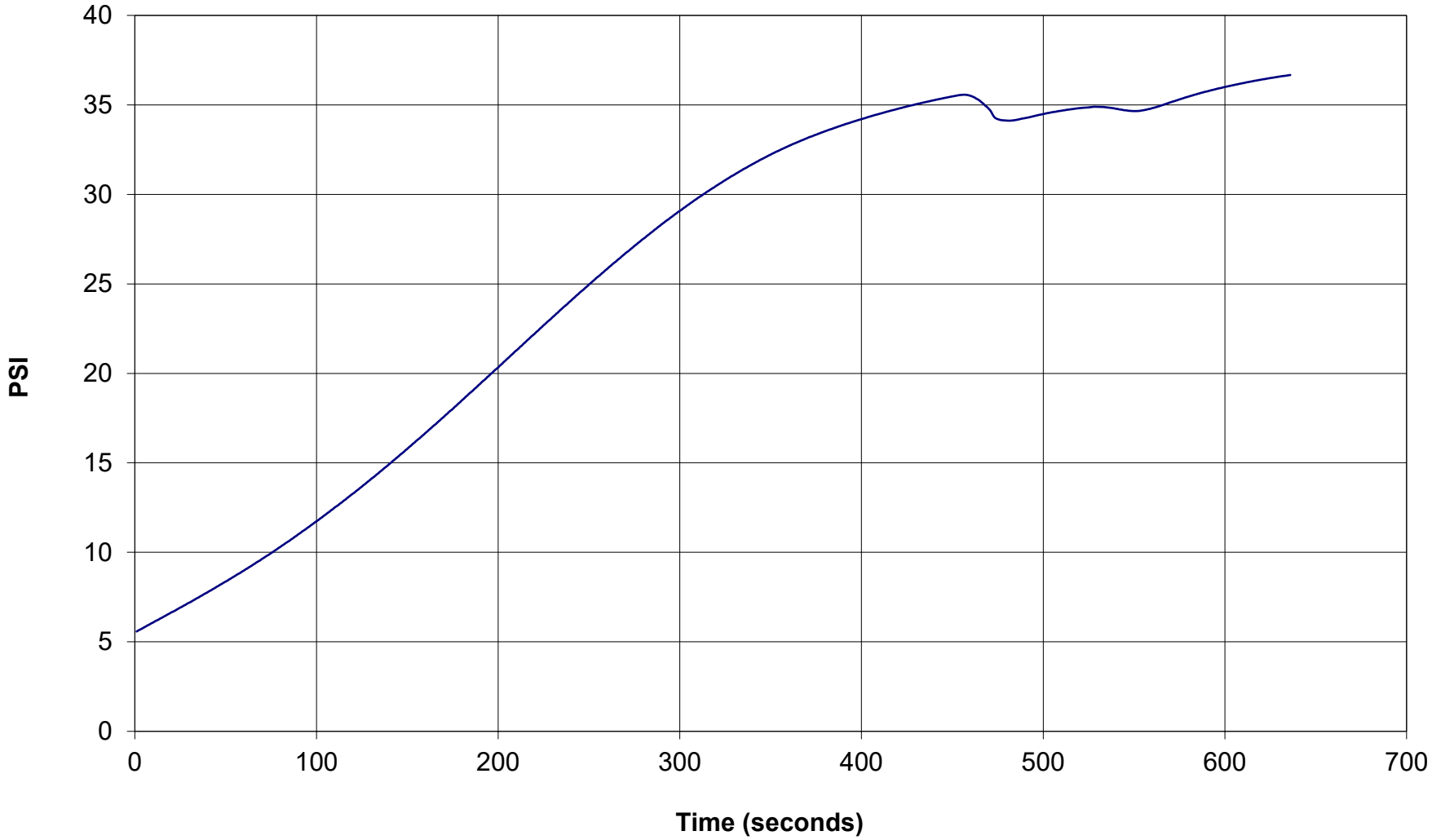
Sounding: CPT-3
Depth (ft): 100.07
Site: Port of Hueneme



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

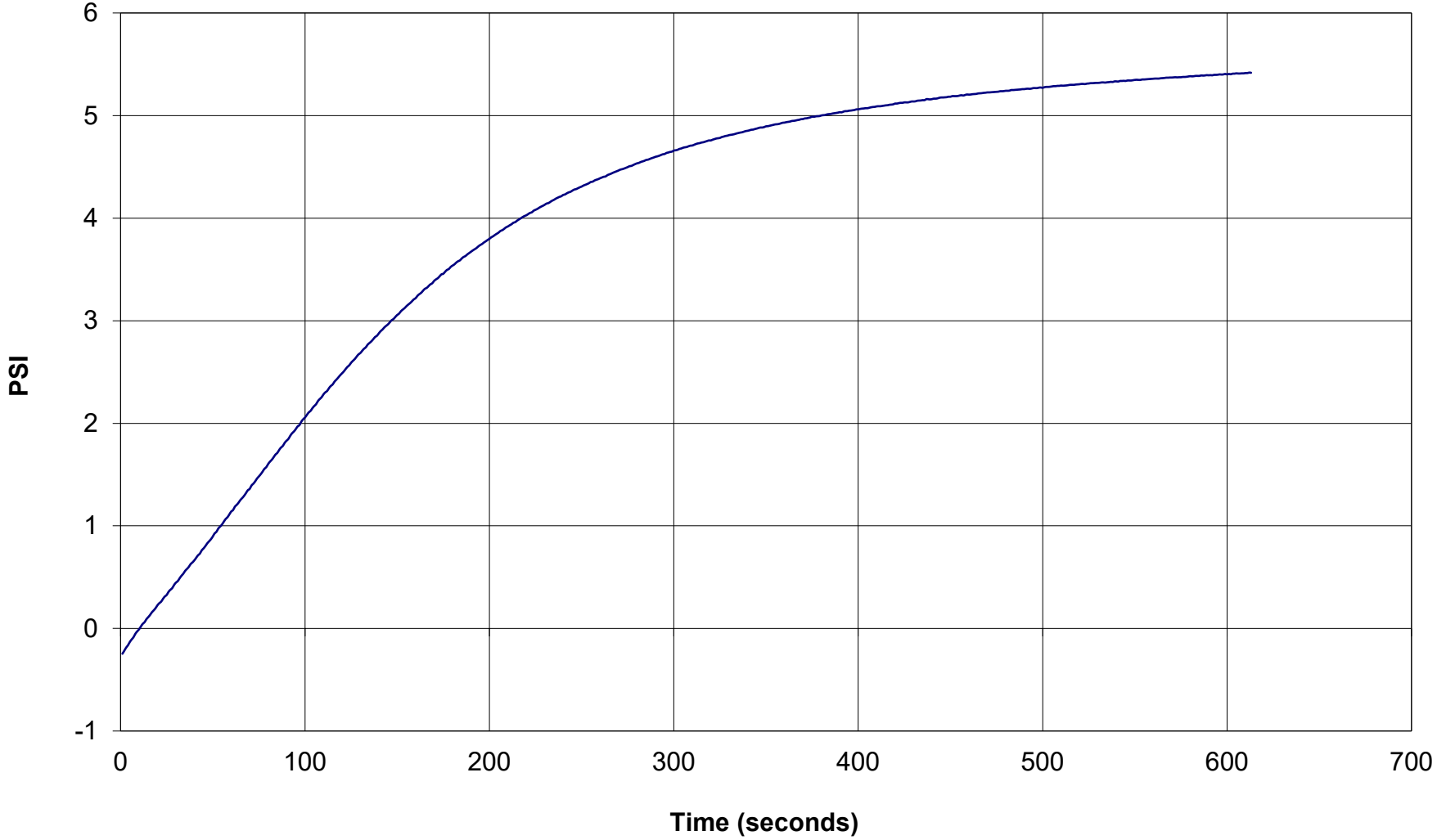
Sounding: CPT-4
Depth (ft): 90.22
Site: Port of Hueneme



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

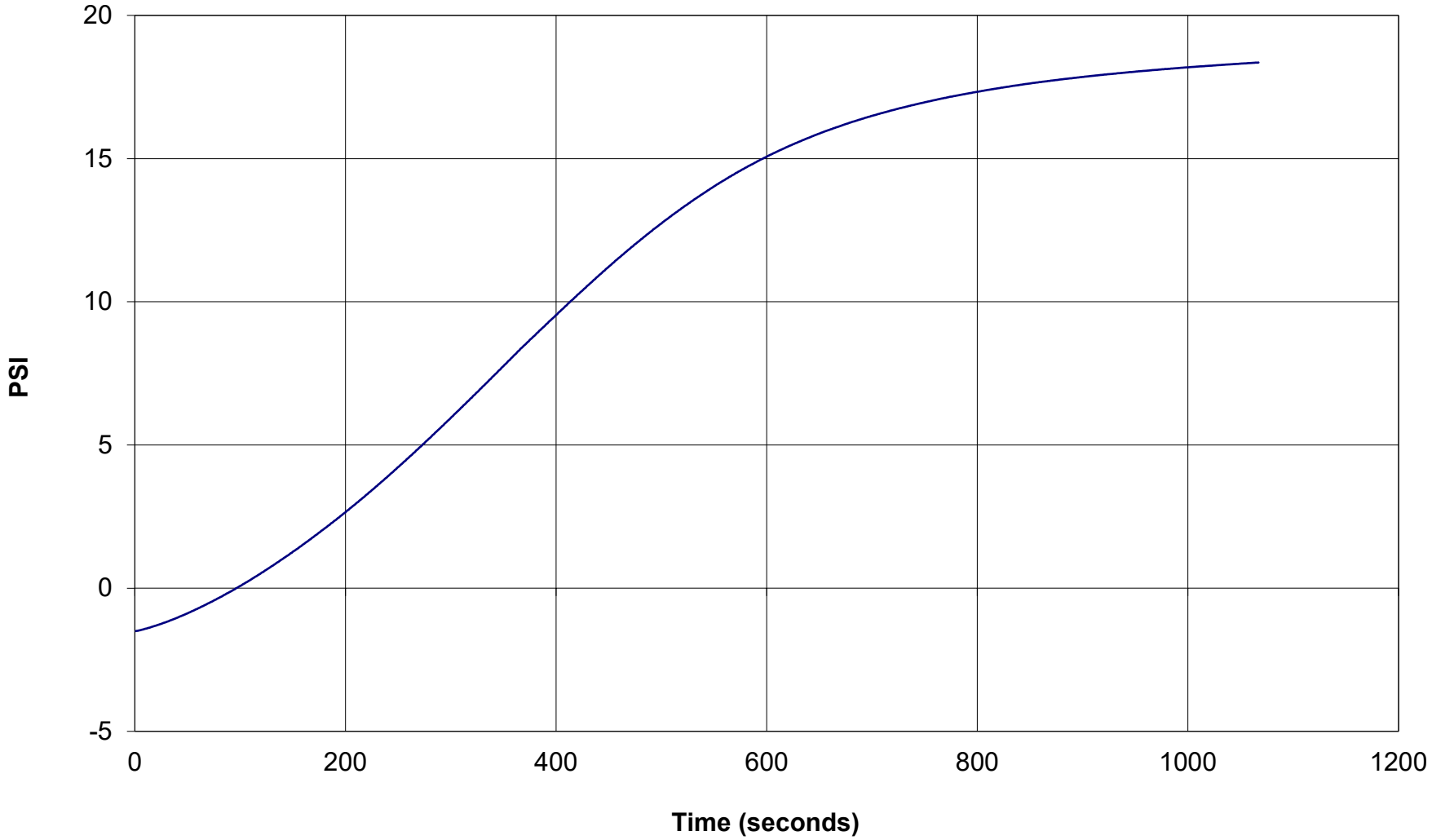
Sounding: CPT-6
Depth (ft): 20.21
Site: Port of Hueneme



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

Sounding: CPT-6
Depth (ft): 52.56
Site: Port of Hueneme

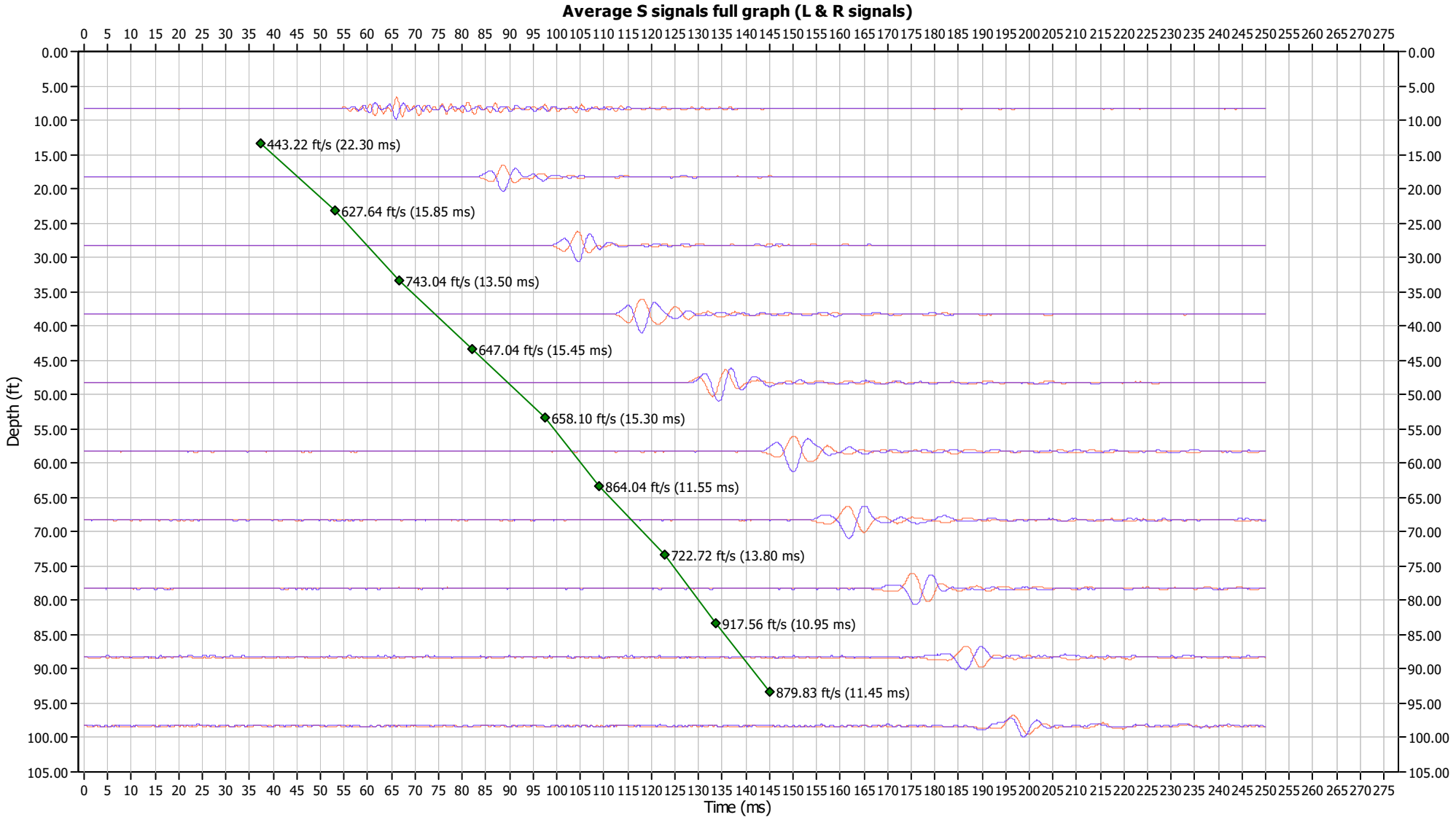


APPENDIX C:

SEISMIC

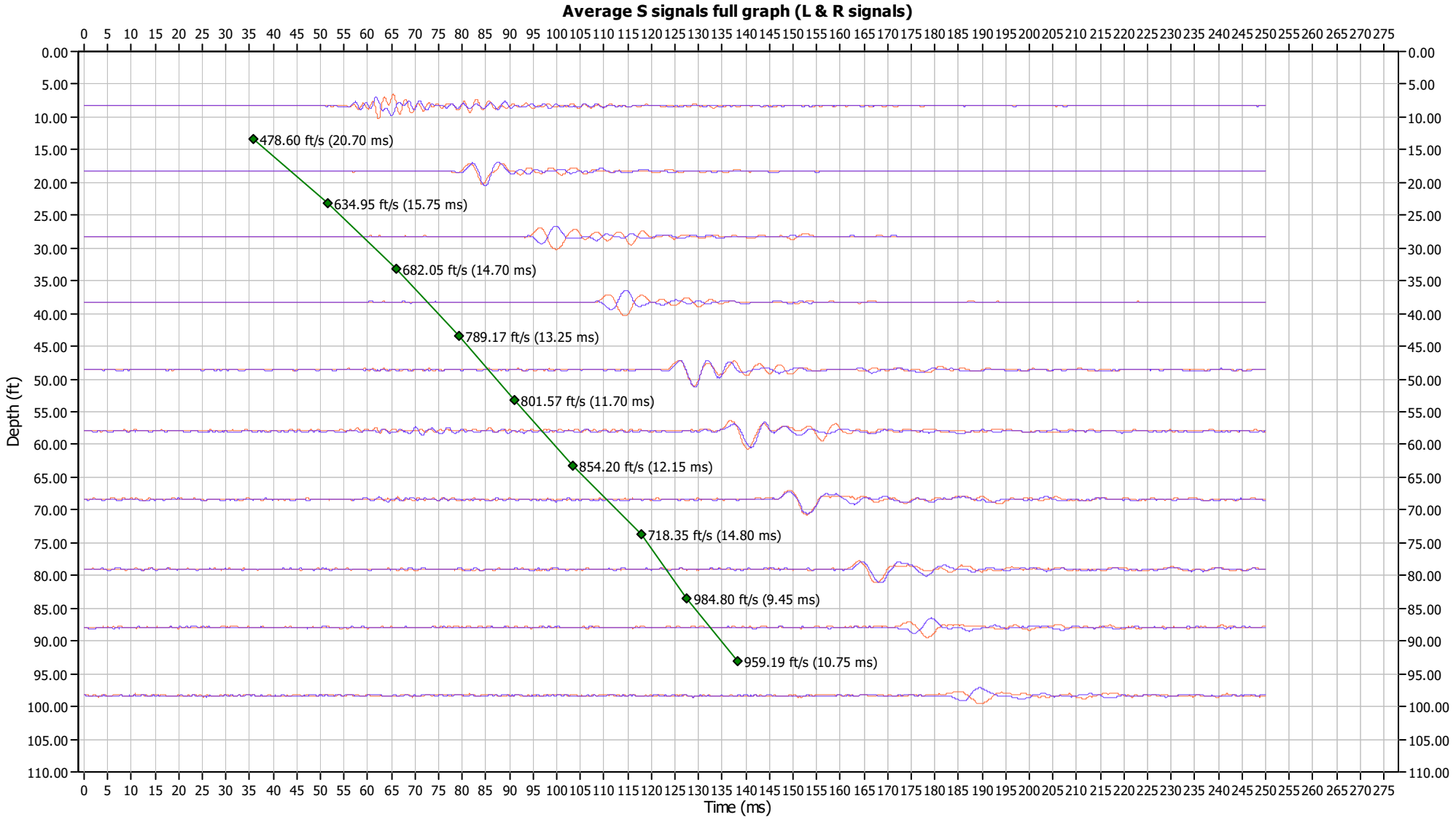
PLOTS & TABLES

Project: D1255059 - MOORE TWINING **Borehole ID: SCPT-5**



Project: D1255059 - MOORE TWINING

Borehole ID: SCPT-7



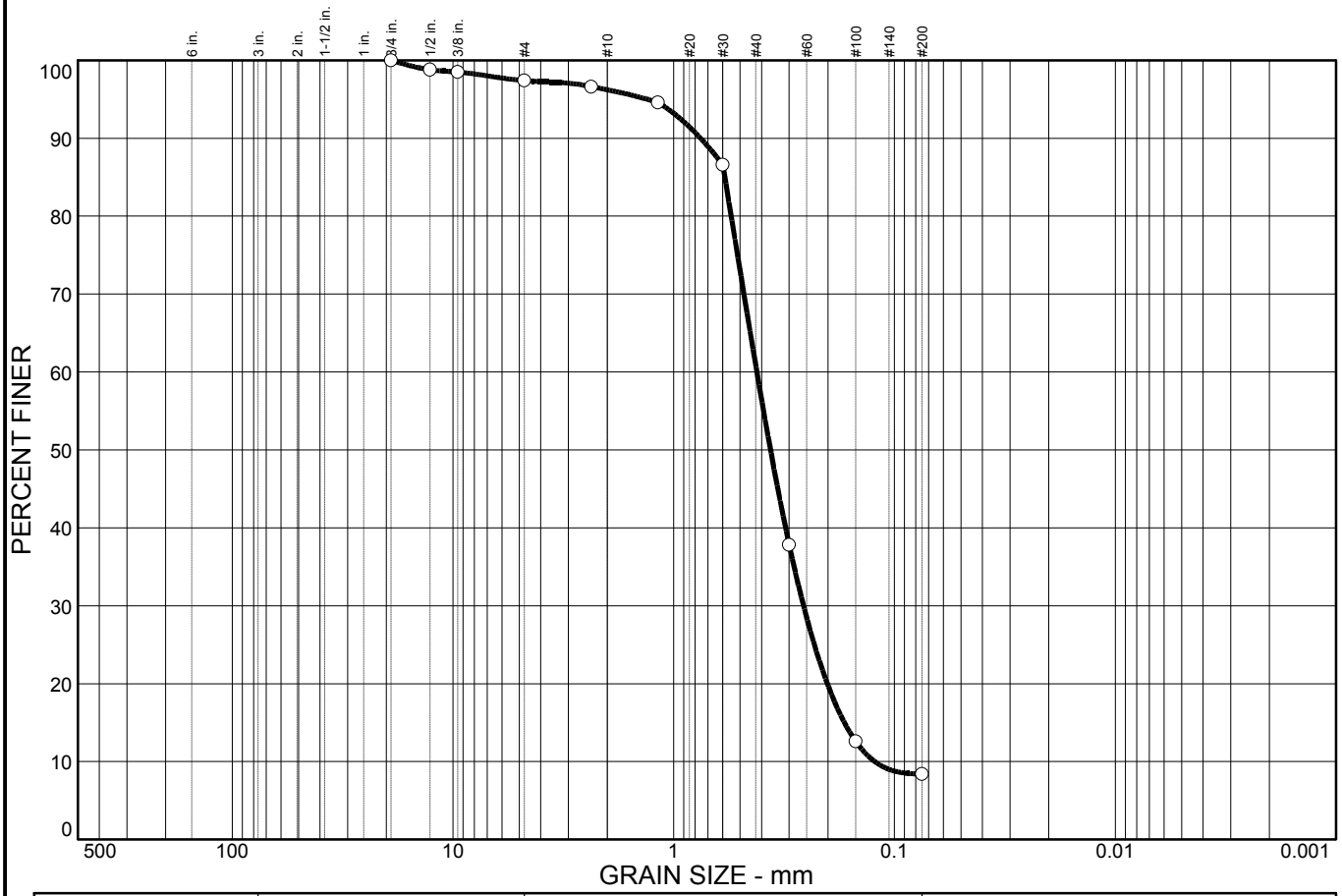
APPENDIX C

RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	To Determine:
Moisture Content (ASTM D2216)	Moisture contents representative of field conditions at the time the sample was taken.
Dry Density (ASTM D2216)	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Grain-Size Distribution (ASTM D422)	Size and distribution of soil particles, i.e., sand, gravel and fines (silt and clay).
Atterberg Limits (ASTM D4318)	Determines the moisture contents where the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state (plastic limit)
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.
Sulfate Content (ASTM D4327)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Expansion Index (ASTM D4829)	Swell potential of soil with increases in moisture content.
Moisture-Density Relationship (ASTM D698)	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.
Resistance-Value (ASTM D2844)	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Chloride Content (Cal Test 422)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Sulfate Content (Cal Test 417)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Minimum Resistivity (ASTM G187)	The minimum resistivity in ohm-centimeters of a soil. Used to determine the corrosion potential for buried metal objects
pH (Cal Test 643)	The acidity or alkalinity of subgrade material.

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	2.6	1.2	35.5	52.3	8.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.8		
3/8 in.	98.5		
#4	97.4		
#8	96.6		
#16	94.6		
#30	86.6		
#50	37.8		
#100	12.6		
#200	8.4		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.587 D_{60}= 0.420 D_{50}= 0.365
D_{30}= 0.258 D_{15}= 0.169 D_{10}= 0.123
C_u= 3.42 C_c= 1.29

Classification

USCS= SP-SM AASHTO=

Remarks

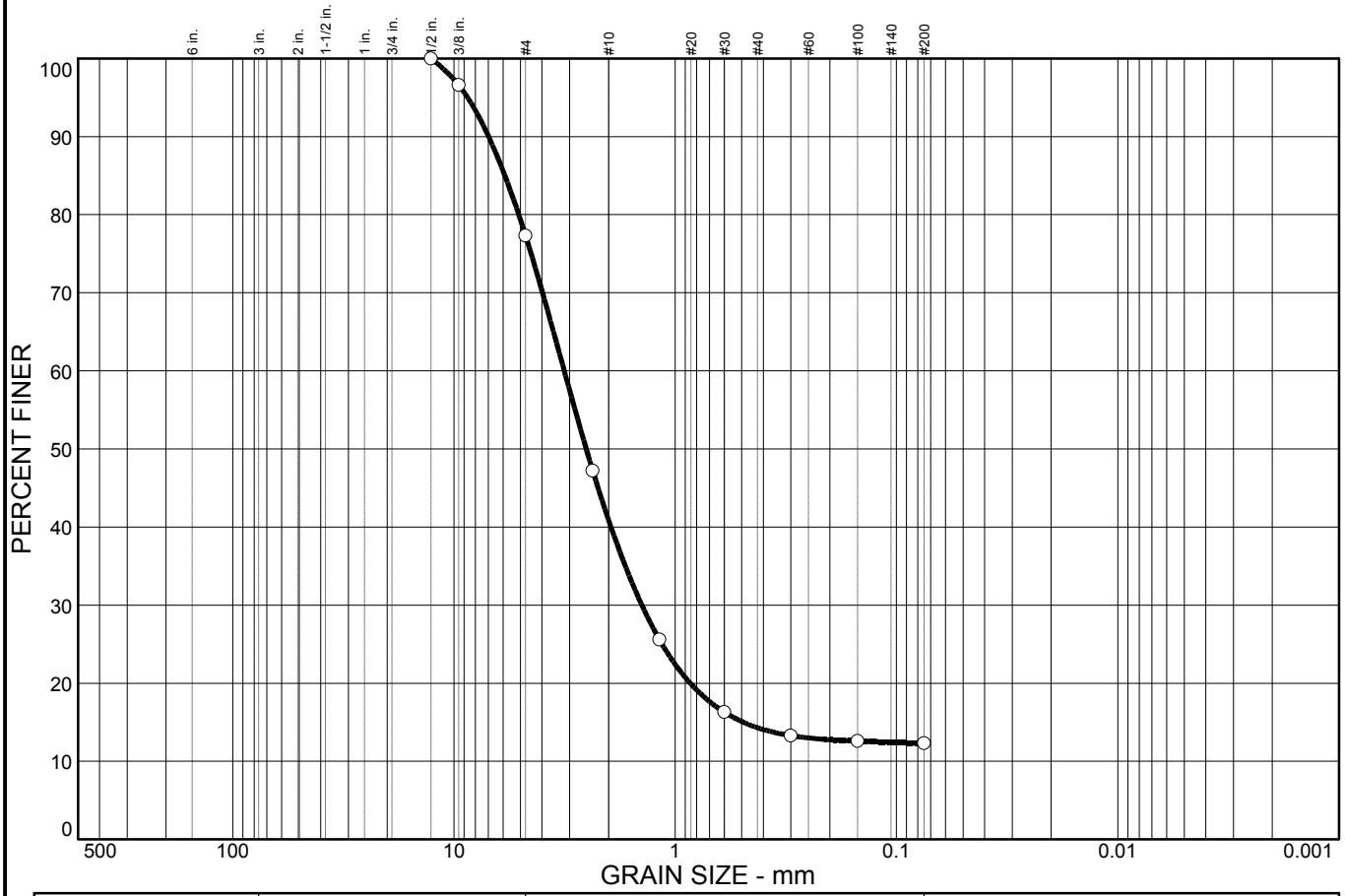
Sample No.: B-1 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 15-16.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	22.7	36.4	26.6	2.0	12.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	96.6		
#4	77.3		
#8	47.2		
#16	25.6		
#30	16.3		
#50	13.3		
#100	12.6		
#200	12.3		

* (no specification provided)

Material Description

Clayey sand with gravel

Atterberg Limits

PL= 15 LL= 25 PI= 10

Coefficients

D_{85}= 5.89 D_{60}= 3.18 D_{50}= 2.53
D_{30}= 1.42 D_{15}= 0.492 D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

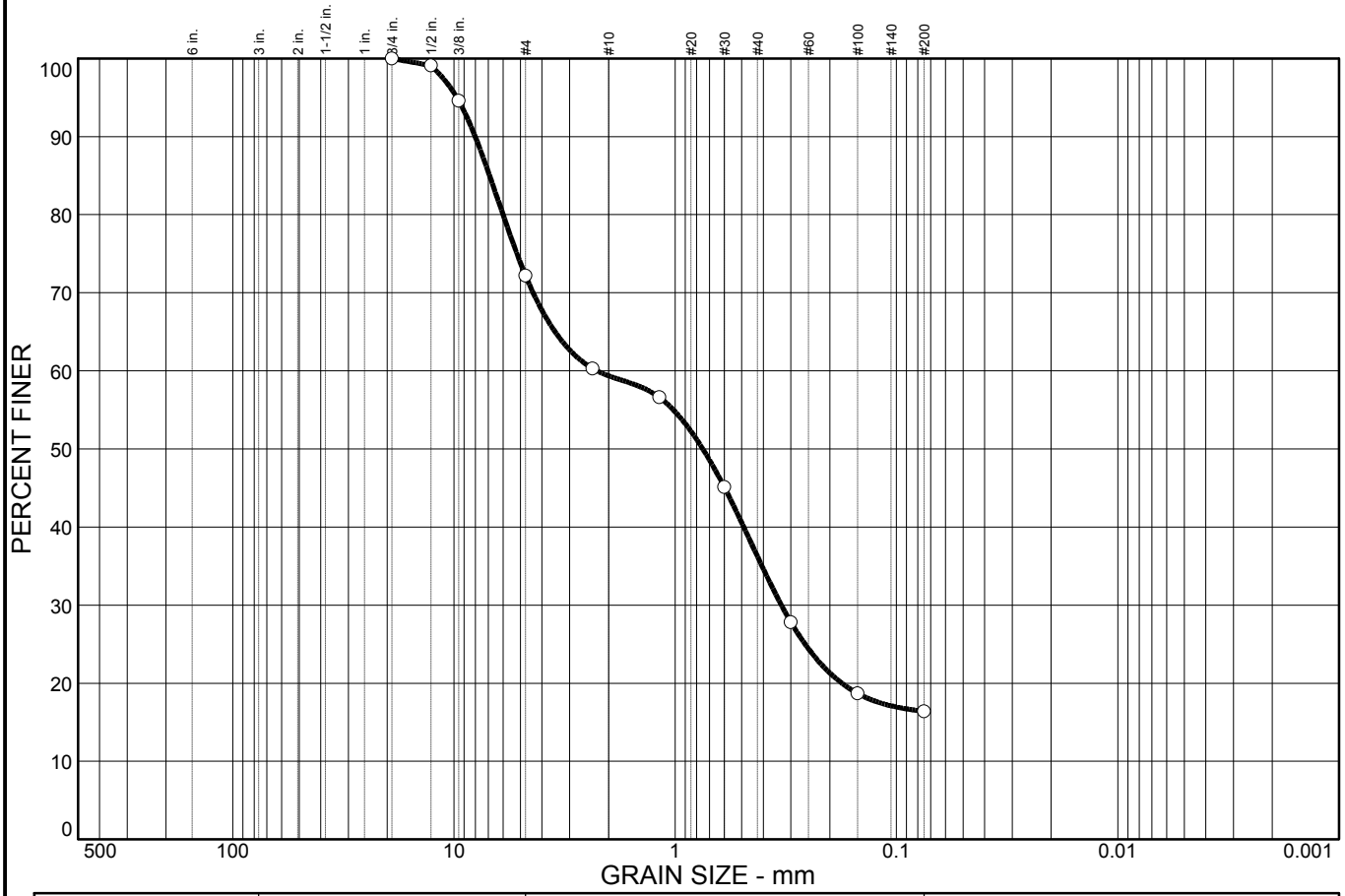
Sample No.: B-1 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 25-26.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	27.8	12.8	23.1	19.9	16.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	99.1		
3/8 in.	94.6		
#4	72.2		
#8	60.3		
#16	56.6		
#30	45.1		
#50	27.8		
#100	18.7		
#200	16.4		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 6.91 D_{60}= 2.26 D_{50}= 0.751
D_{30}= 0.331 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

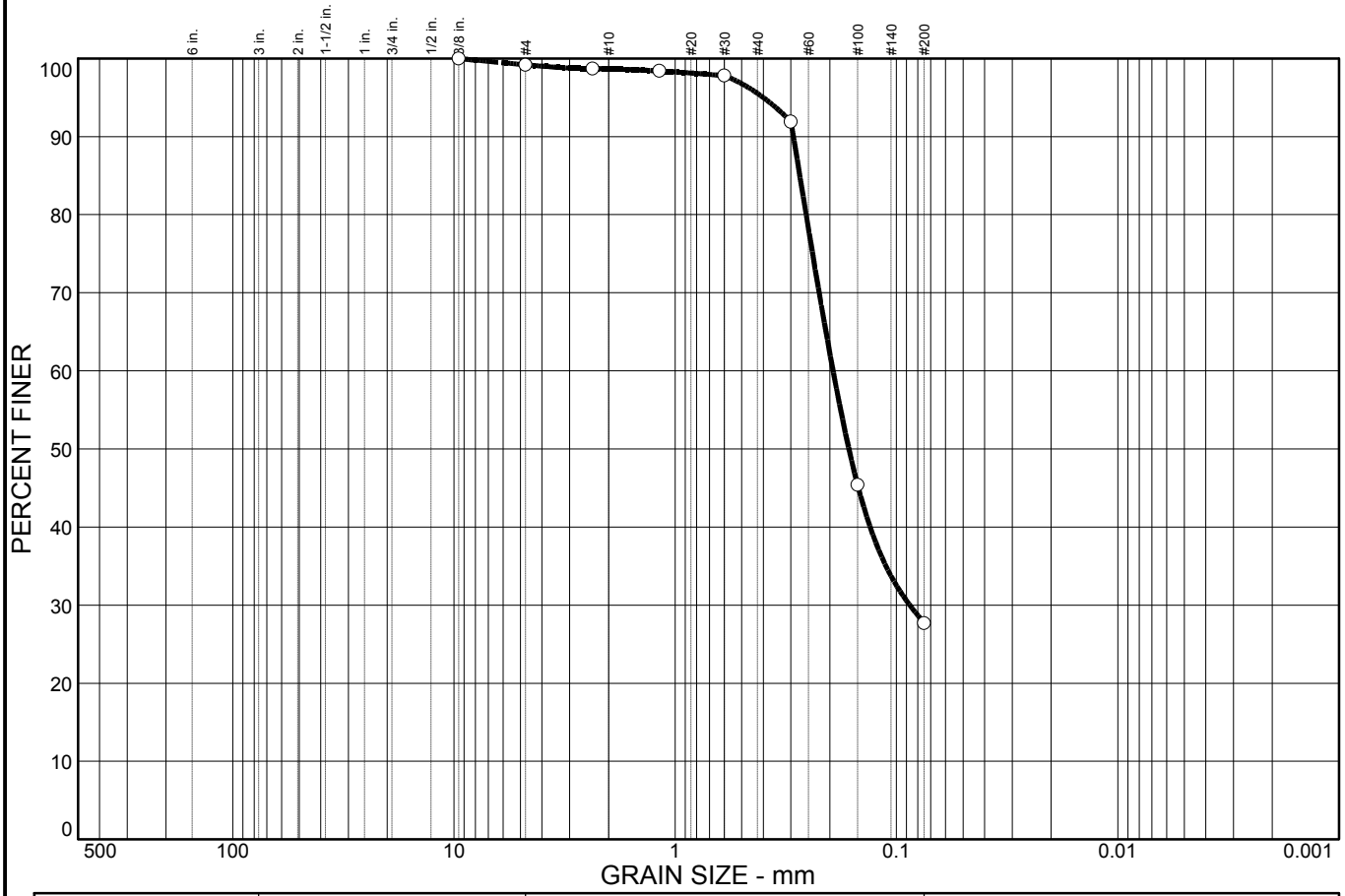
Sample No.: B-1 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 35-36.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.8	0.5	3.1	67.9	27.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.2		
#8	98.7		
#16	98.4		
#30	97.8		
#50	91.9		
#100	45.4		
#200	27.7		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.274 D_{60}= 0.194 D_{50}= 0.164
D_{30}= 0.0871 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

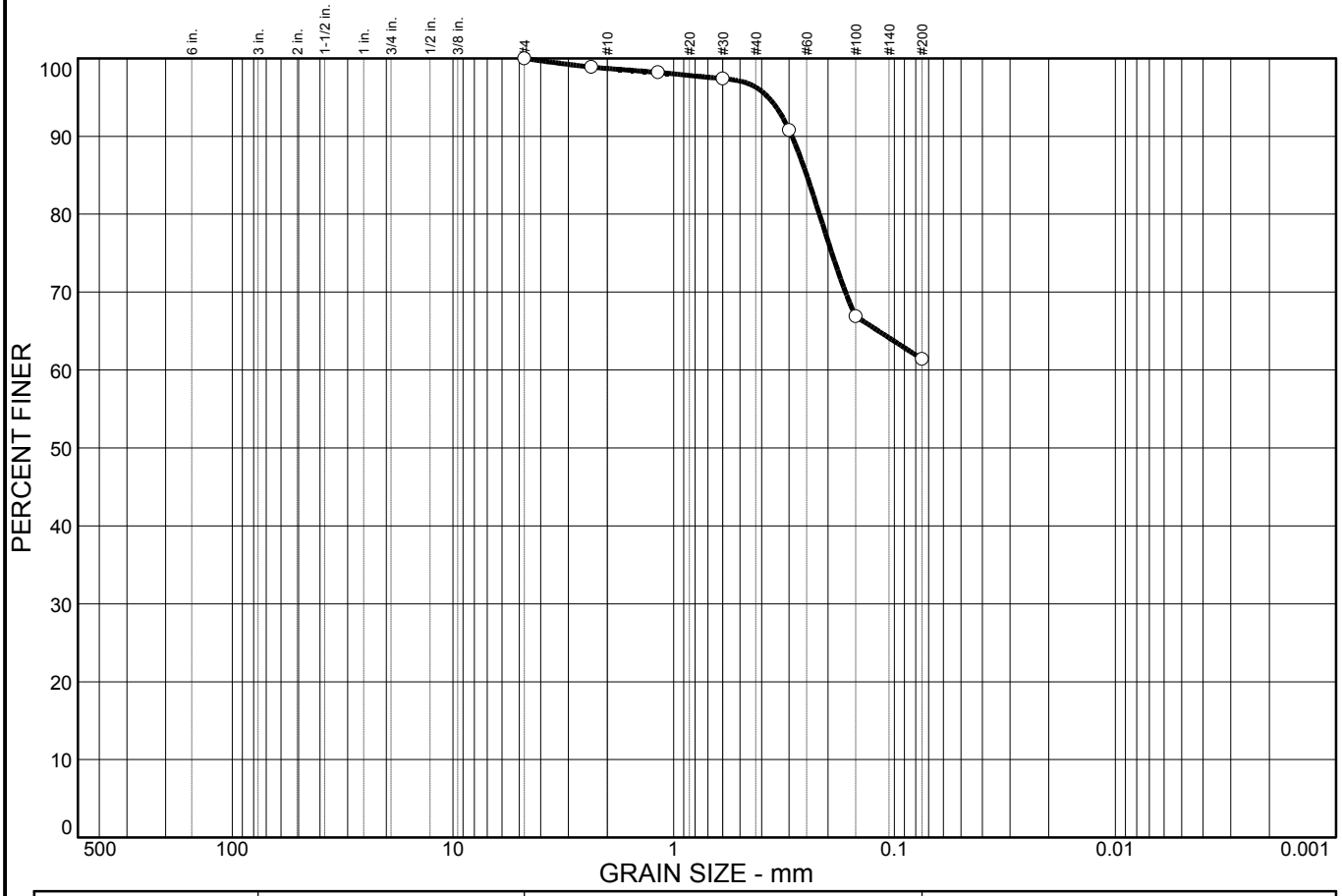
Sample No.: B-2 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 1-4'

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	1.3	2.4	34.9	61.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	98.9		
#16	98.2		
#30	97.4		
#50	90.8		
#100	66.9		
#200	61.4		

* (no specification provided)

Material Description

Sandy silty clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.250 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO=

Remarks

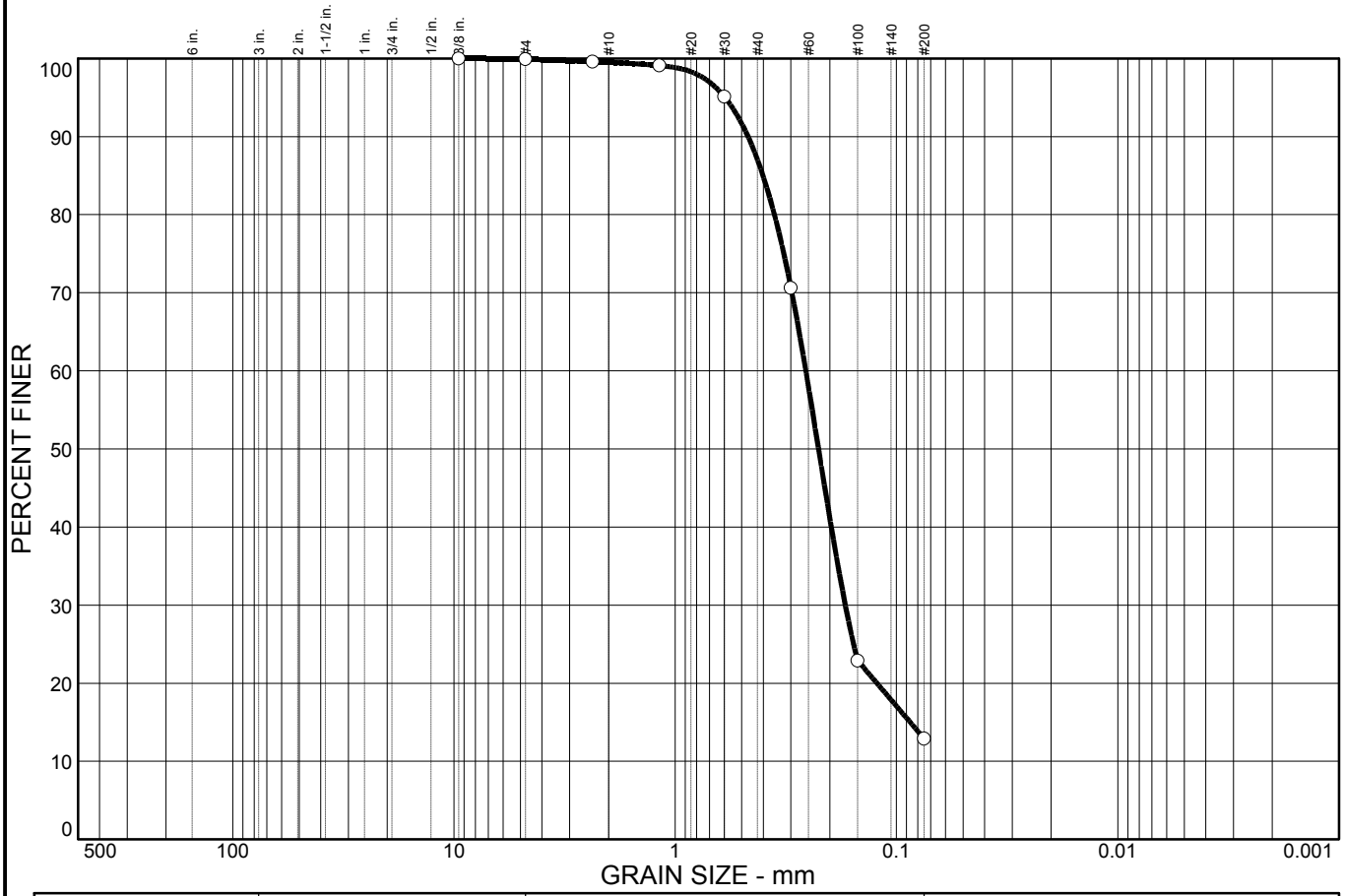
Sample No.: B-2 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 7.5-9'

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.1	0.4	12.5	74.1	12.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.9		
#8	99.6		
#16	99.1		
#30	95.1		
#50	70.6		
#100	22.9		
#200	12.9		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.401 D_{60}= 0.257 D_{50}= 0.225
D_{30}= 0.170 D_{15}= 0.0868 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

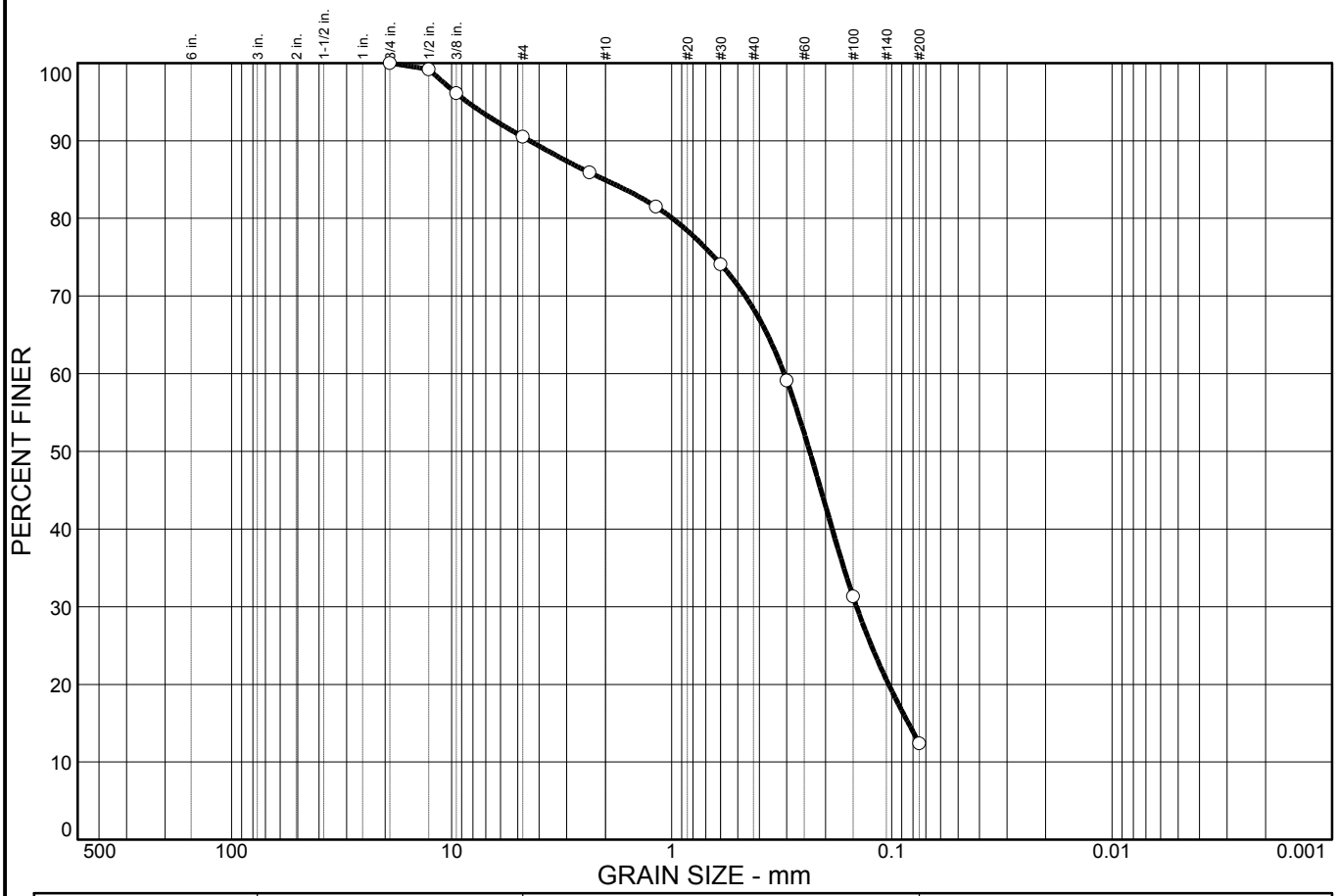
Sample No.: B-2 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 17.5-19'

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	9.5	5.6	16.6	55.9	12.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	99.2		
3/8 in.	96.1		
#4	90.5		
#8	85.9		
#16	81.5		
#30	74.1		
#50	59.1		
#100	31.3		
#200	12.4		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.02 D_{60}= 0.308 D_{50}= 0.236
D_{30}= 0.145 D_{15}= 0.0841 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

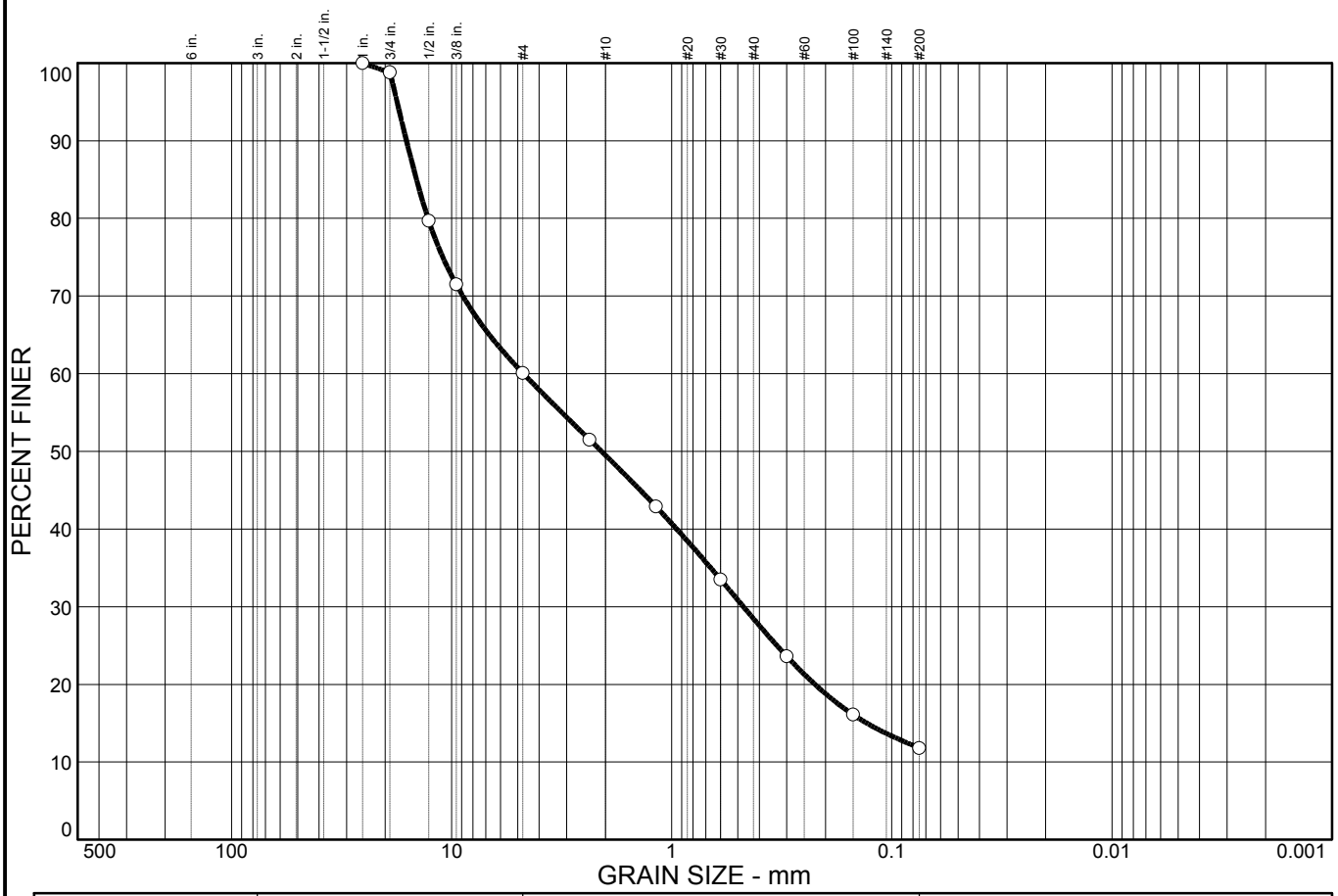
Sample No.: CPT-3 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 1-4'

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	1.2	38.7	10.6	21.1	16.6	11.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	98.8		
1/2 in.	79.7		
3/8 in.	71.5		
#4	60.1		
#8	51.5		
#16	42.9		
#30	33.5		
#50	23.6		
#100	16.1		
#200	11.8		

* (no specification provided)

Material Description

Poorly graded sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 14.4 D_{60}= 4.71 D_{50}= 2.08
D_{30}= 0.473 D_{15}= 0.130 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

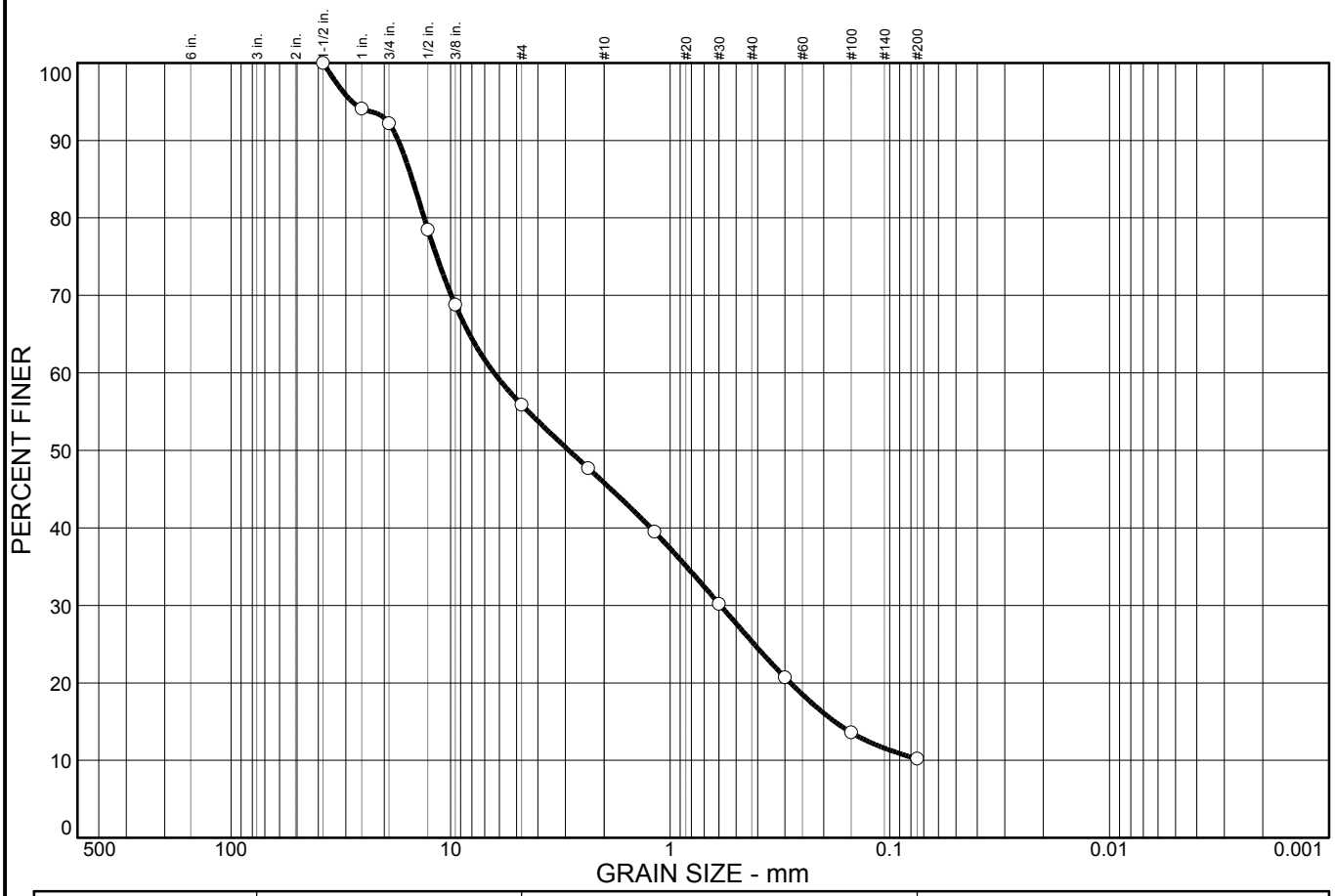
Sample No.: CPT-6 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 8-16"

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	7.8	36.3	10.1	20.5	15.1	10.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	94.1		
3/4 in.	92.2		
1/2 in.	78.5		
3/8 in.	68.8		
#4	55.9		
#8	47.7		
#16	39.5		
#30	30.2		
#50	20.7		
#100	13.6		
#200	10.2		

* (no specification provided)

Material Description

Poorly graded sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 15.0 D_{60}= 6.34 D_{50}= 2.89
D_{30}= 0.592 D_{15}= 0.178 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

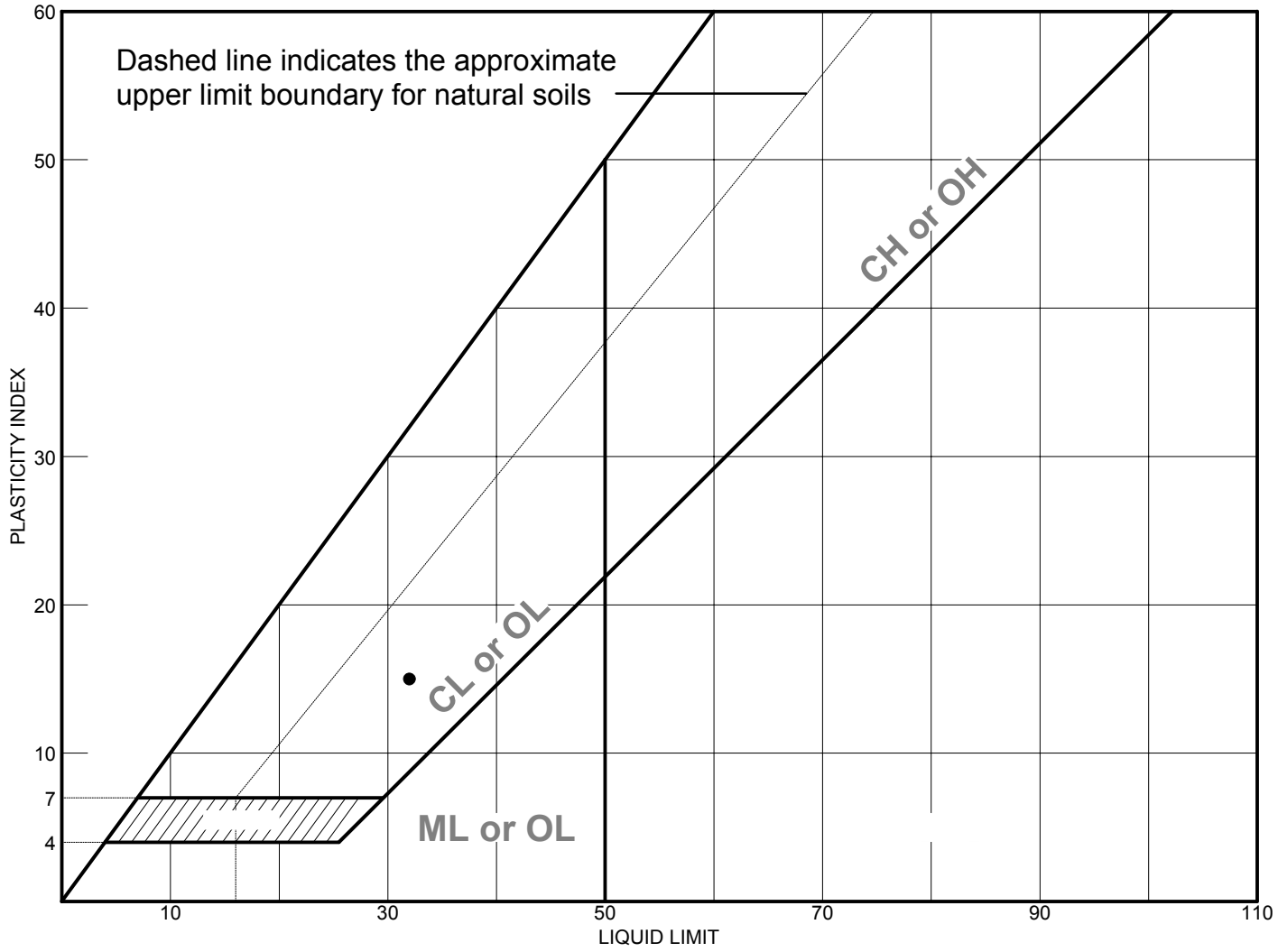
Sample No.: SCPT-7 **Source of Sample:** **Date:** 12/15/25
Location: **Elev./Depth:** 7-12"

Moore Twining Associates, Inc.

Fresno, CA

Client: Michael Baker International, Inc.
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District
Project No: G93908.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay with sand	32	17	15	98.3	84.8	CL

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 7.5-9'

Remarks:

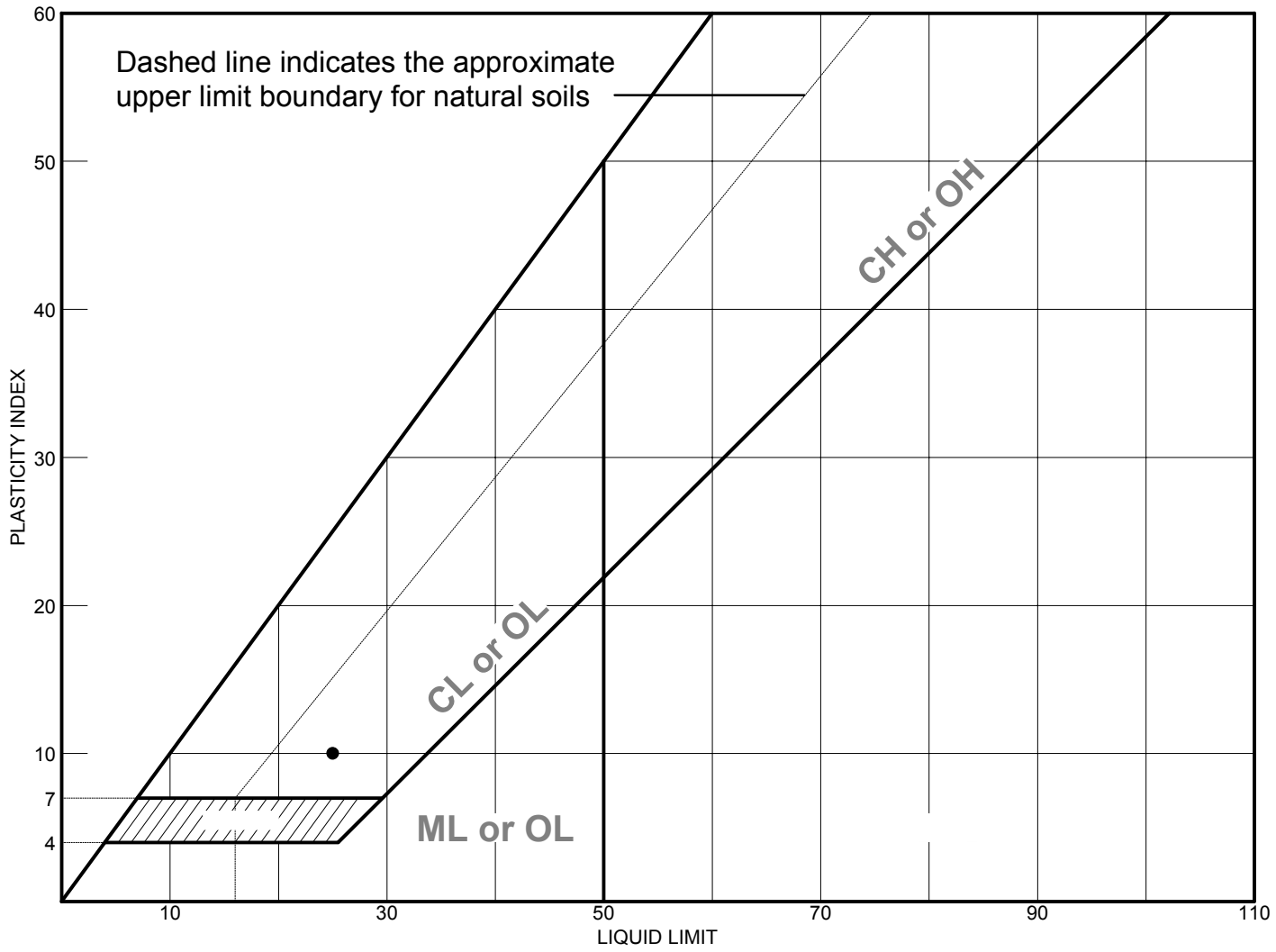
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand with gravel	25	15	10	14.3	12.3	SC

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 25-26.5'

Remarks:

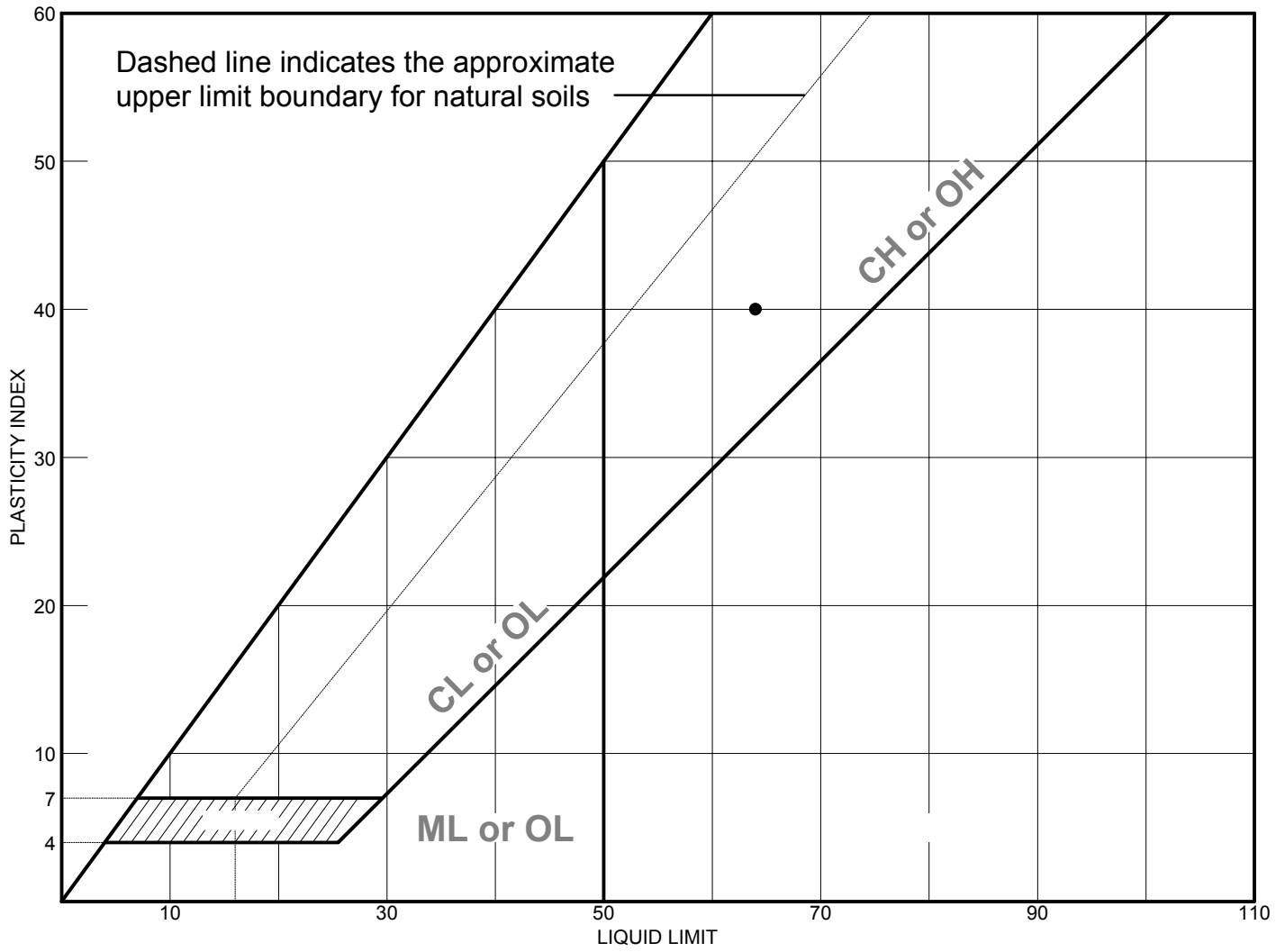
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Fat clay	64	24	40			CH

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 65-66.5'

Remarks:

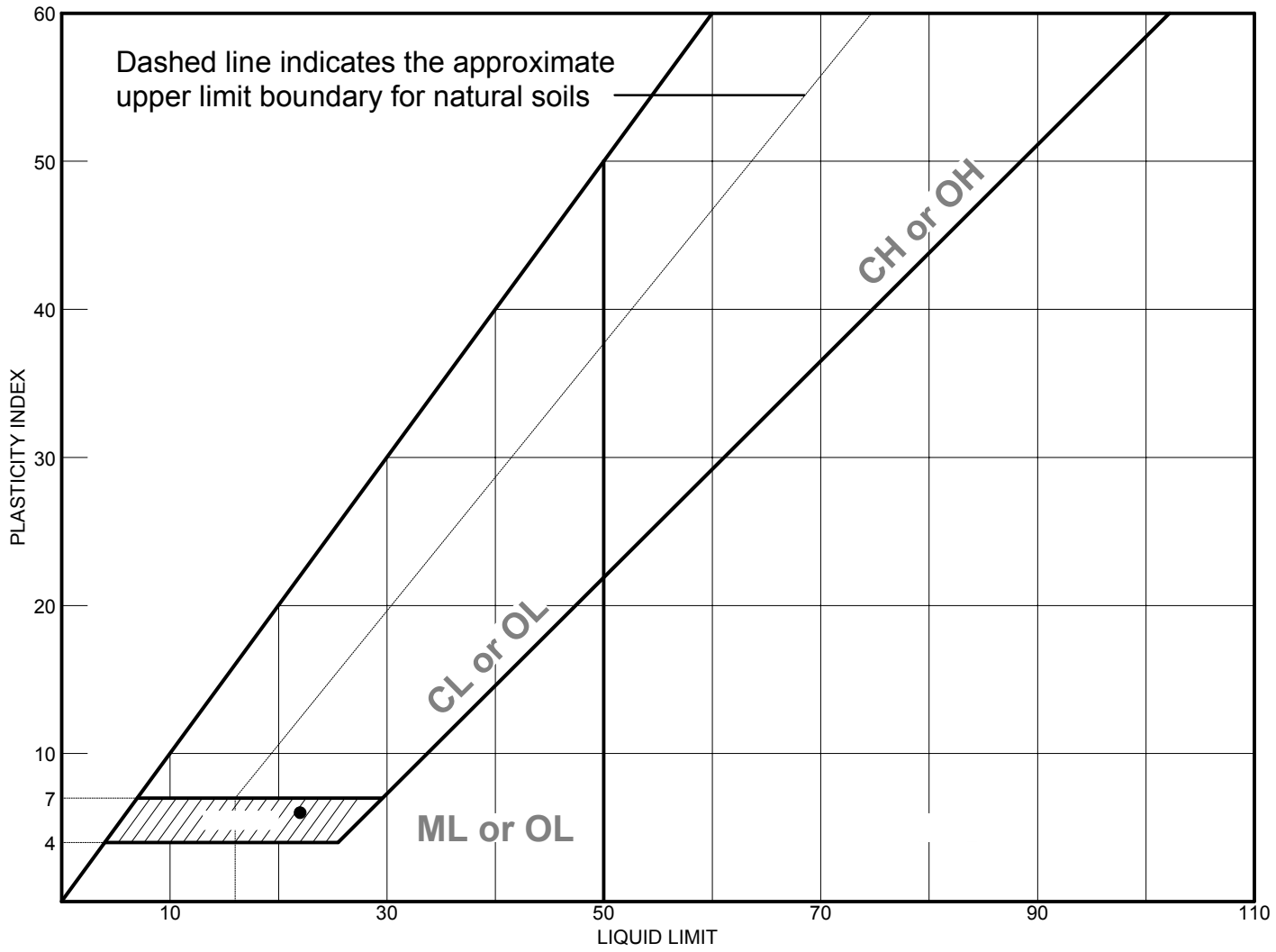
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy silty clay	22	16	6			CL-ML

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 5-6.5'

Remarks:

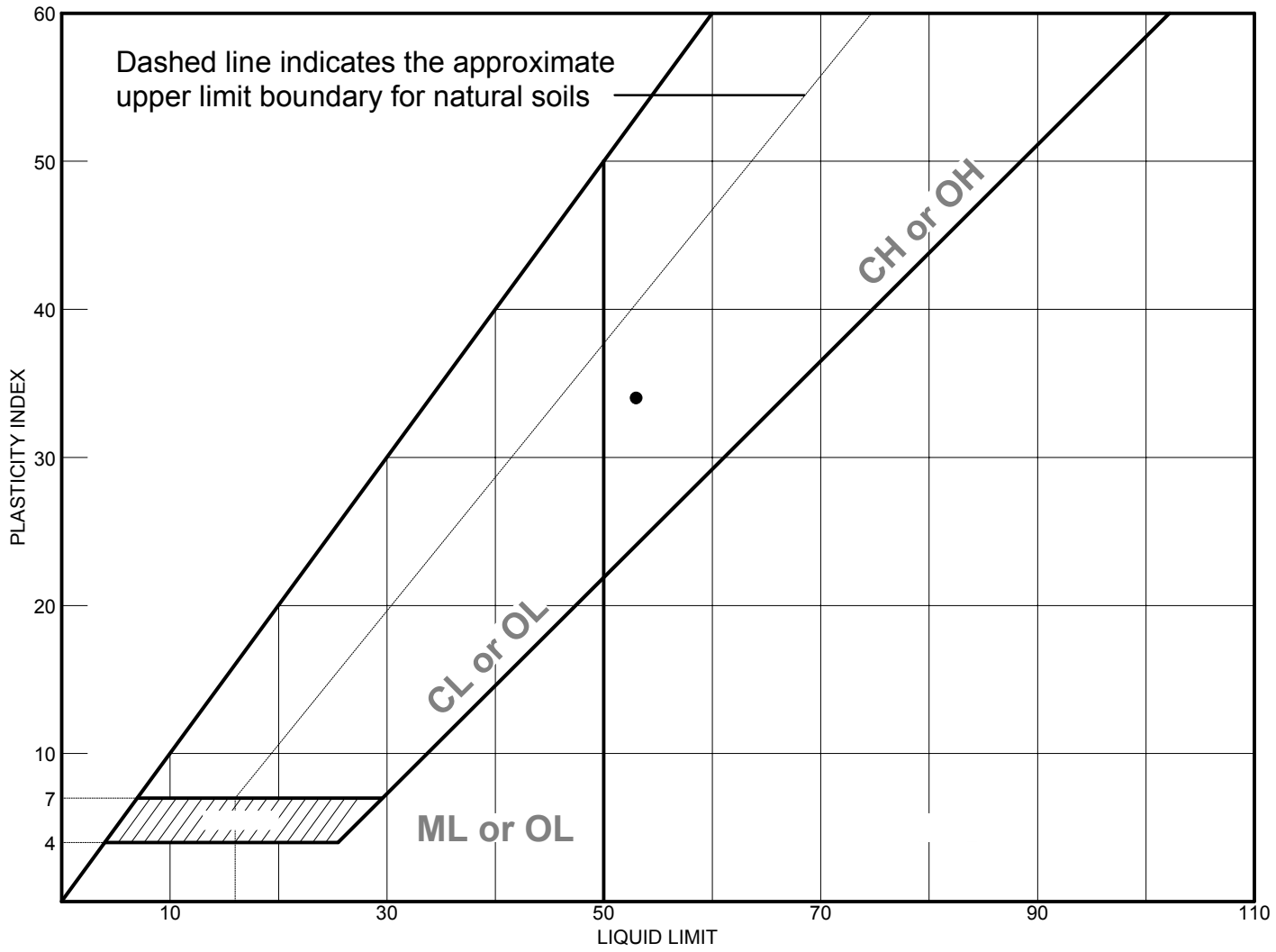
●

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Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Fat clay	53	19	34			CH

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 10-11.5'

Remarks:

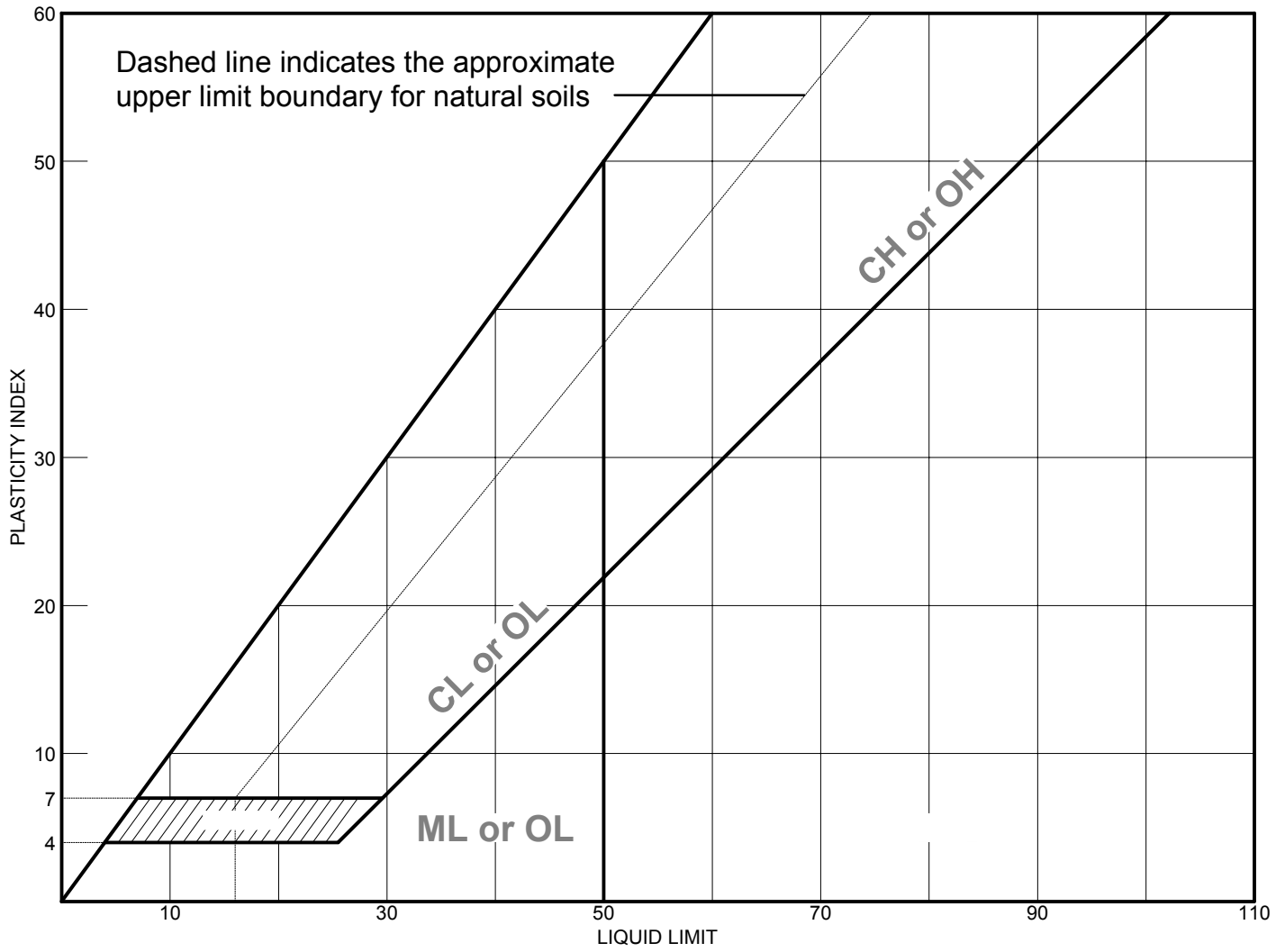
●

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Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt	NV	NP	NP			ML

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 39-41'

Remarks:

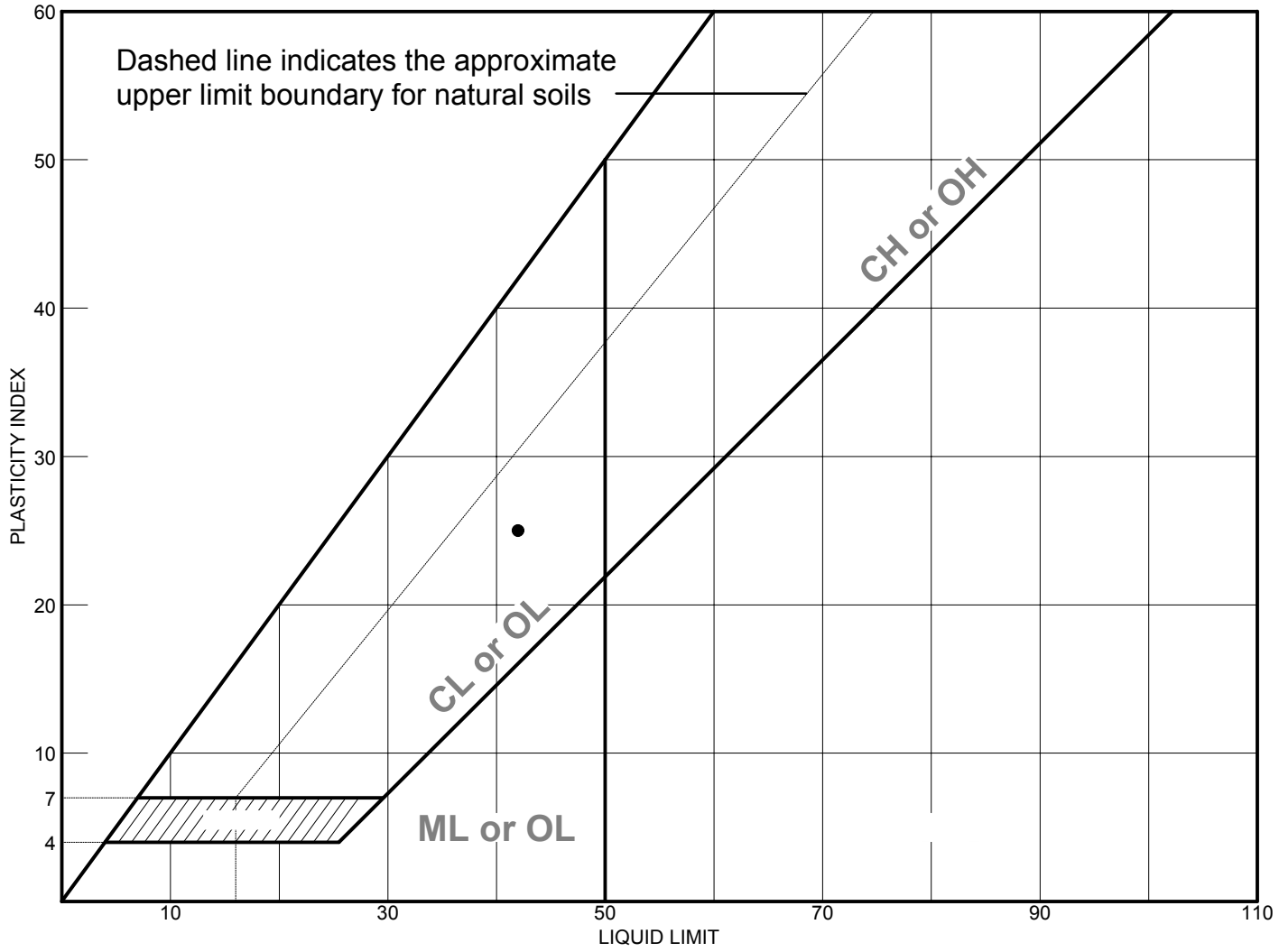
●

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Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay	42	17	25			CL

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 45-45.5'

Remarks:

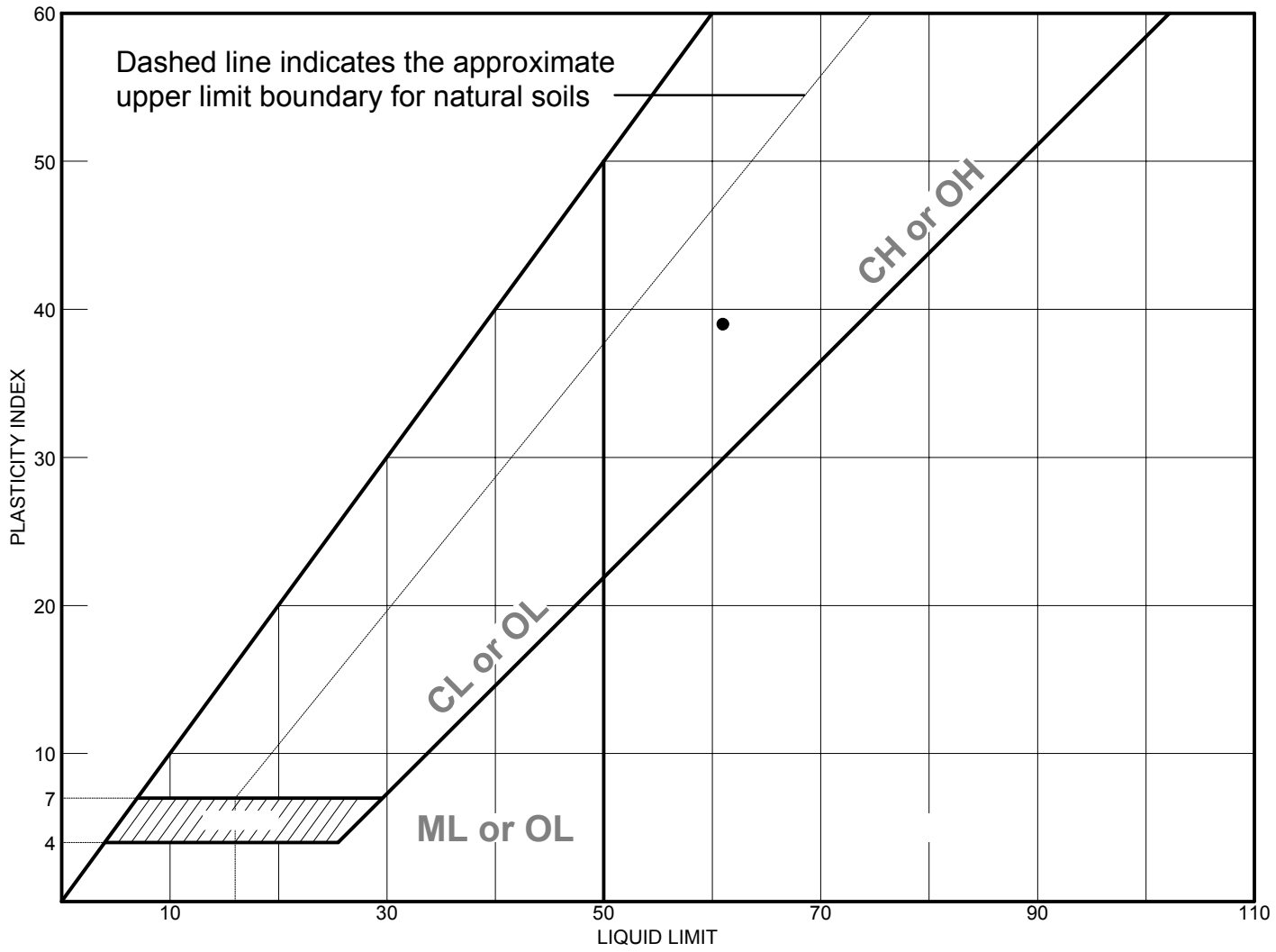
●

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Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Fat clay	61	22	39			CH

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 75-76.5'

Remarks:

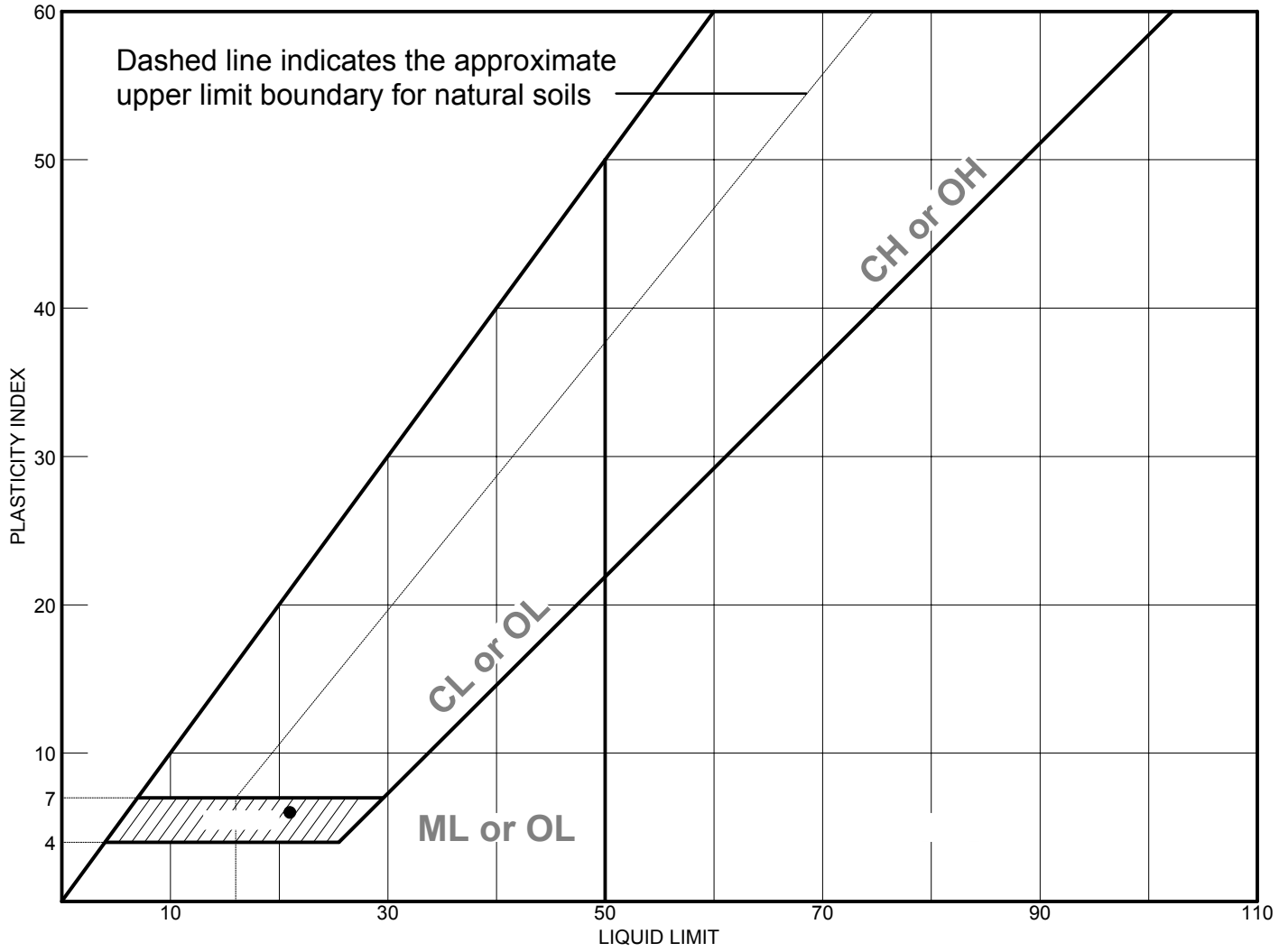
●

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Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty clay	21	15	6			CL-ML

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

● **Source:** **Sample No.:** SCPT-5 **Elev./Depth:** 1-5'

Remarks:

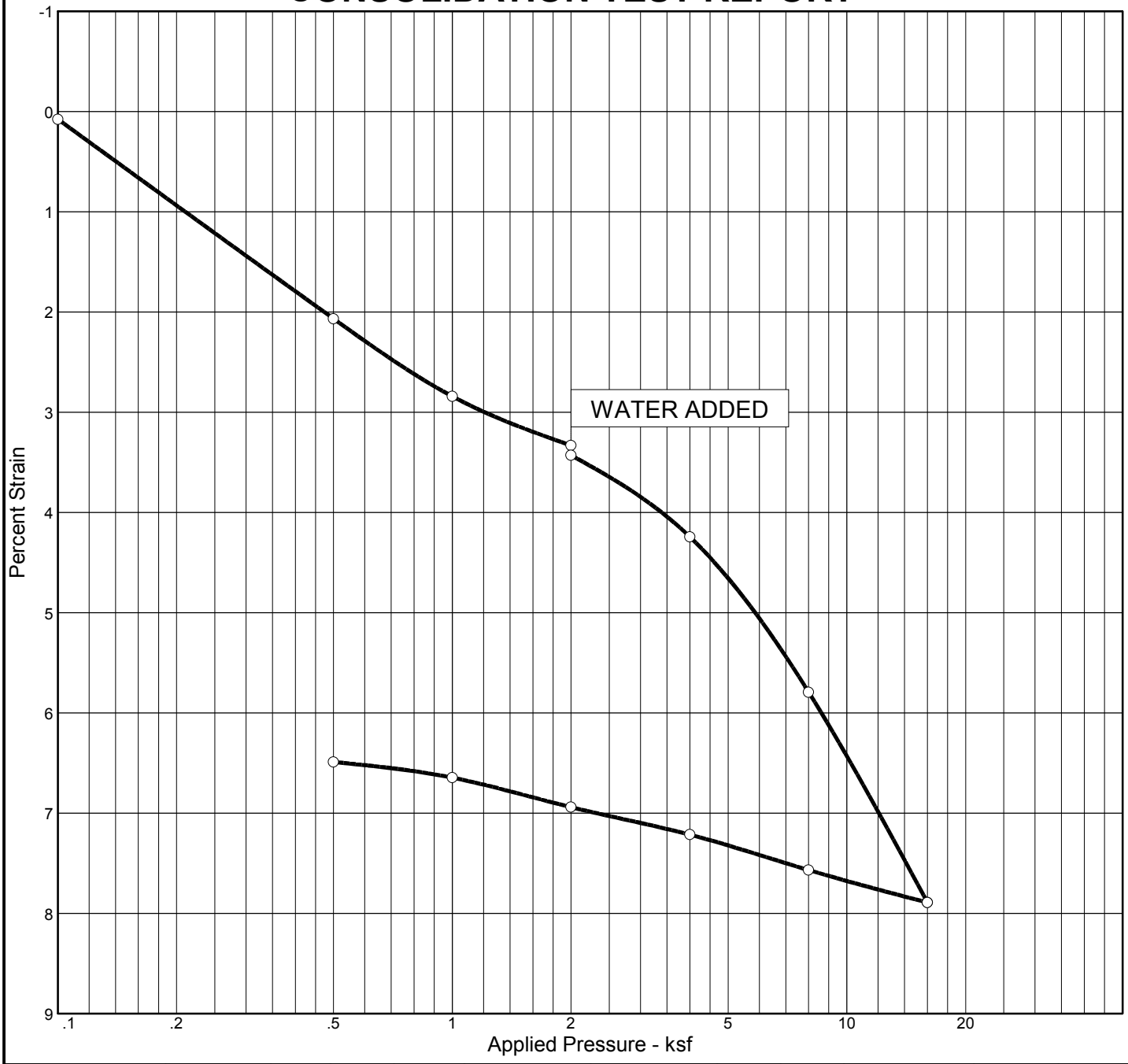
●

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Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
87.9 %	21.6 %	100.1			2.65		5.28	0.12	0.02		0.1	0.652

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand with gravel	SM	

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Source: **Sample No.:** B-1 **Elev./Depth:** 5-6.5'

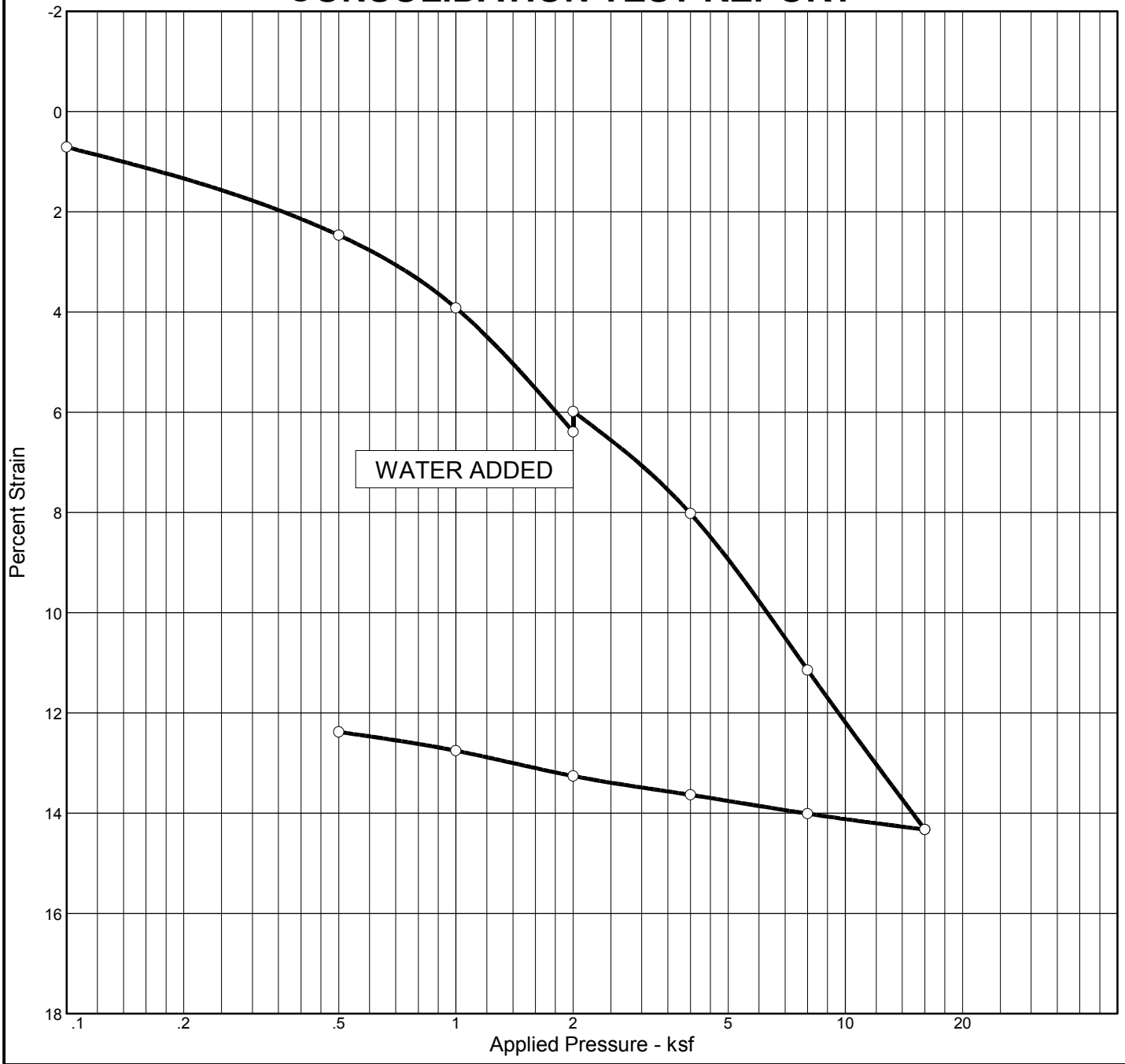
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
102.6 %	31.4 %	91.4			2.65		4.02	0.19	0.02	2.34	0.4	0.810

MATERIAL DESCRIPTION	USCS	AASHTO
Lean clay	CL	

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Source: **Sample No.:** B-1 **Elev./Depth:** 10-11.5'

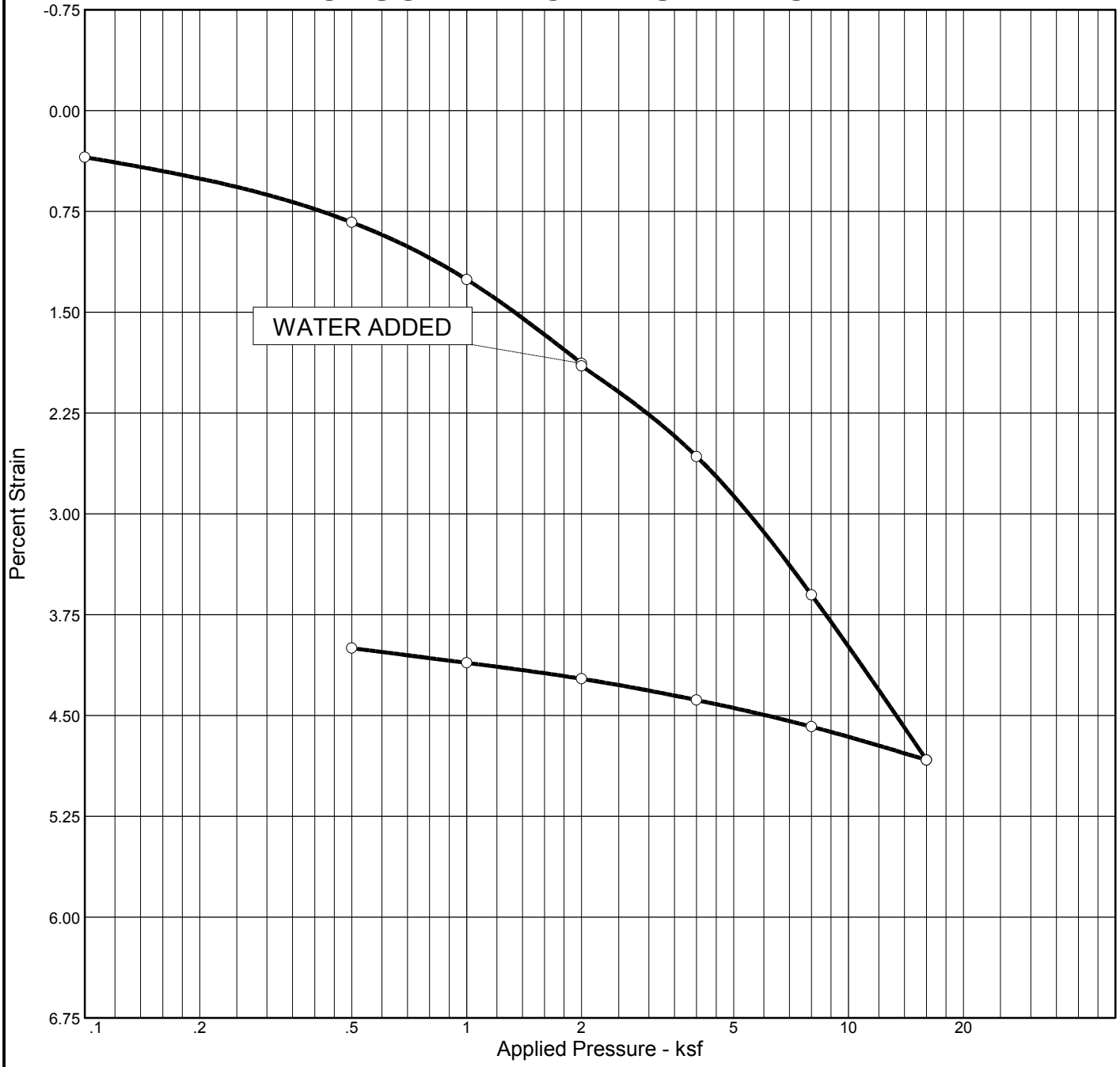
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
119.7 %	27.8 %	102.4			2.65		4.82	0.07	0.01			0.615

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy silty clay	CL-ML	

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Source: **Sample No.:** B-2 **Elev./Depth:** 7.5-9'

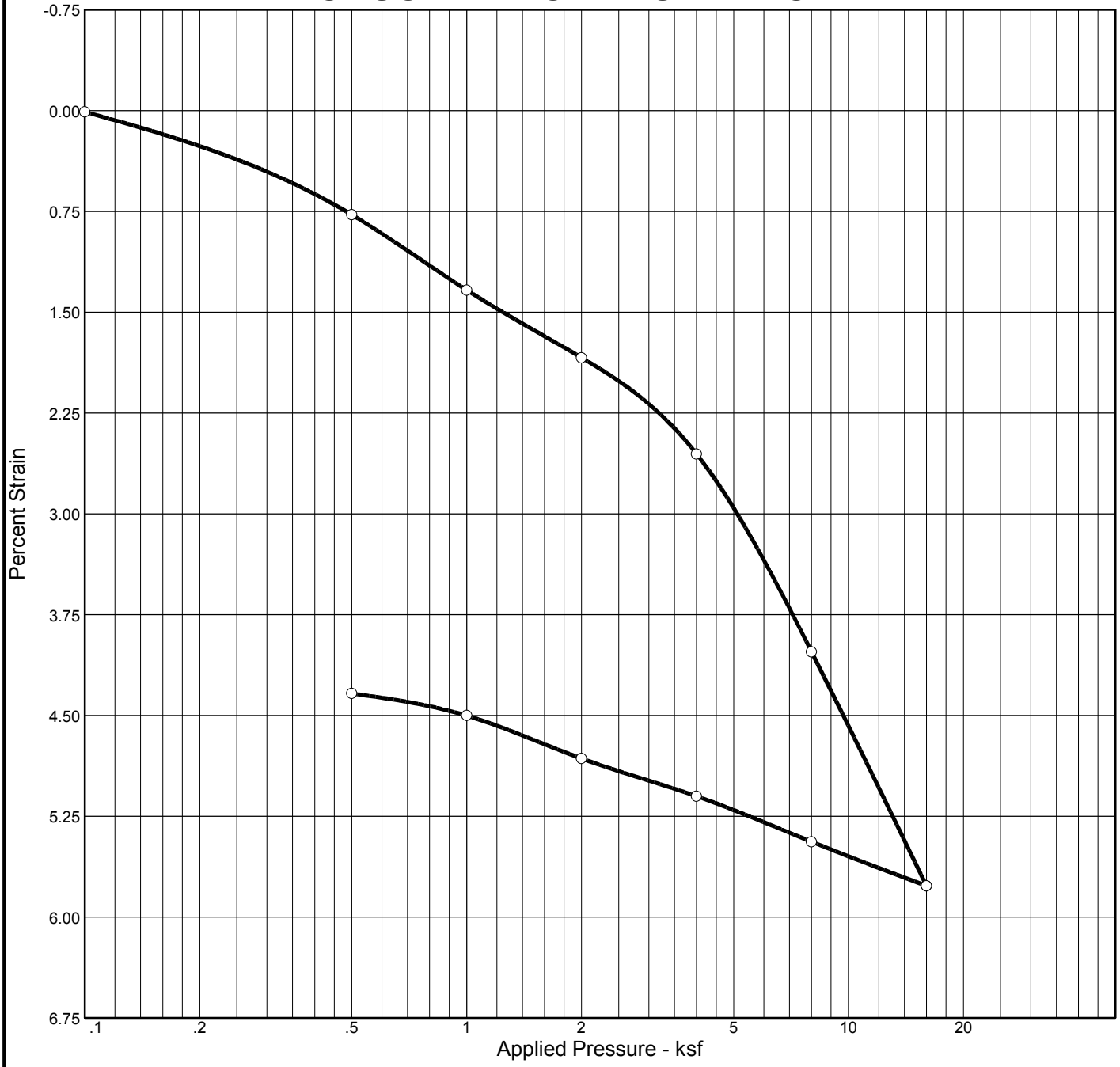
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
103.5 %	29.1 %	94.8			2.65		4.72	0.10	0.02			0.746

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G93908.01 **Client:** Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

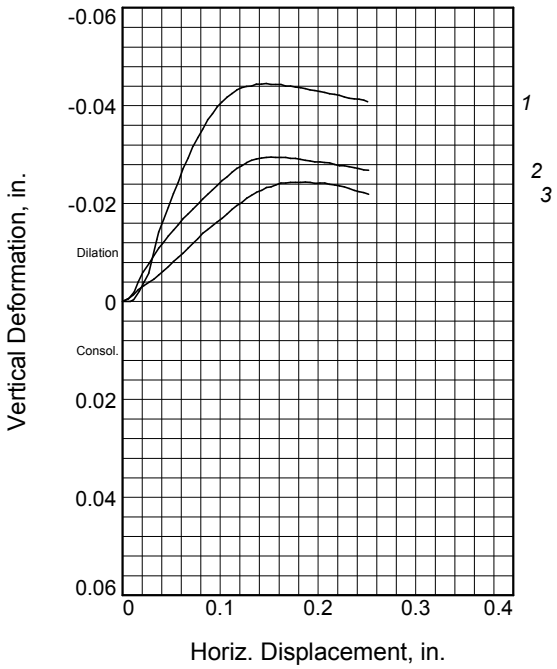
Source: **Sample No.:** B-2 **Elev./Depth:** 41-41.5'

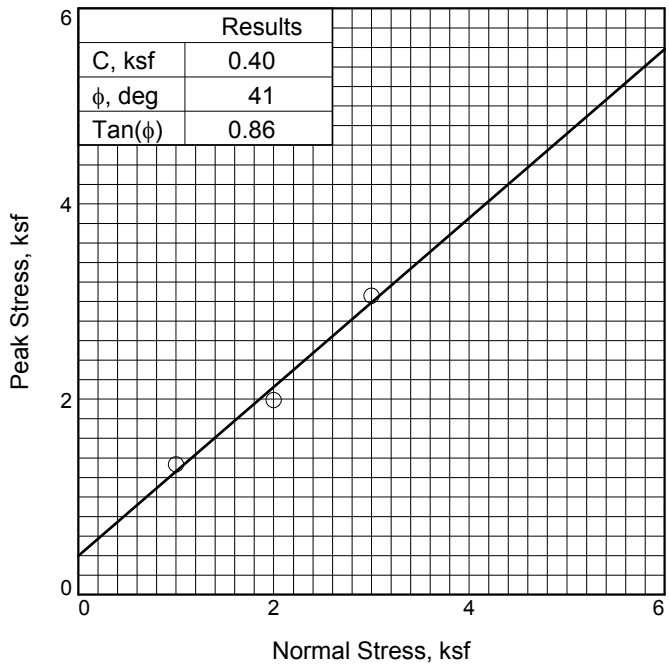
Moore Twining Associates, Inc.

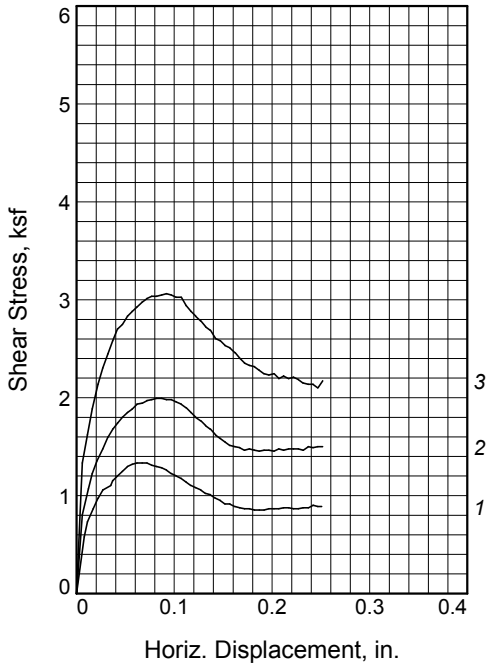
Fresno, CA

Remarks:

Figure







Sample No.		1	2	3
Initial	Water Content, %	18.8	20.5	29.7
	Dry Density, pcf	99.8	106.3	98.6
	Saturation, %	75.5	97.5	116.4
	Void Ratio	0.6579	0.5565	0.6772
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	23.3	19.0	23.7
	Dry Density, pcf	101.3	109.0	100.7
	Saturation, %	97.8	97.2	97.5
	Void Ratio	0.6326	0.5182	0.6436
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.98	0.98
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		1.33	1.99	3.06
Displacement, in.		0.06	0.08	0.09
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		2.6	3.4	3.8

Sample Type:

Description: Poorly graded sand with silt

Specific Gravity= 2.65

Remarks:

Figure _____

Client: Michael Baker International, Inc.

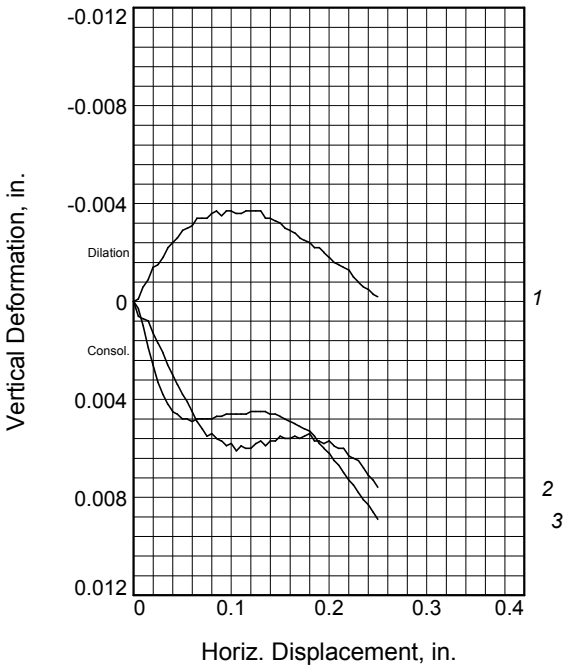
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

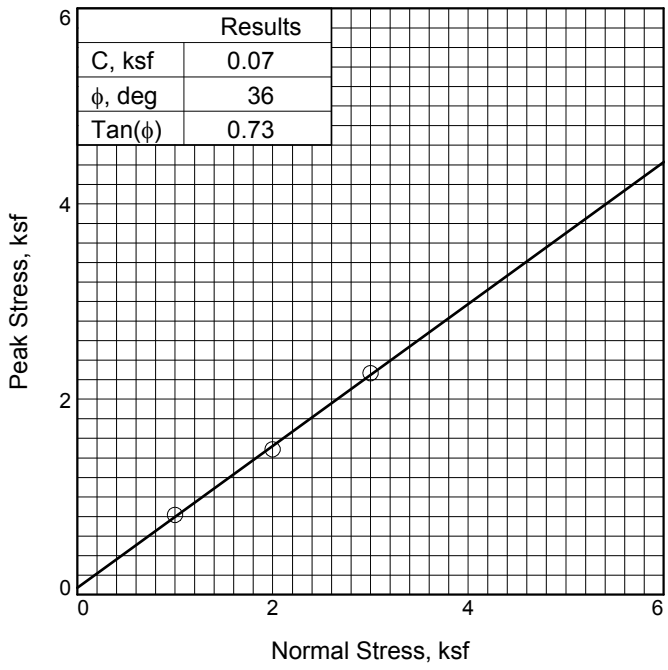
Sample Number: B-1 **Depth:** 15-16.5'

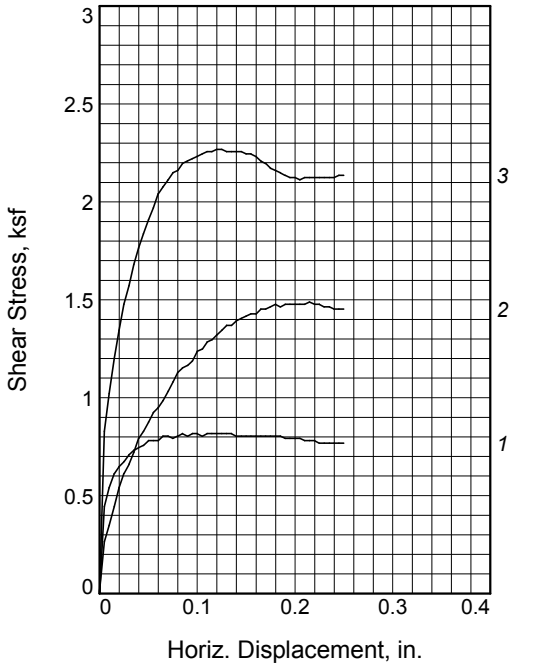
Proj. No.: G93908.01 **Date Sampled:** 12/15/25

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.
Fresno, CA







Sample No.		1	2	3
Initial	Water Content, %	24.6	28.5	24.1
	Dry Density, pcf	95.7	93.7	98.8
	Saturation, %	89.4	98.6	94.9
	Void Ratio	0.7281	0.7649	0.6744
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	25.5	25.1	21.4
	Dry Density, pcf	97.5	98.0	103.7
	Saturation, %	97.1	96.7	95.5
	Void Ratio	0.6965	0.6874	0.5948
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.96	0.95
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		0.82	1.49	2.27
Displacement, in.		0.09	0.22	0.12
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		3.5	8.9	5.0

Sample Type:
Description: Sandy silty clay

LL= 22 **PL=** 16 **PI=** 6
Specific Gravity= 2.65
Remarks:

Figure _____

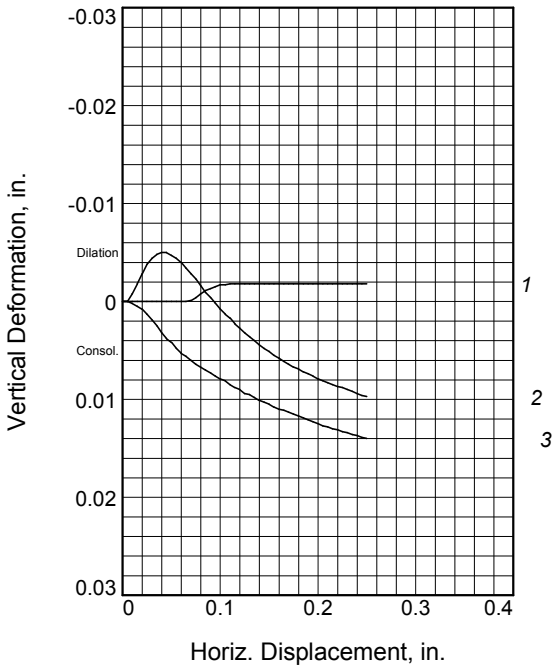
Client: Michael Baker International, Inc.

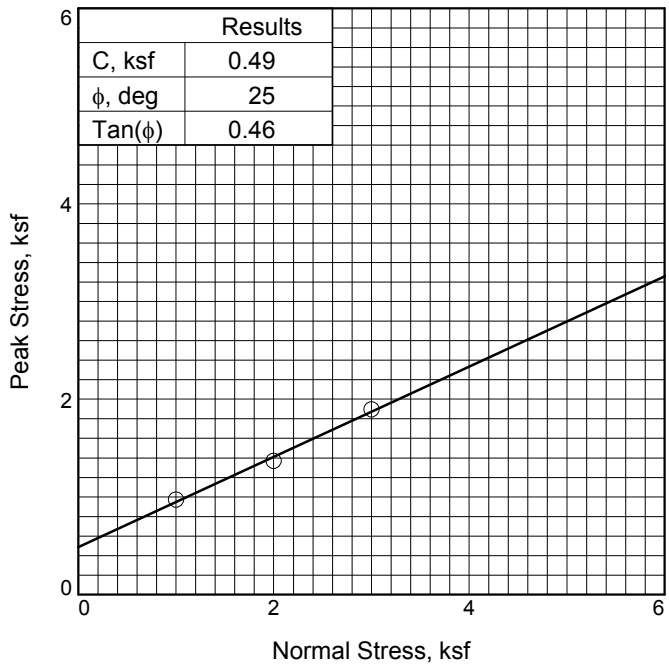
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

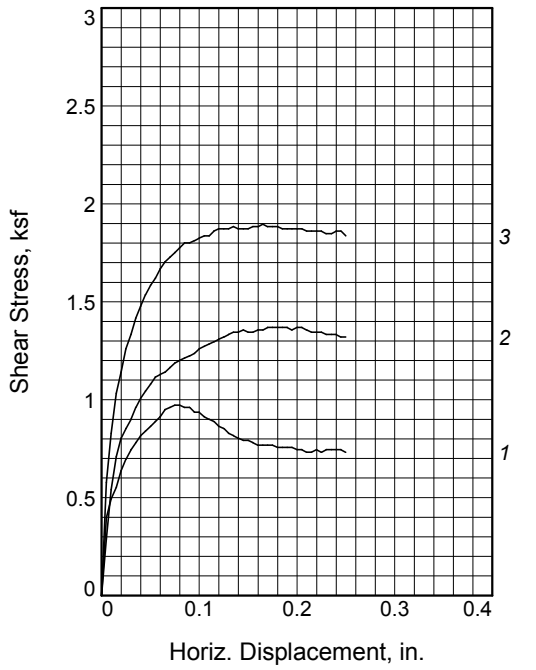
Sample Number: B-2 **Depth:** 5-6.5'

Proj. No.: G93908.01 **Date Sampled:** 12/15/25

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
Fresno, CA







Sample No.		1	2	3
Initial	Water Content, %	36.4	50.8	47.4
	Dry Density, pcf	88.6	74.8	78.0
	Saturation, %	111.2	111.0	112.1
	Void Ratio	0.8676	1.2120	1.1197
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	30.2	38.9	36.7
	Dry Density, pcf	91.8	81.2	89.8
	Saturation, %	99.9	99.4	115.5
	Void Ratio	0.8013	1.0370	0.8418
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.96	0.92	0.87
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		0.97	1.37	1.90
Displacement, in.		0.08	0.17	0.17
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		3.1	7.0	6.8

Sample Type:
Description: Fat clay

Specific Gravity= 2.65
Remarks:

Figure _____

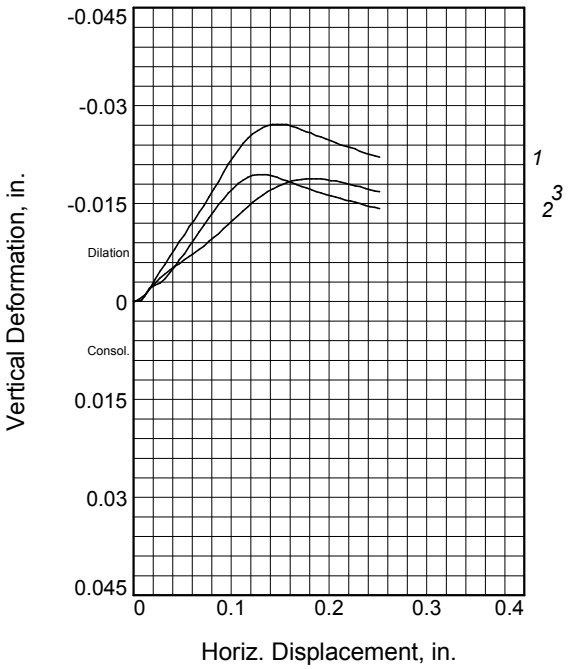
Client: Michael Baker International, Inc.

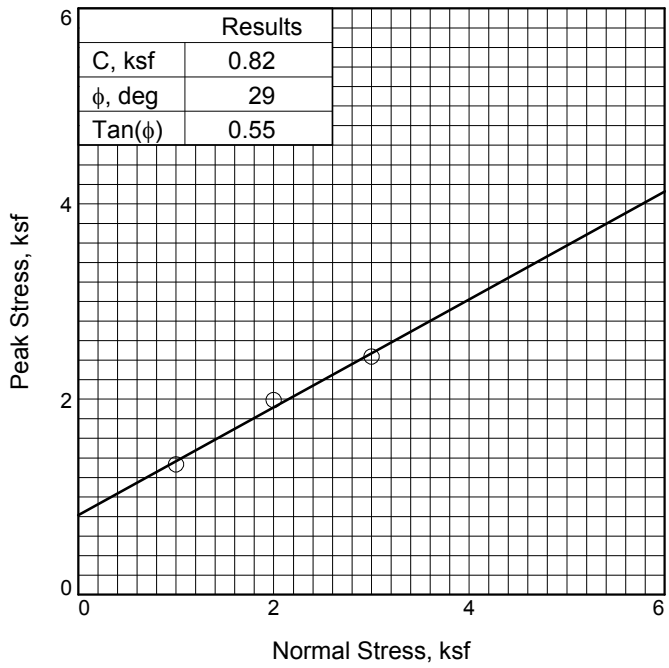
Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

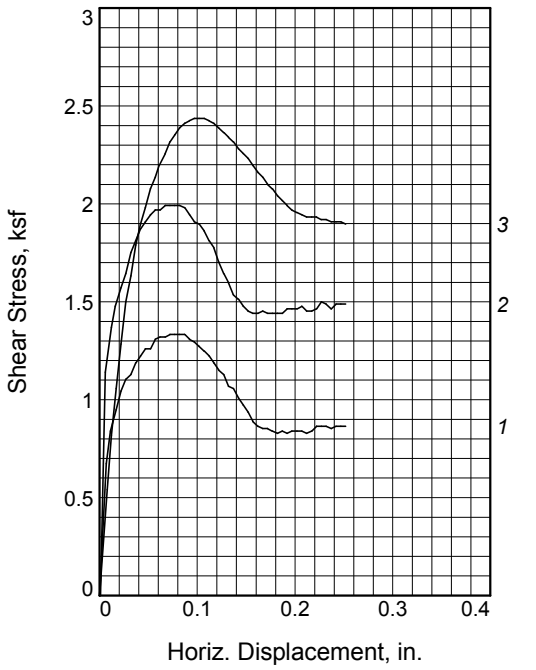
Sample Number: B-2 **Depth:** 15-16.5'

Proj. No.: G93908.01 **Date Sampled:** 12/15/25

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
Fresno, CA







Sample No.		1	2	3
Initial	Water Content, %	24.6	27.4	30.3
	Dry Density, pcf	102.5	100.5	94.6
	Saturation, %	106.5	112.3	107.2
	Void Ratio	0.6134	0.6453	0.7494
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	23.3	25.0	26.7
	Dry Density, pcf	103.9	102.2	98.5
	Saturation, %	104.1	107.2	104.1
	Void Ratio	0.5927	0.6188	0.6803
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.98	0.96
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		1.33	1.99	2.44
Displacement, in.		0.07	0.07	0.10
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		3.0	2.8	4.0

Sample Type:
Description: Clayey sand

Specific Gravity= 2.65
Remarks:

Figure _____

Client: Michael Baker International, Inc.

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Sample Number: B-2 **Depth:** 50-51.5'

Proj. No.: G93908.01 **Date Sampled:** 12/15/25

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
Fresno, CA

MOORE TWINING ASSOCIATES, INC.

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District	REPORT DATE:	2/11/2026
		TEST DATE:	2/10/2026
MTA PROJECT NO.:	G93908.01		
SAMPLE I.D.:	B-2 @ 12-14.5'		
SAMPLED BY:	S. Reich		
SAMPLE DATE:	12/15/2025	TESTED BY:	CJ
MATERIALS DESCRIPTION:	Fat clay		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9350
Pan + Dry Soil Wt., gm	218.4	Dry Soil Wt., lbs	0.6881
Pan Wt., gm	0.0		
Initial % Moisture Content	14.5	Final % Moisture Content	35.9

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.7877	Ring + Sample Wt., lbs	0.9350
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.7877	Remolded Wt., lbs	0.9350
Remolded Wet Density, pcf	108.3	Remolded Wet Density, pcf	118.2
Remolded Dry Density, pcf	94.6	Remolded Dry Density, pcf	87.0

Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.007911
Initial Gage Reading, in:	0.0448		
Final Gage Reading, in:	0.1326		
Expansion, in:	0.0878		
Expansion Index	88	**Comments:**	**Medium Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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FX: 559.268.7126
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Fresno, CA 93721

MOORE TWINING ASSOCIATES, INC.

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District	REPORT DATE:	2/11/2026
		TEST DATE:	2/10/2026
MTA PROJECT NO.:	G93908.01		
SAMPLE I.D.:	SCPT-5 @ 1-5'		
SAMPLED BY:	S. Reich		
SAMPLE DATE:	12/15/2025	TESTED BY:	CJ
MATERIALS DESCRIPTION:	Silty clay		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9612
Pan + Dry Soil Wt., gm	228.2	Dry Soil Wt., lbs	0.8152
Pan Wt., gm	0.0		
Initial % Moisture Content	9.6	Final % Moisture Content	17.9

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8931	Ring + Sample Wt., lbs	0.9612
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8931	Remolded Wt., lbs	0.9612
Remolded Wet Density, pcf	122.8	Remolded Wet Density, pcf	131.5
Remolded Dry Density, pcf	112.1	Remolded Dry Density, pcf	111.5

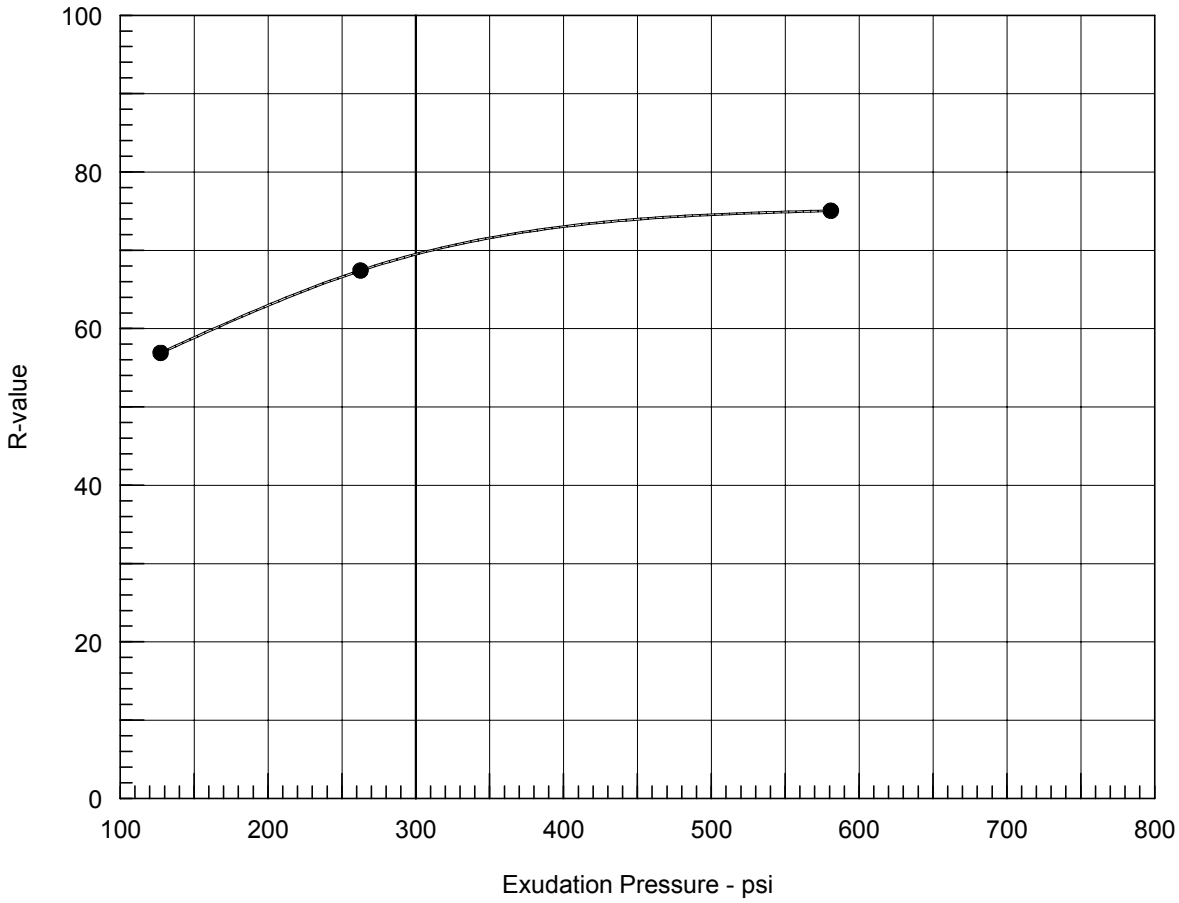
Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.007312
Initial Gage Reading, in:	0.0522		
Final Gage Reading, in:	0.0577		
Expansion, in:	0.0055		
Expansion Index	6	**Comments:**	**Ver Low Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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2527 Fresno Street
Fresno, CA 93721

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	150	117.9	13.3	0.00	42	2.33	127	61	57
2	350	120.6	12.2	0.00	34	2.31	263	71	67
3	350	122.0	11.1	0.00	26	2.35	581	77	75

Test Results	Material Description
R-value at 300 psi exudation pressure = 69	Silty sand

Project No.: G93908.01

Project: Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District

Sample Number: CPT-3 **Depth:** 1-4'

Date: 3/20/2026

Tested by: JR

Checked by: MS

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure N/A

MOORE TWINING ASSOCIATES, INC.

Project Name:	Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District	Report Date:	2/11/2026
		Sample Date:	12/15/2025
Project Number:	G93908.01		
		Sampled By:	S. Reich
Subject:	Minimum Resistivity, ASTM G187	Tested By:	Reggie S.
Material Description:	Silty sand	Test Date:	2/3/2026
Location:	B-1 @ 1-4'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	22,000
75 mls	21,000
100 mls	9,500
125 mls	8,000
150 mls	6,700
175 mls	5,700
200 mls	5,200
225 mls	5,300

Remarks: Min. Resistivity is 5,200 Ohm-cm

www.mooretwining.com

PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

MOORE TWINING ASSOCIATES, INC.

Project Name:	Proposed P1 Parking Structure, Port of Hueneme, Oxnard Harbor District	Report Date:	2/11/2026
		Sample Date:	12/15/2025
Project Number:	G93908.01		
		Sampled By:	S. Reich
Subject:	Minimum Resistivity, ASTM G187	Tested By:	Reggie S.
Material Description:	Silty sand	Test Date:	2/3/2026
Location:	B-2 @ 1-4'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	4,600
75 mls	3,700
100 mls	3,400
125 mls	3,300
150 mls	3,400

Remarks: Min. Resistivity is 3,300 Ohm-cm

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MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

January 30, 2026

Work Order #: **MA16011**

Shaun Reich
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: Port Hueneme

Enclosed are the analytical results for samples received by our laboratory on **01/16/26** . For your reference, these analyses have been assigned laboratory work order number **MA16011**.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.



Lauren Cox
Client Services Representative

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division	**Project:** Port Hueneme	**Reported:**
2527 Fresno Street	**Project Number:** G93908.01	01/30/2026
Fresno CA, 93721	**Project Manager:** Shaun Reich	

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B-1 @ 0-4		MA16011-01	Soil	12/17/25 00:00	01/16/26 11:22

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

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California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division	**Project:** Port Hueneme	**Reported:**
2527 Fresno Street	**Project Number:** G93908.01	01/30/2026
Fresno CA, 93721	**Project Manager:** Shaun Reich	

B-1 @ 0-4

MA16011-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Flag
Inorganics								
Chloride	**0.0010**	0.00060	% by Weight	[CALC]	01/28/26	01/28/26	[CALC]	
Chloride	**10**	6.0	mg/kg	B6A2608	01/26/26	01/28/26	Cal Test 422	
pH	**8.2**	0.10	pH Units	B6A2608	01/26/26	01/28/26	Cal Test 643 M	
Sulfate as SO4	**0.0034**	0.00060	% by Weight	[CALC]	01/28/26	01/28/26	[CALC]	
Sulfate as SO4	**34**	6.0	mg/kg	B6A2608	01/26/26	01/28/26	Cal Test 417	

Notes and Definitions

ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference
mg/kg	milligrams per kilogram (parts per million concentration units)

MOORE TWINING ASSOCIATES, INC.

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION #: 13971

CHAIN OF CUSTODY // ANALYSIS REQUEST

2527 FRESNO STREET • FRESNO, CA 93721 • PH: (559) 268-7021 • FAX: (559) 268-0740

WORK ORDER #: MA16011
PAGE 011 OF 011

REPORT TO:
ATTENTION: S. Reich
COMPANY NAME: Geotechnical Division
ADDRESS:
PHONE:
EMAIL / FAX:

INVOICE TO:
ATTENTION:
COMPANY NAME:
ADDRESS:
PHONE:
EMAIL / FAX:

REPORT COPY TO:

REPORTING:
☑ STANDARD FORMAT ☐ PDF
☐ EDDT (SOURCE) ☐ EXCEL
☐ GEOTRACKER/COELT (LUFT)
GLOBAL ID:
☐ COUNTY ENVIRONMENTAL HEALTH:
☐ STATE WATER RESOURCES CONTROL BOARD:
☐ OTHER:

SAMPLE INFORMATION
SAMPLED BY (PRINT):
SIGNATURE:
☐ PUBLIC SYSTEM ☐ ROUTINE
☐ PRIVATE WELL ☐ REPEAT
☑ OTHER ☐ REPLACEMENT

TURN AROUND TIME
☑ STANDARD
☐ RUSH, DUE ON: 10 Day TAT

SAMPLE TYPES
SOLID:
BS - BIOSOLID
CR - CERAMIC
SL - SOIL/SOLID
LIQUID:
DW - DRINKING WATER
GW - GROUND WATER
OL - OIL
SF - SURFACE WATER
ST - STORM WATER
WW - WASTEWATER

PROJECT INFORMATION
CONTRACT / P.O. NO.:
PROJECT: Port of Hueneme
PROJECT NUMBER: G93908.01
PROJECT MANAGER: S Reich

ANALYSIS REQUESTED

LAB USE

NOTES ON RECEIVED CONDITION:
☐ CUSTODY SEAL(S) BROKEN ☐ SAMPLES(S) DAMAGED
☐ ON ICE ☐ AMBIENT TEMP. ☐ INCORRECT PRESERVATION

CLIENT SAMPLE ID	DATE	TIME	TYPE	PH	Sulfate	Chloride								STATION CODE
B-1@0-4	12-17-25		SL	x	x	x								

COMMENTS / ADDITIONAL INSTRUCTIONS:

RELINQUISHED BY	COMPANY	DATE	TIME	RECEIVED BY	COMPANY
Cameron Galkson		1-16-26	1122		

Payment for services rendered as noted herein are due in full within 30 days from the date invoiced. If not so paid, account balances are deemed delinquent. Delinquent balances are subject to monthly service charges and interest specified in MTA's current Standard Terms and Conditions for Laboratory Services. The person signing for the Client/Company acknowledges that they are either the Client or an authorized agent for the Client, that the Client agrees to be responsible for payment for these services on this Chain of Custody and agrees to MTA's terms and conditions for laboratory services unless contractually bound otherwise. MTA's current terms and conditions can be obtained by contacting our accounting department at (559) 268-7021.

FILE: SC-00005-04AA

Sample Integrity

Page 2 of 2

Moore Twining Associates WO# MA 16011

MTA Bottles: Yes or No

COC Info

Was temperature within range? Chemistry ≤6°C Micro <10°C Temp_____°C	Yes No N/A	Did all bottle labels agree with COC?	Yes No N/A	Were there bubbles in VOA vials? (Volatiles Only)	Yes No N/A
If samples were taken today, is there evidence that chilling has begun? Recvd_____C°	Yes No N/A	Was a sufficient amount of sample received?	Yes No N/A	Was PM notified of discrepancies? PM: By/Time:	Yes No N/A
Did all bottles arrive unbroken and intact?	Yes No N/A	Were correct containers and preservatives received for the tests requested?	Yes No N/A		
Do samples have a hold time <72 hours?	Yes No N/A				

Bottles Received

125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)	1																	
Bacti $Na_2S_2O_3$ Thio 120mL																		
None (P)																		
None (AG)																		
None (CG) 500ml																		
Cr6 Buffer (P) Borate Carbonate Buffer-EPA 7196A																		
Dissolved Oxygen 300ml (P)																		
HNO_3 (P)																		
HCl (AG)																		
H_2SO_4 (P)																		
H_3PO_4 (AG) = TOC																		
NaOH (P), cyanide																		
NaOH + ZnAc (P) sodium hydroxid+zinc acetate																		
$Na_2S_2O_3$ (AG) Thio EPA 547, 538, TTHM																		
$Na_2S_2O_3$ (CG) Thio EPA, 504.1																		
$Na_2S_2O_3$ 250ml (Brown P) EPA 549																		
Thio/K Citrate(AG) EPA 531.2																		
NH_4Cl (AG) 552- ammonium chloride EPA HAA5																		
Other:																		
Client Own																		
Low Level Hg/Metals Double Bag																		
Plastic Bag	1																	
Glass Jar: 125 / 250 / 500																		
Soil Tube: Brass / Steel / Plastic																		
5g Encore																		
Cubitaner 1G / 2.5 G																		

Comments

Filter or Split

	Container	Preservative	Date/Time/Initials
S P F			
S P F			
S P F			
S P F			
S P F			

Labeled by:_____@_____ Labels checked by: fm @ 1124

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

January 30, 2026

Work Order #: **MA16012**

Shaun Reich
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: Port Hueneme

Enclosed are the analytical results for samples received by our laboratory on **01/16/26** . For your reference, these analyses have been assigned laboratory work order number **MA16012**.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.



Lauren Cox
Client Services Representative

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division	**Project:** Port Hueneme	**Reported:**
2527 Fresno Street	**Project Number:** G93908.01	01/30/2026
Fresno CA, 93721	**Project Manager:** Shaun Reich	

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B-2 @ 0-4		MA16012-01	Soil	12/18/25 00:00	01/16/26 11:22

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division	**Project:** Port Hueneme	**Reported:**
2527 Fresno Street	**Project Number:** G93908.01	01/30/2026
Fresno CA, 93721	**Project Manager:** Shaun Reich	

B-2 @ 0-4

MA16012-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Flag
Inorganics								
Chloride	ND	0.00060	% by Weight	[CALC]	01/28/26	01/28/26	[CALC]	
Chloride	ND	6.0	mg/kg	B6A2608	01/26/26	01/28/26	Cal Test 422	
pH	**8.1**	0.10	pH Units	B6A2608	01/26/26	01/28/26	Cal Test 643 M	
Sulfate as SO4	**0.011**	0.00060	% by Weight	[CALC]	01/28/26	01/28/26	[CALC]	
Sulfate as SO4	**110**	6.0	mg/kg	B6A2608	01/26/26	01/28/26	Cal Test 417	

Notes and Definitions

ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference
mg/kg	milligrams per kilogram (parts per million concentration units)

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

MOORE TWINING ASSOCIATES, INC.

CHAIN OF CUSTODY // ANALYSIS REQUEST

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ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION #: 13971

WORK ORDER #: MA16012
PAGE 011 OF 011

REPORT TO:	☐ INVOICE TO:	☐ REPORT COPY TO:	REPORTING:
ATTENTION: S Reich	ATTENTION:		☑ STANDARD FORMAT ☐ PDF ☐ EDT (SWRCB) ☐ EXCEL ☐ GEOTRACKER/COELT (LUFT) GLOBAL ID: _____ ☐ COUNTY ENVIRONMENTAL HEALTH: _____ ☐ STATE WATER RESOURCES CONTROL BOARD: _____ ☐ OTHER: _____
COMPANY NAME: Geotechnical Division	COMPANY NAME:		
ADDRESS:	ADDRESS:		
PHONE:	PHONE:		
EMAIL / FAX:	EMAIL / FAX:		

SAMPLE INFORMATION

SAMPLED BY (PRINT):

SIGNATURE:

☐ PUBLIC SYSTEM ☐ ROUTINE
☐ PRIVATE WELL ☐ REPEAT
☑ OTHER ☐ REPLACEMENT

TURN AROUND TIME
☑ STANDARD
☐ RUSH, DUE ON: 10 Day TAT

SAMPLE TYPES

SOLID:
BS - BIOSOLID
CR - CERAMIC
SL - SOIL/SOLID
LIQUID:
DW - DRINKING WATER
GW - GROUND WATER
OL - OIL
SF - SURFACE WATER
ST - STORM WATER
WW - WASTEWATER

PROJECT INFORMATION

CONTRACT / P.O. NO.:

PROJECT: Port of Hueneme

PROJECT NUMBER: G93908.01

PROJECT MANAGER: S Reich

ANALYSIS REQUESTED

LAB USE

NOTES ON RECEIVED CONDITION:
☐ CUSTODY SEAL(S) BROKEN ☐ SAMPLES(S) DAMAGED
☐ ON ICE ☐ AMBIENT TEMP. ☐ INCORRECT PRESERVATION

CLIENT SAMPLE ID	DATE	TIME	TYPE	PH	Sulfate	Chloride									SAMPLING CODE
B-2@0-4	12-18-25		SL	x	x	x									

COMMENTS / ADDITIONAL INSTRUCTIONS:

RELINQUISHED BY	COMPANY	DATE	TIME	RECEIVED BY	COMPANY
Cameron Sadoski		1-16-26	1122		

Payment for services rendered as noted herein are due in full within 30 days from the date invoiced. If not so paid, account balances are deemed delinquent. Delinquent balances are subject to monthly service charges and interest specified in MTA's current Standard Terms and Conditions for Laboratory Services. The person signing for the Client/Company acknowledges that they are either the Client or an authorized agent of the Client, that the Client agrees to be responsible for payment for the services on this Chain of Custody and agrees to MTA's terms and conditions for laboratory services unless contractually bound otherwise. MTA's current terms and conditions can be obtained by contacting our accounting department at (559) 268-7021.

FLSC-00005-04A

Sample Integrity — Page 2 of 2 — Moore Twining Associates — WO# MA 16012 — MTA Bottles: Yes or No

COC Info

Was temperature within range? Chemistry ≤6°C Micro <10°C Temp_____°C	Yes No (N/A)	Did all bottle labels agree with COC?	Yes No N/A	Were there bubbles in VOA vials? (Volatiles Only)	Yes No N/A
If samples were taken today, is there evidence that chilling has begun? Recvd_____C°	Yes No (N/A)	Was a sufficient amount of sample received?	Yes No N/A	Was PM notified of discrepancies? PM: By/Time:	Yes Noe N/A
Did all bottles arrive unbroken and intact?	(Yes) No N/A	Were correct containers and preservatives received for the tests requested?	Yes No N/A		
Do samples have a hold time ≤72 hours?	Yes (No) N/A				

Bottles Received

125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)	1																	
Bacti $Na_2S_2O_3$ Thio 120mL																		
None (P)																		
None (AG)																		
None (CG) 500ml																		
Cr6 Buffer (P) Borate Carbonate Buffer-EPA 7196A																		
Dissolved Oxygen 300ml (P)																		
HNO_3 (P)																		
HCl (AG)																		
H_2SO_4 (P)																		
H_3PO_4 (AG) - TOC																		
NaOH (P) cyanide																		
NaOH + ZnAc (P) sodium hydroxid+zinc acetate																		
$Na_2S_2O_3$ (AG) Thio EPA 547, 538, TTHM																		
$Na_2S_2O_3$ (CG) Thio EPA , 504.1																		
$Na_2S_2O_3$ 250ml (Brown P) EPA 549																		
Thio/K Citrate(AG) EPA 531.2																		
NH_4Cl (AG) 552- ammonium chloride EPA HAA5																		
Other:																		
Client Own																		
Low Level Hg/Metals Double Bag																		
Plastic Bag	1																	
Glass Jar: 125 / 250 / 500																		
Soil Tube: Brass / Steel / Plastic																		
5g Encore																		
Cubitaner 1G / 2.5 G																		

Comments

Filter or Split

	Container	Preservative	Date/Time/Initials
S P F			
S P F			
S P F			
S P F			
S P F			

Labeled by:_____@_____ Labels checked by: fm @ 1125

APPENDIX D

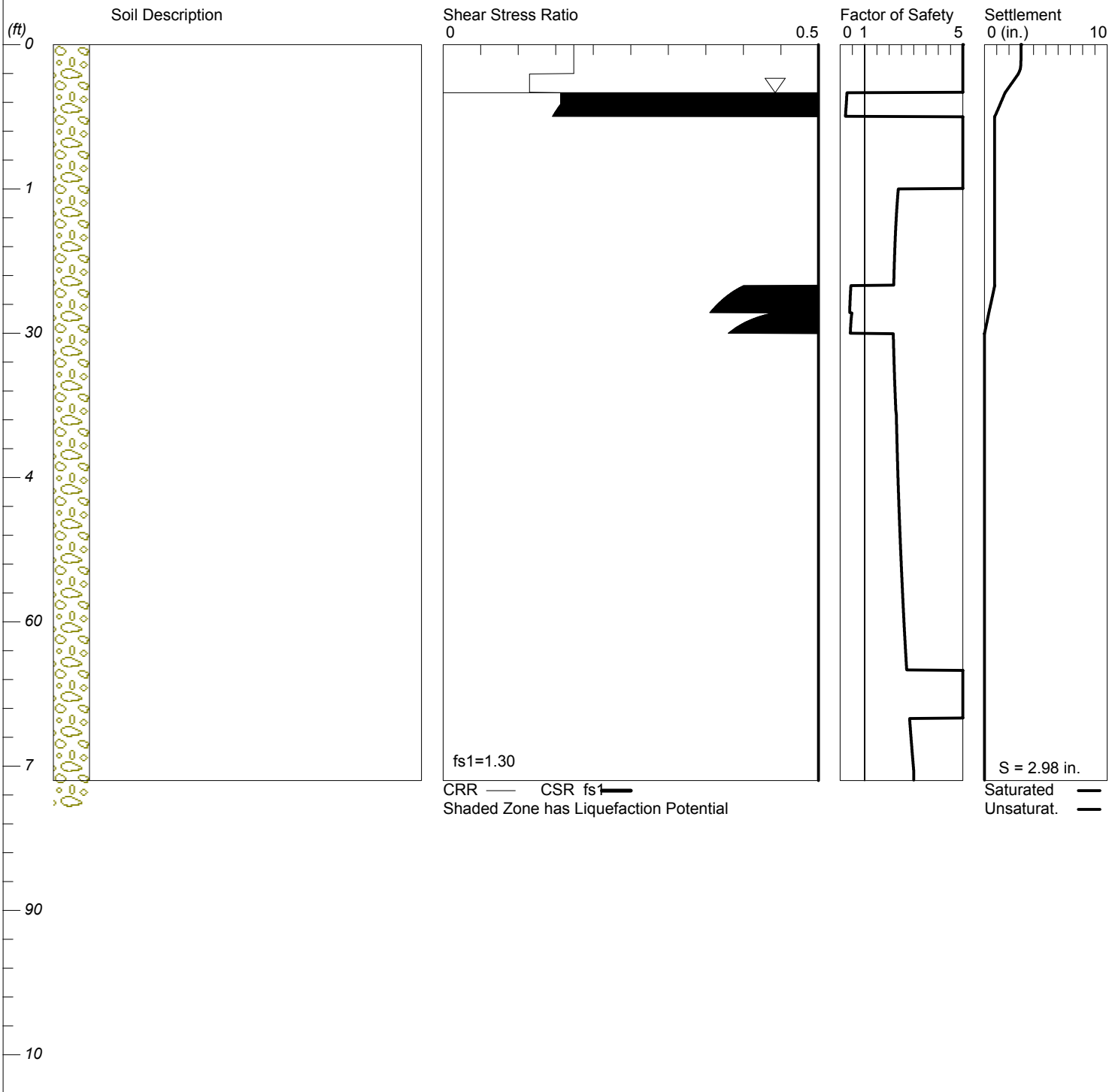
GRAPHICAL SUMMARIES OF LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

Port Hueneme - Proposed Parking Structure

Hole No.=B-1 Water Depth=5 ft

Magnitude=7.52
Acceleration=0.67g



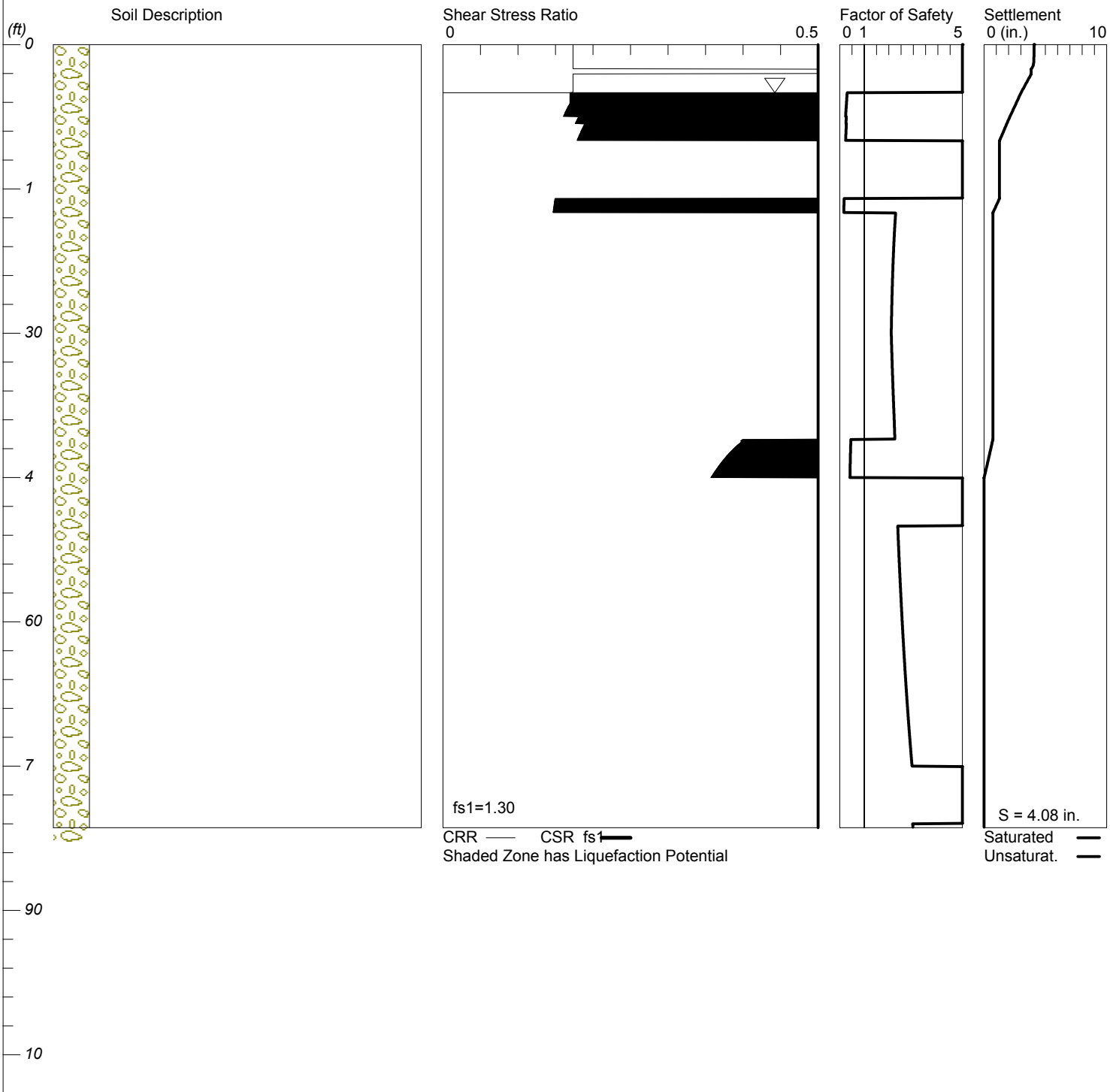
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Port Hueneme - Proposed Parking Structure

Hole No.=B-2 Water Depth=5 ft

Magnitude=7.52
Acceleration=0.67g



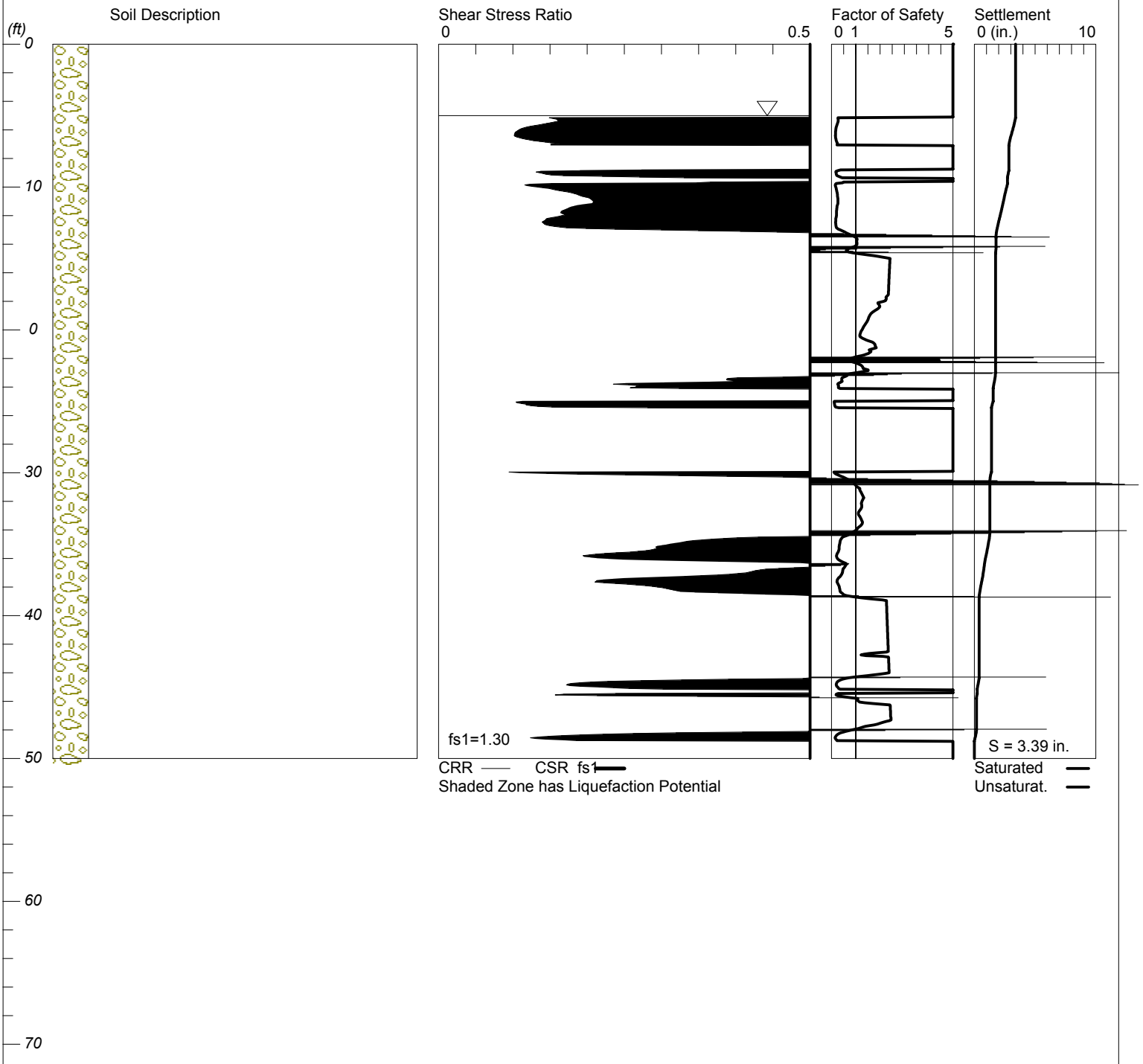
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Port Hueneme - Proposed Parking Structure

Hole No.=CPT-3 Water Depth=5 ft

Magnitude=7.52
Acceleration=0.67g



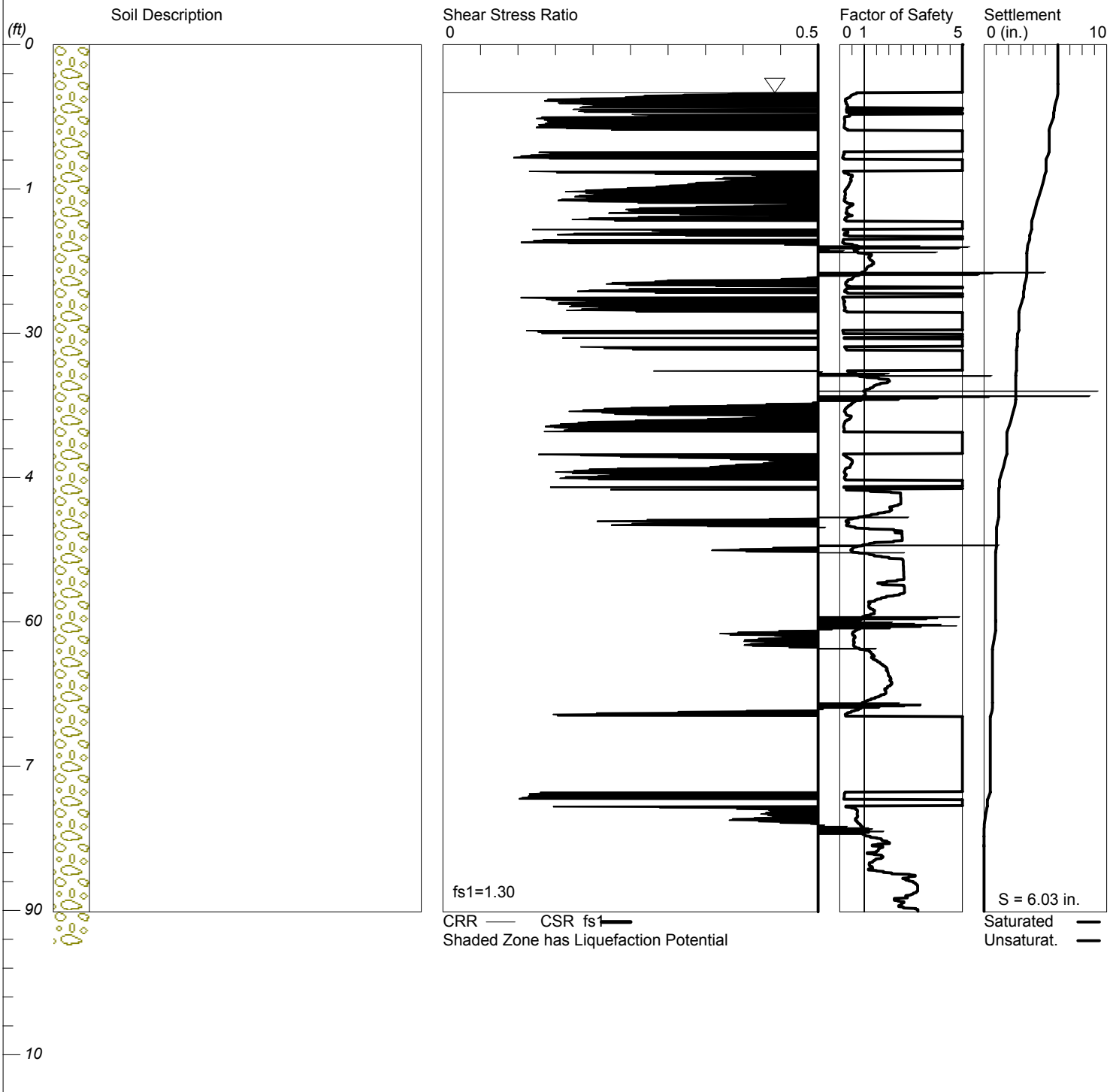
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Port Hueneme - Proposed Parking Structure

Hole No.=CPT-4 Water Depth=5 ft

Magnitude=7.52
Acceleration=0.67g



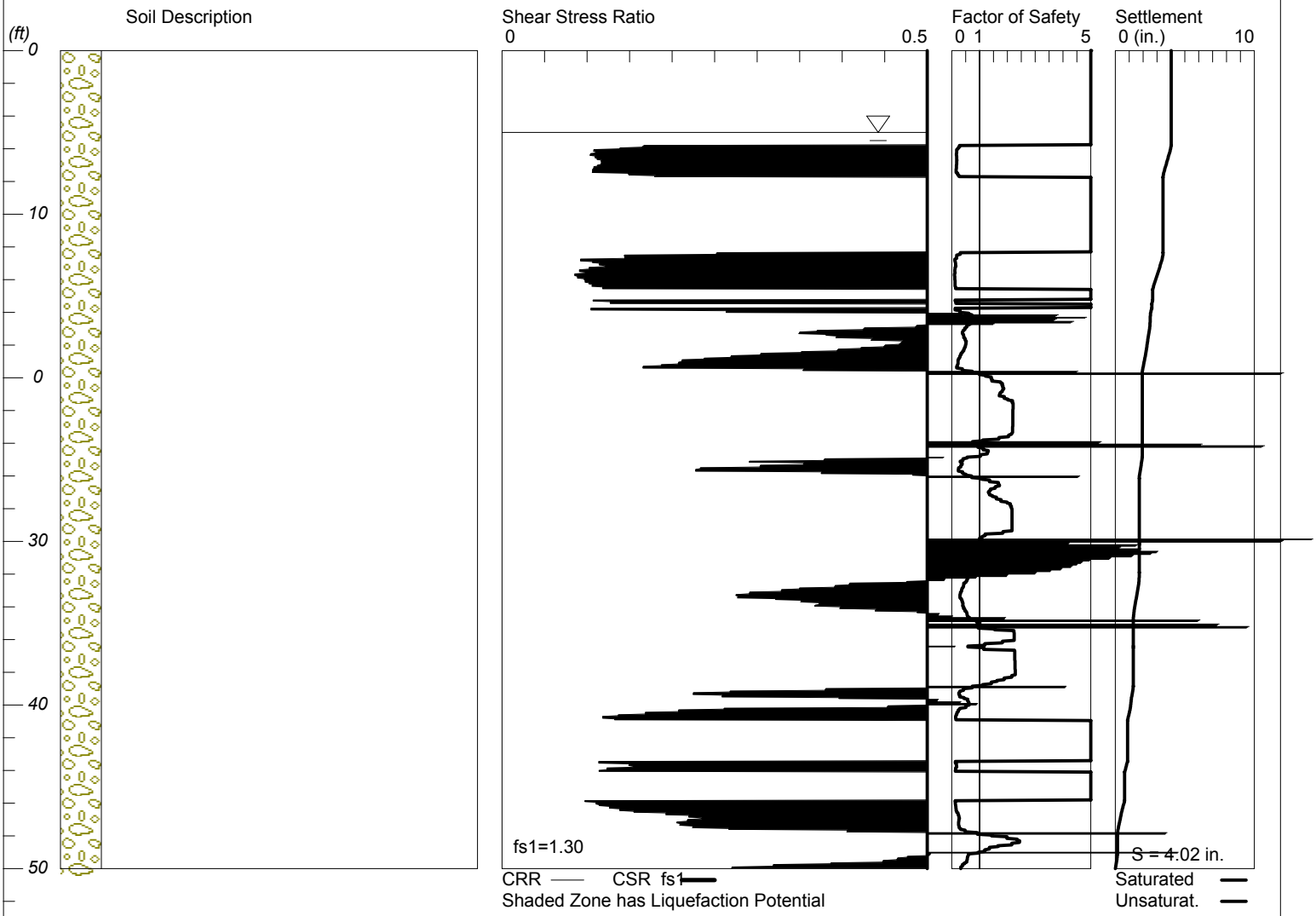
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Port Hueneme - Proposed Parking Structure

Hole No.=SCPT-5 Water Depth=5 ft

Magnitude=7.52
Acceleration=0.67g



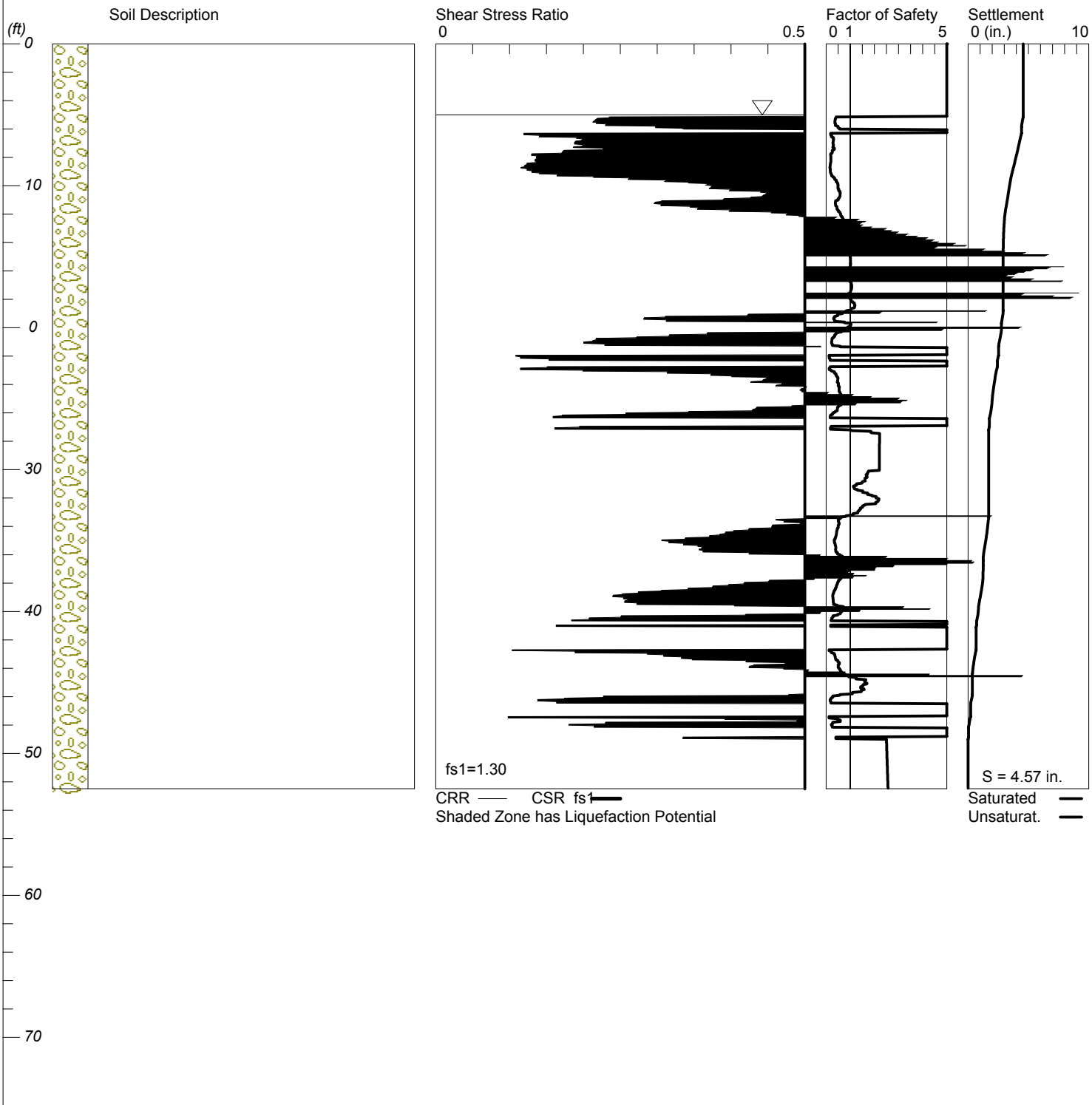
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Port Hueneme - Proposed Parking Structure

Hole No.=CPT-6 Water Depth=5 ft

Magnitude=7.52
Acceleration=0.67g



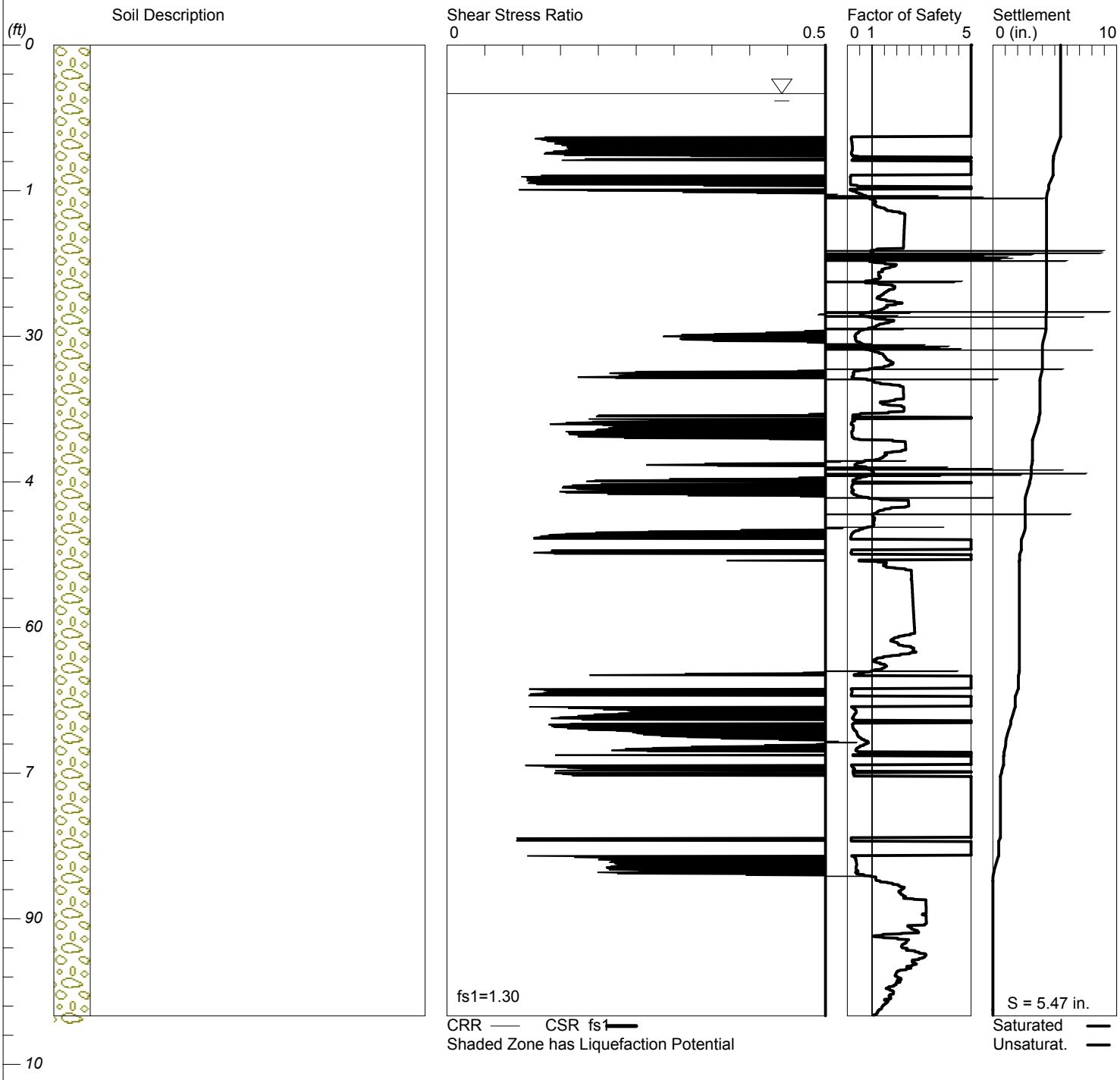
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Port Hueneme - Proposed Parking Structure

Hole No.=SCPT-7 Water Depth=5 ft

Magnitude=7.52
Acceleration=0.67g



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