Laney Library & Learning Resource Center

Commissioning Plan

October 2020

PERALTA COMMUNITY COLLEGE DISTRICT

REDCAR ANALYTICS

Contents

1. Executive Summary

The Red Car Analytics Commissioning Team has developed this Commissioning Plan to provide direction on the commissioning process and scope of commissioning, defining related activities and team responsibilities. The Plan includes definitions of commissioning terms, descriptions of commissioning processes, a comprehensive team directory, detailed information on roles and responsibilities, project and system descriptions, lists of documents related to commissioning, commissioning schedule milestones, coordination and communication protocols, and resolution pathways and procedures. It describes the scope, approach, time frame, responsibilities, and technical requirements for the commissioning activities.

The Commissioning Authority (CxA) is Red Car Analytics.

1.1. Abbreviations

A/E	Architect & Design Engineers
BAS	Building Automation System
BOD	Basis of Design
CC	Controls Contractor
CM	Construction Manager
Cx	Commissioning
CxA	Commissioning Authority (Red Car)
CxC	Commissioning/MEP Coordinator (GC)
EC	Electrical Contractor
EOR	Engineer of Record
FM	Fire Marshal
FPT	Functional Performance Test
GC	General Contractor
IC	Irrigation Contractor
IOM	Installation and Operations Manuals
IOR	Inspector of Record
MC	Mechanical Contractor
MEP	Mechanical Electrical Plumbing
O&M	Operations and Maintenance
OPR	Owner's Project Requirements
OR	Owner's Representative
PC	Project Coordinator
PM	Project Manager
SOO	Sequence of Operation
TAB	Test-Adjust-Balance

1.2. General Project Information

Project:	Laney Library and Learning Resource Center
Location:	Oakland, CA
Building Certification:	targeting LEED NC v4 Certification
Total Square Footage:	73,746 gsf
Construction Period:	Nov 2021 to Nov 2023

2. Commissioning Team Roles and Responsibilities

The Commissioning Authority (CxA) reports results, findings, and recommendations directly to the Owner. In general, the CxA shall coordinate the commissioning activities directly with the design team and the MEP coordinator (CxC), and shall distribute reports to the General Contractor, Project Manager, Architect/Engineers, and Commissioning Coordinator. The design team roles and responsibilities are identified in this Cx Plan. The contractors' commissioning responsibilities are detailed in the project specifications and this Cx Plan.

Where items in this plan differ from the specifications, the CxC shall bring these issues to the attention of the Owner's Representative and the Commissioning Authority for resolution.

2.1. Commissioning Team Directory

Name	Company	Role	Phone	email
Greg Cheifetz	Peralta Community College	Owner		
Eric Skiba	Noll & Tam	Architect	510.542.2200	eric.skiba@nollandtam.com
Gavin Ross	Noll & Tam	Architect	510.542.2260	gavin.ross@nollandtam.com
Anna de Anguera	Cavagnero	Architect	510.499.6427	annad@cavagnero.com
Angela Wisely	Brightworks	Sustainability Consultant	415.230.2136	angela.wisely@brightworks.net
Glenn Friedman	Taylor Engineering	Mechanical & Plumbing Engineering	510.220.5895	gfriedman@taylor-engineering.com
Joe Arnstein	Taylor Engineering	Mechanical & Plumbing Engineering	704.582.2081	jarnstein@taylor-engineering.com
Stefan Gracik	Alter Engineering	Energy Modeling	724.968.6938	stefan@alterengineers.com
Paul Carey	O'Mahony & Myer	Electrical Engineering	415.218.0629	pcarey@ommconsulting.com
Sean Henderson	Mantlela	Landscape/Irrigation Engineering	513.532.4685	sean@mantlela.com
Michele Sagehorn	Red Car Analytics	Commissioning Authority	707.591.4555	michele@redcaranalytics.com

2.2. General Descriptions and Responsibilities

General descriptions of the commissioning roles are as follows:

Owner or Owner's Representative (OR)

- Oversees the work of the CxA
- Assists the CxA with directing the project team
- Provides final approval and sign-off of Cx Plan
- Responsible for the development of the Owner's Project Requirements document
- Arbitrate disagreements between the CxA and others

Commissioning Authority (CxA)

- Defines the project specific scope and requirements of commissioning
- Develops and oversees the Commissioning Plan and related activities
- Assists the team with implementation of the Commissioning Plan
- Advises the Owner on acceptance of design, construction and commissioning

Architect

- Assists Owner in development of the Owner's Project Requirements document
- Coordinates/Manages submittal documents
- Provides resolution to design-related issues
- Participates as needed in commissioning process

Engineers

- Provides Basis of Design document
- Completes and signs T24 Commissioning Design Checklist Compliance forms
- Reviews submittals and provides comments
- Provides clarifications to design intent
- Provides resolution to design-related issues
- Participates as needed in commissioning process

General Contractor (GC)

- Provides a competent person in the role of MEP Coordinator (CxC)
- Provides coordination with subcontractors for all commissioning related activities
- Incorporates commissioning activities into the master construction schedule
- Directs subcontractors to provide resolution to construction issues
- Notify the commissioning agent of any change orders that may affect commissioned systems
- Coordinates owner training with subcontractors and commissioning agent
- Assists in resolving any warranty issues raised during the End-of-Warranty Review

MEP Coordinator of the GC (CxC)

- Acts as the representative of the construction team and main point of contact for the CxA
- Delegates commissioning tasks to the subcontractors
- Coordinates and manages subcontractors and commissioning schedule
- Collects, assembles and manages commissioning documentation from subcontractors
- Works with subcontractors to correct installation/commissioning deficiencies as quickly as possible
- Provides support for functional testing as needed

MEP Subcontractors (Subs)

- Reviews and comments on multiple releases of the Cx Plan
- Provides competent personnel to execute commissioning tasks
- Coordinates with GC on scheduling of commissioning tasks and potential conflicts
- Demonstrates and documents approved installation and operation of equipment
- Assists the team to provide resolution to construction issues
- Coordinates with equipment vendors for proper documentations and procedures

TAB Subcontractor

- Coordinates with the commissioning team in the weeks prior to balancing
- Completes air and water balancing, per AABC or NEBB requirements and project specifications
- Provides a field copy to CxA prior to functional testing
- Identifies and reports on issues discovered in the field while balancing
- Demonstrates TAB results to CxA during Functional Performance Tests
- Coordinates with MEP Coordinator and CxA for resolution to issues

Controls Subcontractor (CC)

- Performs all the tasks of MEP Subcontractors as listed above
- Provide Point-to-Point checks and calibration of all sensors prior to FPTs
- Provide point trends and assistance with remote access to building automation system
- Provide a person capable of demonstrating Functional Performance Test scripts and proper system operation
- Provide list of all schedules, set points, and alarms

Equipment Vendors

- Provides documentation on furnished equipment, including complete submittals, equipment data, installation manuals, O&M manuals, start-up procedures, and warranties

2.3. Project Management Protocols

The following protocols will be used on this project:

- The CxA will communicate directly to the appropriate party and inform both the OR and CxC
- Deficiencies found during testing shall be corrected by the contractor within 7 days of receiving an Issues Log from the CxA
- Resolution of minor deficits during Functional Performance testing may be permissible, as determined by the CxA at the point when the deficient are found
- Problem solving: The CxA may recommend solutions to problems, however the burden of responsibility to solve, correct and retest problems is with the contractor
- The OR shall arbitrate disagreements between the CxA and others.

Issue	Protocol
Requests for information or formal documentation requests:	The CxA goes through the architect, engineer, or CxC.
Minor or verbal information and clarifications:	The CxA communicates directly to the informed party and informs the CxC.
Notifying contractors of significant deficiencies:	The CxA documents and communicates deficiencies through the CxC and OR. The CxA may discuss deficiency issues with contractors on an informal basis and will immediately notify the CxC or GC.
Scheduling commissioning meetings:	The CxA coordinates with the CxC. The CxC will coordinate meeting attendance for all required parties and will provide advance notification to the CxA.
Making a request for significant changes:	The CxA has no authority to issue change orders or construction directives. Any actions or observations of the CxA that might result in changes to the contract documents shall be approved by the PM and the design team will issue the changes.
Making small changes in specified Sequences of Operation (SOO)	The CxA notifies A/E of suggested change, which, if approved, is then implemented via RFI. Implemented changes in the SOO shall be documented by the CC in the as-built records, will become part of the O&M/ Systems Manuals, and the CC will notify the CxC and OR of the change in writing.
Making small changes to correct deficiencies	The CxA may request small changes to correct deficiencies from the responsible contractor. The CxA will immediately inform the GC's CxC of any such request. The respective contractor will be required to notify the CxC of the deficiency and correction.
Subcontractors disagreeing with requests or interpretations by the CxA	The OR will arbitrate disagreements between the CxA and others.

2.4. Project Management Protocols

The CxA will use a platform to store and share the commissioning documents, so that they can be viewed/downloaded at any time by all commissioning team members. The platform used will be either Microsoft sharepoint or box, where specific folders will be organized to make it easy to find the essential documents (e.g. – current version cx plan, etc.).

Red Car Analytics uses a cloud based program to document and maintain the commissioning issue logs (design/construction). The use of this program is simple and requires no password or special software, only the participants' email. The issue log(s) are essentially an excel spreadsheets, but allows parties to collaborate and to always have access to the latest update. The software also allows attachments to be stored and tracked with the associated issue. The use of this software allows for more detail and history that may prove useful to the owner or operator of the building.

3. Systems to be Commissioned

The following systems and equipment will be commissioned in this project. All general references to equipment in this document refer only to systems and equipment that will be commissioned.

Mechanical

- HVAC System & Controls
- Air Handling Unit(s)
- Hydronic Pump(s)
- VAV Terminal Unit(s)
- Building fan(s)
- Building Management System/Controls

Plumbing

- Domestic HW System

Electrical/Lighting

- Lighting and Controls

Miscellaneous

- Irrigation/Irrigation Controls

4. Commissioning Process – Design Phase

The general sequence of Cx tasks and timing within the design schedule is shown below. The Architect and/or General Contractor are responsible to include commissioning tasks into the design and construction schedules and organize the participants.

Commissioning Schedule	Schematic Design	Design Development	Construction Documents	Bidding and Contract Award
Cx Review/Kickoff Meeting				
Verify Owner Project				
Basis of Design				
Develop Cx Plan				
LEED Design Review				
Title 24 CXR-E Design Review				
Cx Specifications				
Controls Integration Meeting				
Development of Training Requirements				
Development of System Manual Content				

MEP Cx

4.1. Title 24 Cx Schematic Design Kickoff Meeting

Title 24-2013 requires a meeting in early Schematic Design between the owner, Cx Design Reviewer, and the design team. The intent of the meeting is to lay out the process and timing for the design review(s) and provide the team with the Title 24 forms. It is recommended that the facility manager and/or facility mechanics participate in the design phase.

4.2. Owner's Project Requirement

Identifying and documenting the project's design intent, or Owners Project Requirements, provides the design and construction teams with an understanding of the design goals for the project. The OPR is used to evaluate the design and construction efforts, assuring the goals are met, as well as helping to develop Functional Performance Tests. The OPR is developed by the CxA with guidance from the owner/owner representative at the beginning of the project. It is a living document and should be updated throughout the project. The CxA reviews the OPR for clarity and completeness.

The OPR should be documented during programming or schematic design, including:

- Energy efficiency goals
- Ventilation requirements
- Project program, including facility functions and hours of operation, and need for afterhours operations
- Equipment and systems expectations

4.3. Basis of Design

The Basis of Design documents the thought processes and assumptions behind design decisions and is meant to show compliance with the OPR. The Basis of Design includes general information about the project, as well as specific technical design information about the proposed equipment and systems. This includes HVAC load calculations, overview of the sequence of operations, assumptions about temperature, light, hours of occupancy, etc. Plumbing and electrical BOD are developed to indicate how the domestic hot water and lighting controls are integrated into the building as well. This document may also be used during functional testing to confirm the design strategy and to help troubleshoot issues that may arise.

4.4. Commissioning Plan

The Commissioning Plan (Cx Plan) is the roadmap for all activities related to Title 24 2013 and LEED commissioning. Commissioning begins during early design and continues through construction and into the post-occupancy period; therefore this document is intended to provide requirements for both design and construction teams. A preliminary Cx Plan is developed during the early design phase and updated by the CxA as needed throughout the project.

4.5. Commissioning Design Review

A review of the project's design drawings and specifications is performed as a part of the commissioning requirements prior to the completion of Construction Documents. An initial review is initiated at mid-construction documents, with a back-check in subsequent issues. These reviews are conducted by the CxA, as a consultant to the owner, but not as the Engineer of Record. Issues related to the design must ultimately be resolved by the Engineer of Record.

The review focuses on the functionality of the systems, completeness and coordination of the drawings and specifications, maintainability of the systems, and overall compliance with the OPR and BOD. A Design Issue Log is maintained by the CxA to track all issues, responses, and actions related to commissioning. A/E members shall provide a written response to items found in the Design Issues Log.

4.6. Title 24 Code Design Review

There is a compliance form that are required by code to be submitted to the building department when filing for a building permit. The initial in-person design review meeting is to be held during the schematic phase of design. The owner, design team representatives (including the project architect and mechanical and electrical design engineers), and Design Reviewer meet to discuss the following:

- Project coordination, including involvement with the Design Reviewer
- Project scheduling, including design review
- Project scope
- Owner's Project Requirements
- Basis of Design
- Design elements and assumptions
- HVAC system selection
- Construction Documents Design Review checklists
- Energy efficiency measures

At the 90% CD phase, drawings and specifications are provided to the Design Reviewer, who reviews the construction documents using form NRCC-CXR-E. The Design Reviewer reviews the construction documents for clarity, completeness, and adherence to the owner's goals.

The NRCC-CXR-E acknowledges the design review has been executed, with signatures from the responsible engineer(s), and Design Reviewer. The form is provided to the Architect and placed on drawings to be submitted with the application for the building permit.

4.7. Commissioning Specifications

Commissioning specification language is incorporated into the construction documents. Section 019113 in Division 1 is dedicated to the General Commissioning Requirements. This language communicates commissioning roles and responsibilities to the construction team including the following:

- Components and systems that are commissioned
- Parties involved and their respective responsibilities for the commissioning process
- Commissioning schedule management
- Issue and non-compliance management
- Submittal review requirements and approval
- Definition of terms
- Scope and rigor of the start-up process, including responsibility for developing and executing startup checklists, and for the approval of these documents
- Scope and rigor of Functional Performance Testing, including responsibility for writing, executing, witnessing, and signing-off the tests
- Operations and Maintenance documentation requirements
- Training requirements for facility staff and building users (or Owner's representatives?)
- System manual requirements

4.8. Development of the System Manual

The CxA shall develop and outline the requirements of the Systems Manual with input from the Owner. The Systems Manual provides future operating staff with the information needed to understand and optimally operate the commissioned systems. It provides Facilities staff with the information necessary to monitor, maintain, and optimize system operations on an ongoing basis, and aids in the long-term success of building operations' energy efficiency strategies according to the design intent. The Systems Manual will be updated after Functional Performance Tests are complete, and within ten (10) working days of receiving as-built BAS drawings, Sequences of Operation, and other documentation from the CxC.

The Systems Manual will include the following documentation submitted by the CxC:

1) System Single Line Diagrams
2) As-built Sequences of Operations, control drawings, and as-built setpoints
3) Operating instructions for integrated building systems
4) Recommended schedule of maintenance requirements and frequency, if not included in the project O&M manuals
5) Recommended schedule for calibrating sensors and actuators
6) Basic Operation:
7) Written narrative of equipment operation
8) Interfaces, interlocks and interaction with other equipment and systems

4.9. Development of Training Requirements

The CxA shall review the training requirements and confirm that they meet the scope of the owner's requirements for the operator training. The training requirements must be completed before the bid documents are finalized, and be incorporated into the commissioning specifications as part of the bid package.

The training requirements shall include the following:

- List of those who should receive operational training, by position and name
- List of systems that require operator training
- Level of instruction required for each system
- Determination of whether the raining provided by the equipment manufacturer is acceptable
- Tracking method to ensure that all required positions or persons receive training

4.10. Controls Integration

They are essentially a series of meetings that go over the project's control issues to enhance an understanding of the full sequences of control and interactions. When conducted during design, the meeting will reduce change orders and Requests for Information (RFIs). When conducted during design and again during submittal review in construction, the meetings will shorten the time required to program and reprogram the controls, perform testing and troubleshooting, and will enhance building operation and control for facility staff.

In design phase the meeting(s) include at minimum; the MEP engineers, controls contractor (if available), CxA, and owner representative. Discussion will include integration and interoperability issues between equipment, systems and disciplines to ensure that integration issues and responsibilities are clearly described in the specifications. The controls design as well as the sequence of operations for each system will be reviewed and clarified for all parties.

5. Commissioning Process – Construction Phase

The general sequence of Cx tasks and timing within the construction schedule is shown below.

Commissioning Schedule	Early Construction Phase	Installation	Start-up	Project Close-out	Post-Occupancy
Construction Team Kickoff Meeting					
Submittal Review					
Controls Integration Meeting					
Installation Verification Checklists or Mockup Performance Test Witness					
Equipment Start-up (Verification) or Witness Installation					
Test and Balancing (Verification)					
Functional Testing					
Owner Training / Verification					
Cx Report					
Systems Manual					
Post-Occupancy / Warranty Review					

MEP Cx

5.1. Commissioning Kick-Off Meeting

A commissioning kick-off meeting shall be conducted by the CxA. In attendance shall be the respective representatives of the OR, GC, CC, PC, MC, EC, IC, TAB, and all subs installing equipment to be commissioned. At the meeting, the commissioning process will be reviewed, and management and reporting lines will be determined. The flow of documents will be reviewed, and process questions will be addressed. The general list of each party's responsibilities, including the development of the Installation Verification and Startup Checklists and Functional Performance Tests for each piece of equipment, deliverables, proposed commissioning schedule, training and close-out will be presented and finalized.

The intent of the meeting is to provide an understanding with all parties as to the project's commissioning process and their respective responsibilities. The CxA will develop meeting minutes and a list of action items to be distributed to all key participants.

5.2. Commissioning Schedule

The CxA shall develop an initial commissioning schedule. The CxC shall incorporate the Cx schedule into the overall construction schedule. The Cx schedule shall be updated and refined by the CxA as construction progresses.

The following sequential priorities will be followed in the development of the commissioning schedule:

1. Equipment is not temporarily started (for heating or cooling), until Installation Verification Checklist and Start-up checklist items and all manufacturers' pre-start procedures are completed, and moisture, dust and other environmental and building integrity issues have been addressed.
2. TAB is not performed until the envelope is completely enclosed and ceiling complete, unless the air is ducted.
3. The controls system and equipment it controls are not functionally tested until all points have been calibrated and Installation Verification Checklists have been completed.
4. Functional testing does not begin until Installation Verification Checklists, Start-up and TAB have been completed for a given system. This does not preclude a phased approach.

5.3. Submittals and Documentation

5.3.1. Submittals and Sequence of Approvals

Description	Responsible Party	Delivery
HVAC, DHW, Lighting Controls, Electrical, Renewable Energy Systems, Irrigation Systems		
Equipment Submittal	Subcontractors	Prior to ordering equipment
Manufacturer's Installation and Operations Manuals	Subcontractors	8 weeks prior to startup
Single Line Diagram - showing equipment configuration	A/E or Subcontractor	8 weeks prior to startup
Sequence of Operations	A/E	Prior to construction (100% CD)
Startup & Installation verification Checklist - Draft Submittal	Subcontractors	8 weeks prior to startup
Procedure for piping, flushing and ductwork testing	MEP Sub	8 weeks prior to startup
Completed Startup & Installation Verification Checklists	Subcontractors	After Startup Checklist approval
Draft Functional Test Scripts for Team review	CxA	4 weeks prior to FPTs
Completed Controls Point-to-Point Checklist	BAS Sub	After Start-ups
Vibration/Sound Control Devices	MEP/TAB Sub	Provide report to CxA
Completed Piping, Flushing, Ductwork documentation	MEP Sub	After approval of procedures
Test and Balance Report-including vibration/sound requirements	TAB Sub	After approval of Completed Startup Checklists
Title 24 Acceptance Tests	Subcontractors	Prior to Functional Performance Tests
Functional Performance Tests (FPT)	Team	After approval of TAB Report
Response to Issues Logs	Team	As Issues are remedied
Training Agenda Submittal	Subcontractors	2 weeks prior to training
Training of Staff & Maintenance	Subcontractors	After Training Agenda approval
Training Log	GC	After Training is complete

5.4. Equipment Submittals

Equipment submittals are provided to the CxA at the same time as the EOR is reviewing the documents. The CxA will send all comments to the EOR several days prior to the architect's deadline. The EOR will review and merge the comments as necessary. It is the responsibility of the architect to incorporate all party's comments prior to returning the equipment submittal to the subcontractor. CxA review is based on adherence to contract documents, though may also have a focus on maintainability, access, and other functional aspects of the equipment.

5.5. Contractor Documentation

Once the equipment submittals are approved, each MEP subcontractor shall submit additional Contractor Documentation that is used in the commissioning process. These documents are to be provided as a package within 4 weeks of the approved submittals. Included in this package are the following:

5.5.1. Manufacturer's Installation and Operations Manual (IOM)

This manual generally can be downloaded from the manufacturer's website. The document may or may not be the same as the Operations and Maintenance Manual. It includes the recommended steps and procedures for proper installation, calibration, and configuration of the equipment. Include project Tag Numbers on all documents. Identify options that are being provided and/or cross out sections that are not appropriate to this project's installation.

5.5.2. Single Line Diagrams (SLDs)

Single line diagrams show equipment connections to integrated systems.

5.5.3. Sequence of Operations (SOO)

The contract documents generally contain the SOO, developed by the EOR. If this is not included in the drawing or specifications, the controls contractor (or subcontractor) submits the SOO to the EOR for review and approval.

Additionally, if there is a piece of equipment that is stand-alone or using it's own internal controls; a sequence of operation is required from the manufacturer for that specific piece of equipment.

Other systems may finalize their sequence of operation during the submittal process. Such systems might include; Emergency Electrical/Security Protocols, Lighting systems, Automatic Shades, etc.

The SOO is a narrative describing the equipment and systems' startup, shutdown, capacity modulation, emergency and failure modes, alarms, and interlocks to other equipment. The CxA shall review and comment on the approved SOO and request clarifications and/or suggestions for all commissioned systems.

5.5.4. Installation Verification Checklists

These are the checklists that incorporate steps to install, configure, and calibrate the components of the equipment. Included are steps found in the manufacturer's recommendations, contract document requirements, and the contractor's standard procedures. These are sometimes references as Pre-Start, pre-checks, or pre-functional checks and are generally performed prior to the equipment being operated. They may be combined with the Start-up Checklists as long as all the procedures are incorporated.

5.5.5. Startup Checklists

Startup Checklists are designed to verify the equipment is configured and adjusted to comply with the manufacturer requirements and contract specifications. These steps include procedures involving the equipment's operational functions. Typically, pressure and temperature adjustments and verifications, modification of settings and schedules, and confirmation of functional operations are all part of the Startup Checklist. The contractor's standard Startup Checklist can be incorporated into these other requirements. If vendors are providing the startup, a copy of their checklist is required for review and acceptance. Ultimately, the subcontractor is responsible for the Startup procedures to be complete.

5.5.6. TAB Outline Plan

The CxC shall submit the outline of the TAB plan and approach prepared by the TAB contractor to the CxA and the BAS contractor at the same time when other submittals and contractor documentation is being processed. A full description of the procedures and the equipment to be verified, along with the design values, shall be provided.

A written explanation of the intended use and specific requirements of the BAS for the successful and timely completion of the TAB should be included in the plan. The EOR shall review and approve the plan. The CxA shall review the proposed plan for understanding and coordination issues and may provide comment, but is not responsible for approving the TAB plan.

5.6. CxA Reviews

The CxC shall submit equipment and contractor documentation provided by the manufacturer and developed by the installing contractor for review by the CxA. The CxA shall suggest additional startup procedures to be incorporated, based on contract documents and manufacturers recommendations.

5.7. Special Submittals and Notifications

5.7.1. Changes to Previous Submittals

The Subs, GC or A/E shall notify the CxA of any new design intent or operating parameter changes, modified control strategies or sequence of operation, or other change orders that may affect commissioned systems.

5.7.2. Controls Points List

The MEP EOR shall provide the CxA with a project specific full points list, at least thirty (30) days prior to performing the functional tests.

5.8. Controls Integration Meetings

The CxA, CxC, EOR, CC and OR (or Owner's designated Facility Representative) will conduct controls integration meeting(s) in coordination with team members as appropriate, including the controls programmer for the project. The meetings shall occur after the software and database drawings are issued for initial review, but prior to the development of the database and code for any piece of equipment. The meetings shall discuss and clarify the following issues:

- Points database
- Sequence of Operation, setpoints, and schedules
- Functional interlocks
- Operator workstation graphics
- Field sensor and panel location
- Integration with other systems

5.9. Miscellaneous Meetings

The CxA may attend selected planning and job-site meetings in order to remain informed on construction progress and to update parties involved in commissioning. The CxC shall provide the CxA with information regarding substitutions, RFIs, change orders and any Architect's Supplemental Instructions (ASI) that may affect commissioning of equipment, systems or the commissioning schedule.

During construction, meetings between various commissioning team parties will be scheduled by the CxA through the CxC.

5.9.1. Site Observations

The CxA shall make periodic visits to the site to observe equipment and system installations. Additional visits may be made to observe the contractor's pre-functional testing and verification of installations. The CxA shall be given adequate notice (no less than seven working days) by the CxC.

5.10. Startup

Installing Subs are responsible for each part of Installation Verification Checklists and Startup Checklists for commissioned equipment and systems. The parties responsible for each part of these checklists are identified on the checklists. The startup procedures are directed and executed by the Sub or equipment vendor.

The Subs shall provide to the CxC the manufacturer checklists, Installation Verification, and Startup Checklists, including actual field checkout sheets used by the field technicians. The CxC shall forward the documents to the CxA. These documents shall become part of the Commissioning Final Report.

5.10.1. Execution of Startup Checklists and Startup

The Installation and Startup Checklists are directed and executed by the Sub or vendor. To document the process of startup, the site technician performing the tasks shall check off items on the checklists as they are completed. Only individuals having direct knowledge of a line item being completed shall check or initial the forms.

The Subs and/or vendors execute the checklists and submit a signed and dated copy of the completed Installation and Startup Checklists to the CxC, who shall forward it to the CxA. The CxA may review Startup Checklists in progress.

5.10.2. Deficiencies and Non-Conformance

The Subs shall clearly list any outstanding items from the startup procedures that were not successfully completed. The procedures form and deficiencies shall be provided to the CxA within two (2) days of test completion. The Subs and vendors shall correct and retest any deficiencies or uncompleted items, involving the CxC as necessary, prior to the start of functional performance testing. The CxC shall notify the CxA when all deficiencies and uncompleted items have been resolved.

5.10.3. Electrical Systems Checkout Plan

The Electrical contractor shall use the approved Electrical Checkout plan; which will include MOPs (Methods of Procedure) and required testing forms as their start-up verification. The Electrical contractor shall review the FPTs in advance when provided by the CxA and provide written comments and detailed amendments no less than fourteen (14) days prior to the start of functional testing. The CxA shall consider the written comments in finalizing the FPTs within seven (7) days of receipt.

5.10.4. Controls and Checkout Plan

The BAS contractor shall utilize the BAS Point-to-Point and calibration checks as their startup verification.

All controls-related Startup Checklists and verifications must be completed and accepted by the CxA prior to TAB. The BAS contractor shall execute the assigned tests and trend logs and be available for assistance for mechanical system Functional Performance Tests.

5.10.5. Startup Document Review

The Subs shall provide the CxC with copies of the completed Installation Verification Checklists, Startup Checklists, and manufacturer's startup forms for all commissioned equipment and systems. The CxC shall forward the completed Cx documentation to the CxA as it is submitted, and in all cases, a minimum of five (5) working days prior to the start of Test Adjust Balance.

5.11. Test and Balance (TAB)

The final TAB plan (as refined from the TAB outline plan) shall be provided to the CxA for review at least thirty (30) days prior to the commencement of TAB work. The TAB report shall be provided to the CxA upon completion, no later than five (5) days following completion of TAB work and 10 days prior to functional testing.

The TAB contractor shall submit weekly written lists of completed tests and reports of discrepancies to the CxC. The CxC shall forward to the CxA.

In general, TAB work does not begin until the BAS control system has been verified by Point-to-Point and calibration checks and these documents are accepted by the CxA.

The BAS contractor shall also meet with the TAB contractor prior to the start of TAB and review the TAB plan to determine the capabilities of the control system for use in TAB. The BAS contractor shall provide the TAB contractor with any necessary instruments for configuring terminal unit boxes and instruct the TAB contractor in their use. The BAS contractor shall also provide a technician qualified to operate the controls to assist the TAB contractor in performing TAB activities, as needed.

5.12. Functional Performance Test (FPT) and Verification Procedures

Functional testing is the dynamic testing of systems (rather than just components) under full operation. Systems are tested under all modes of operation as defined in the sequences of operations, such as during cooling and heating loads, low and high loads, component failures, occupied and unoccupied modes, varying outside air temperatures, emergency and power failure, alarms, interlocks, and other operating conditions. Testing proceeds from components- to subsystems- to systems- and finally to interlocks and connections between systems.

Functional testing and verification shall be achieved by manual testing, by monitoring the performance and analyzing the results using the BAS trend log capabilities, or by stand-alone data-loggers, depending on the equipment and sequence as referenced in the FPTs. The systems shall be run through all of the control system's sequences of operation and verified to be responding as stated.

5.12.1. Development of FPTs

The CxA shall develop the Functional Performance Test procedures in a sequential written form. The CxA reviews all equipment submittals, manufacturer's recommended tests, and any change orders affecting equipment or systems, updated points list, control sequences and setpoints. The CxA may require clarification from the CxC, Subs and the A/E regarding sequences and operation for this purpose. The CxA shall utilize this data in preparing test forms and sequential test procedures to verify proper operation of each piece of equipment and system. Tests are prepared to yield results that are predictable and repeatable.

FPT procedures shall be distributed for review by all parties in advance of the scheduled tests. FPTS shall be submitted to the A/E for approval and finalized by the Commissioning Authority prior to the start of testing.

The BAS contractor shall review the FPTs in advance when provided by the CxA and provide written comments and detailed amendments no less than fourteen (14) days prior to the start of functional testing. The CxA shall consider the written comments in finalizing the FPTs within seven (7) days of receipt.

5.12.2. Contractor Review and Approval of FPTs

Thirty (30) days prior to performing any functional tests, proposed FPTs will be provided to the subcontractors. The Subs shall review and provide the CxA with their review of any procedures that are inconsistent with their understanding of the equipment operations, or conditions that may be unsafe or incompatible with maintaining the warranty.

5.12.3. Functional Test Prerequisites

Prior to the initiation of Functional Performance Tests, the CxA shall verify that Startup Checklists have been completed. BAS installation must be substantially completed (automatic operation) and their conformance to test requirements must be documented and accepted by the CxA before commencing functional testing of systems. TAB for air and hydronic systems must be completed and at a minimum, the preliminary reports from the field submitted to the CxA.

5.12.4. Title 24 Acceptance Tests

Contractors are to submit a copy of all Title 24 Acceptance Tests to CxC, who will forward to CxA prior to the start of Functional Performance Tests. These tests are a further verification, beyond the other startup procedures, that the equipment is properly configured and ready for testing.

5.12.5. Execution of Functional Testing Procedures

5.12.5.1. Process

The CxA shall schedule Functional Performance Tests through the CxC. A meeting shall be held by the CxC at the start of the FPTs to review the planned tests and assure the required parties are prepared.

The CxA shall oversee, witness, and document the Sub's demonstration of the Functional Performance Test procedures for all equipment and systems according to the Specifications and the Cx Plan. The Subs or manufacturer's representatives shall execute and demonstrate the tests following the procedures developed by the CxA.

5.12.5.2. Preliminary Testing

The controls contractor, subcontractor, or vendor shall run through all functional testing prior to demonstrating the functional tests to the CxA. This is done to ensure that all readily observable deficiencies are corrected prior to witness testing. If, during functional performance testing, it becomes clear that preliminary testing by the Sub was not performed or not completed effectively, testing may be postponed at the discretion of the CxA. Testing shall recommence or resume after completion of preliminary testing.

5.12.5.3. Deficiencies and Re-Testing

In the process of witnessing Functional Performance Tests, the CxA will document non-compliant tests and significant system deficiencies on the procedure or test form. The CxA will notify the PC and CxC of deficiencies or non-conformance issues by documenting them on the Commissioning Issues Log in a timely fashion. Corrections of minor deficiencies may be made during the test demonstrations at the discretion of the CxC and CxA.

Decisions regarding minor deficiencies and corrections will be made at as low a level as possible; e.g., from the CxA, CxC and the Sub. The CxC and Subs will schedule re-testing as required, providing written notification to the CxA. For disputed items, the Owner will be the final authority.

Any additional retesting and site visits necessitated by equipment or systems not being fully functional when otherwise stated by the responsible contractor shall be at the expense of the responsible contractor.

Any additional verification or back-checks of issues due to equipment or systems not being functional when otherwise signed off by the responsible contractor shall be at the expense of the responsible contractor.

5.12.5.4. Issues Log

The CxA shall document all deficits found throughout commissioning in an Issues Log that is updated, distributed, and sent to the commissioning team. This document represents the observations by the CxA, as a third party representative of the owner. Subs shall correct deficiencies, sign them off, and notify the CxA in writing when ready for re-testing. It is the responsibility of each responsible party to address each issue by responding in writing as to what remedy or response the contractor has to the identified item. The CxA shall schedule re-testing through the GC.

5.12.5.5. Facilities Staff Participation

Facilities Operations staff is encouraged to attend and participate in the testing process. This process neither constitutes nor replaces formal training.

5.12.6. Execution of Functional Testing Procedures

Prior to functional testing, the GC's Controls Contractor (CC) shall set up trends on the BAS as specified in the FPTs and/or contract documents. The CC shall download and submit trend data to the GC, who shall forward it to the CxA for review. The data must be electronic and in spreadsheet or database format.

Trending Requirements:

- Trend logs must be established before functional testing; Commissioning Agent to review 24 hours of trend data prior to functional testing
- Remote access must be established prior to functional testing and the CxA shall be given access
- Submit trends for all points listed in specifications. Equipment should be trended during a period similar to design conditions
- Trend data must be saved in CSV (Comma delimited) (*.csv) format or database format
- All data is to be within the 14 day trend period for any particular submittal period
- Status or Change of Value (COV) data may be saved with other COV data in a single file, but not with Time Series data
- Provide 14 continuous days of data, 24 hours a day, with time intervals as specified in Functional Performance Tests

5.13. O&M Manuals and Warranty Documentation

The CxC shall assemble all the turn-over documents required by the contract documents; this may include O&M manuals, equipment warranties, contractor guarantees, and as-builts. Typically these documents are reviewed and accepted by the A/E team. Additionally, the CxC should furnish these documents to the Operations Staff for internal review and input, and as a precursor to training. The CxC shall also review the turn-over package for completeness, approve each equipment warranty and will verify that all requirements for maintaining the warranty are valid and clearly stated. The CxA shall be provided the final turn-over package and will verify that it complies with the contract requirements.

5.14. Final Commissioning Report

A final summary report by the CxA will be provided to the OR. The report shall include an executive summary, list of participants and roles, overview of commissioning and testing scope, and a general description of testing and verification methods and results. All outstanding non-compliance items shall be specifically listed. Recommendations for improvement to equipment, system interactions or operations, future actions, commissioning process changes, etc. may also be listed.

5.15. Systems Manual

The CxA shall develop the Systems Manual with input from the CxC, Subs, OR, CC, TAB, and Operations staff. The Systems Manual provides future operating staff with the information needed to understand and optimally operate the commissioned systems. It provides Facilities staff with the information necessary to monitor, maintain, and optimize system operations on an ongoing basis, and aids in the long-term success of building operations' energy efficiency strategies according to the design intent. The Systems Manual will be developed after Functional Performance Tests are complete, and within ten (10) working days of receiving as-built BAS drawings, Sequences of Operation, and other documentation from the CxC.

The Systems Manual will include the following documentation submitted by the CxC:

1. System Single Line Diagrams
2. As-built Sequences of Operations, control drawings, and as-built setpoints
3. Operating instructions for integrated building systems
4. Recommended schedule of maintenance requirements and frequency, if not included in the project O&M manuals
5. Recommended schedule for calibrating sensors and actuators if not included in O&M manuals.
6. Basic Operation:
    a. Written narrative of equipment operation
    b. Interfaces, interlocks and interaction with other equipment and systems

5.16. Training and Orientation of Personnel

Owner training and orientation on equipment and systems shall be provided by the GC and Subs in accordance with the contract documents. The GC and Subs shall submit training plans to the CxA at least 30 days prior to scheduling training. Training plans shall include the name and qualifications of the trainer, the targeted audience, and a list of topics to be covered. The CxA shall review the training for system overview, design intent, and design criteria.

The operations personnel should be trained on the safe and proper operation, maintenance, diagnosis and repair of each piece of equipment. Submitted O&M information should be used during these training activities.

5.16.1. Training Sessions / Agenda

For each piece of equipment or system, a written training agenda will be developed and submitted for review and approval. OR and CxA will review and comment on each agenda in accordance with the training plan and project specifications. Recommended topics include:

1. Systems and equipment conceptual overview – what is the equipment, what is its function, and with what other systems or equipment does it interface
2. Sequences of operation in all modes of operation
3. Review of information in the System Manual
4. Review of the pertinent record drawings
5. Warranty details
6. Relevant health and safety practices and concerns
7. Common problems and their diagnosis and/or repair
8. Review and demonstration of servicing and preventive maintenance
9. Hands-on training for Facility staff
10. Proper maintenance schedules, tasks and procedures with demonstrations
11. Emergency response and recovery procedures in accordance with 2012 NFPA 3, Section A.1.3.2(5)
12. Emergency response and recovery procedures in accordance with 2012 NFPA 3, Section A.1.3.2(5)

5.16.2. Training Record

Following the training, the CxC and the Contractor will prepare a Training Record which will include, for each piece of equipment, a check-off of training covering each topic per the Project Training Plan. A log with the trainer and attendees signatures and date shall be included. PM, CxA, and the A/E will review the final Training Report.

5.17. Current Facility Requirements

New to LEED version 4 is the requirement for the CxA to assemble and maintain a current facility requirements (CFR) and operations and maintenance (O&M) plan that contains the necessary information to operate the building efficiently. The minimum documentation for the plan are listed below:

- Sequences of operation for the building
- Building occupancy schedule
- Equipment run-time schedule
- Setpoints for all HVAC equipment
- Lighting levels throughout the building
- Minimum outside air requirements
- Changes in schedules or setpoints for different seasons, days of the week, and times of day
- Systems narrative describing the mechanical and electrical systems and equipment
- Preventive maintenance plan for building equipment described in the systems narrative
- Cx program that includes periodic Cx requirements, ongoing Cx tasks, and continuous tasks for critical facilities.

5.18. Warranty Period Commissioning

The CxA will conduct a Post-Occupancy and Warranty Period Commissioning Review. Post-Occupancy Commissioning will include a site visit approximately 8 – 10 months after occupancy, but before the warranty period has expired. During this site visit the CxA will interview the owner's representative and/or facility staff to identify any problems with commissioned equipment, or concerns they have with operating the building as originally intended. Any unresolved items on the Issues Log will be reviewed. The CxA will participate in forming a plan with the owner to have these items resolved. The CxA may witness deferred testing. The CxA will document their findings in a site visit report.

5.18.1. Post-Occupancy Trend Analysis

The CxA may analyze the trend data for systems integration issues and to review operation over variable conditions and times. Trend reviews provide a record of actual operating conditions and may reveal anomalies in settings, schedules, or responses that are inconsistent with the prescribed sequence of operations, where energy efficiencies or equipment operations may be compromised. This method of assessment is also an example of the recommended long term continuous commissioning to assure building operations are optimally maintained.

5.19. Ongoing Commissioning Plan

The CxA shall issue an ongoing Commissioning Plan (Cx Plan) before or as part of the 10-month review of the building operation. The Ongoing Commissioning Plan is effectively a recurrence of the functional performance testing, monitoring based commissioning, and reporting procedures to ensure the building continues to perform according to the OPR, BOD and approved design and construction documentation throughout the lifetime of the building. The CxA shall provide blank functional performance tests for all commissioned systems, the issues log and direction for testing new and retrofitted equipment over time.

Ongoing Cx activities can be performed by in-house operating personnel or by a third party CxA and is required to occur at least twice a year to account for the seasonal variation.

Geotechnical Investigation and Geologic Hazards Evaluation Laney College Library Learning Resource Center

Oakland, California

04.72190021-PR-001 03 | March 31, 2023

Final

Peralta Community College District

Document Control

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Client Information

Client	Peralta Community College District
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02	February 28, 2020	Final	Project Submittal	FDP/VP/AP/AF/TC	DW/RLB	RLB
03	March 31, 2023	Final	Project Submittal	RR/JS/AP	JS/RLB/AF	RLB

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March 31, 2023

Dear Ms. Smith,

Fugro is pleased to submit this final geotechnical investigation and geologic hazards evaluation report for the proposed new Library Learning Resource Center project at Laney College in Oakland, California. Our work was authorized by the District professional service agreement (Requisition No. 2-129461, PO No. 3-118689) dated February 19, 2019, Amendment No. 1 (Requisition No. 2-135365, PO No. 3-122826) dated December 11, 2019, Amendment No. 2 (Requisition No. 2-145003, PO No. 3-131458) dated September 1, 2021, and the Amendment No. 3 (PO No. PCCD1-3000135642) dated August 3, 2022 and was executed in general accordance with the scopes listed in our Proposals No. 04.72189129-P-001(Rev.02), No. 04.72190021-P-001(Rev.00), No. 04.72160021-P-002(02), and No. 04.72190021-P-003 (01), dated February 19, 2019, July 19, 2019, July 12, 2021, and April 27, 2022, respectively.

This report was prepared to identify the key geologic and geotechnical aspects of the site and provide geotechnical recommendations for design and construction of the project. This report also summarizes the results of our geotechnical and geologic site data review, field exploration, laboratory testing, and geologic and seismic hazard evaluations for the project site. We appreciate this opportunity to be of service to the District. Should you have any questions or require additional information, please contact us.

Sincerely,

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Appendices

1. Introduction

This report presents the results of the geotechnical investigation and geologic hazards evaluation conducted by Fugro USA Land, Inc. (Fugro) for the new Library Learning Resource Center on the Laney College campus. The campus is located at 900 Fallon Street in the City of Oakland and County of Alameda, California, as shown on the Vicinity Map (**Plate 1**). A topographic map of the area, along with coordinates for the site (Lat. 37.794899°N and Long.122.262363°W) are presented on the Topographic Site Map (**Plate 2**). Previously, Fugro performed a geotechnical study of the same site in 2002 and the results were presented in a report dated March 27, 2002.

This report was prepared in accordance with guidance from the California Geological Survey (CGS) – Note 48, *Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*, (CGS, 2019), the American Society of Civil Engineers (ASCE) ASCE/SEI 7-16 Standard, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2016), and following the regulations of the 2019 California Building Code (2019 CBC; California Building Standards Commission, 2019).

Because the course of the design changed during the review process by California Geology Survey (CGS) and the Department of the State Architects (DSA), relevant information throughout the body of the previous report dated February 28, 2022, is updated in this report. The discussions and communications with CGS and responses to their comments are shown in Response to CGS Comments, **Appendix I** of this report.

In addition, the design team decided to use Deep Mixing Method (DMM) ground improvement and shallow foundation system in lieu of proposed deep foundation and retaining wall system in our 2020 report. The detailed design assumptions, discussions, recommendations, and specifications are presented in DMM Design and Recommendations, **Appendix J**. Updated information and discussions shown in **Appendices I** and **J** supersede the similar subject in this report.

1.1 Project Description

According to the preliminary building layout plan provided by Noll & Tam Architects and Planners and as shown on the Site Plan (**Plate 3**), we understand that the proposed Library Learning Resource Center site is in the southeast corner of the Laney College main campus and is bounded by 7th Street on the southwest, Lake Merritt Channel on the east, a cooling tower structure and Building E on the northeast, and a handicap parking lot on the northwest. The site is located about 100 feet southwest of the Bay Area Rapid Transit (BART) underground tube easement. According to site survey information provided by CSW/Stuber-Stroeh Engineering

Group, Inc. (April 2019), the existing surface elevations at the proposed building area varies from Elevations of +18 feet to +21 feet (NAVD 88).

The new building is planned to be an at grade, 3-story high building with an estimated footprint area of about 24,197 square feet and project size of 75,622 square feet. The proposed building location is about 130 to 160 feet away from the edge of the Lake Merritt Channel west bank. No significant raising of the existing site grade is anticipated for the project according to the project drawings provide by Noll & Tam Architects dated October 14, 2022.

At the time of our study, the site was occupied by several portable classroom buildings, a small bathroom structure, a small storage shed, and associated concrete walkways and landscaping. Short retaining walls up to about 3 feet high were located to the northeast of the classroom buildings, which retained the existing generally level pad of the existing improvements. Based on available aerial photographs of the site, these existing improvements appeared to be installed between August 2007 and September 2008. These improvements will be removed prior to the new construction.

1.2 Scope of Services

The purpose of our geotechnical investigation and geologic hazards evaluation was to identify key geotechnical, geologic hazards, and seismology aspects of the site in accordance with CGS Note 48 that could impact the project and provide geotechnical recommendations for design and construction of the project. The scope of our services performed included the following:

- Compile and review available geotechnical and geologic data that is contained in our files and provided by others, including existing geologic and seismic hazard maps and other generally available related literature.
- Review previous geotechnical investigation reports for the site and vicinity by Fugro and others, including results of previous exploratory borings, Cone Penetration Tests (CPT), and laboratory testing.
- Conduct a field exploration program including one (1) exploratory boring to a depth of about 76-1/2 feet and eight (8) CPTs to a maximum depth of about 75-1/2 feet;
- Perform geotechnical laboratory testing on selected soil samples for classification, index, strength, consolidation, and corrosivity testing.
- Identify the site geotechnical and geologic conditions (e.g., stratigraphy, subsurface soil characteristic and engineering properties, depths to groundwater, and geologic hazards) that could impact the project, as mandated by CGS Note 48.
- Perform engineering analyses using the field and laboratory data, including detailed liquefaction triggering, post-liquefaction deformation, dynamic densification, lateral spreading, and slope stability evaluations.

- Develop site-specific seismic design criteria per 2019 California Building Code (CBC), including a site-specific ground motion response analysis and a Probabilistic Seismic Hazard Analysis (PSHA).
- Respond the CGS and DSA review comments.
- Provide ground improvement design, drawings, and specifications.
- Communicate with the structural engineer and assist in finalizing the foundation design.
- Prepare this report to summarize the results of our geotechnical and geologic data review, field exploration, laboratory testing, geologic hazards evaluations, and engineering analyses, and to provide geotechnical conclusions and recommendations for design and construction of the project.

Chemical analytical assessment of onsite materials or groundwater for contaminants was beyond our scope of work.

2. Data Review, Exploration and Laboratory Testing

2.1 Review of Existing Data

As part of our study, Fugro reviewed relevant geotechnical, geologic, and seismic data, as well as results of previous explorations and laboratory testing performed in the vicinity of the project site, including the following reports, literature, and maps. The conclusions from our review of the existing data are presented in subsequent sections of this report.

2.1.1 Previous Geotechnical Data and Reports

- Woodward-Clyde-Sherard and Associates, March 9, 1966. *Soil Investigation for the Proposed Peralta Junior College Civic Center Site, Phase 1 – Preliminary Studies*, WCS No. S10312.
- Woodward-Clyde-Sherard and Associates, May 1, 1967. *Peralta College – Chinatown General Neighborhood Renewal Area (GNRA)*, WCS No. 11032.
- Kaldveer Associates, September 9, 1991. *Feasibility Foundation Investigation, Proposed Pool Improvements, Laney College*, Kaldveer No. K1329-1-863.
- Harza Kaldveer, October 22, 1993. *Geotechnical Investigation for Proposed Pool Replacement, Laney College*, Harza No. K1329.
- Fugro, March 27, 2002. *Geotechnical Investigation, New Art Building at Laney College*, Fugro No. 1430.001.
- Fugro, March 29, 2005. *Geotechnical Study and Geologic Hazard Evaluation, Laney College Art Building*, Fugro No. 1430.005.
- Geotechnical Engineering Inc., March 20, 2006. *Additions to Building A & Chiller Room Adjacent to Building B, Laney College*, GEI No. 41357.
- Fugro, August 25, 2006. *Geologic Hazards Evaluation, Laney College Building A Renovation*, Fugro No. 1430.008.
- Fugro, June 10, 2008. *Geotechnical Review, Proposed New Laney College Library Site Study*, Fugro No. 1813.002.
- Terraphase Engineering, May 31, 2012. *Geotechnical Design Report, Proposed Laney College Building Efficiency for a Sustainable Tomorrow (BEST)*, Terraphase No. 0034-001-003.

2.1.2 Geologic Maps, Literature, and Hazard Zonation Maps

- Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006. *Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California*, USGS Open File Report 2006-06-1037.
- Helley and Graymer, 1997. *Quaternary Geology of Alameda County, and Parts of Contra Costa, Santa Clara, San Mateo, San Francisco, Stanislaus, and San Joaquin Counties, California: A Digital Database*, USGS Open File Report 97-97.

- Rogers and Figuers, December 30, 1991. *Engineering Geologic Site Characterization of the Greater Oakland-Alameda Area, Alameda and San Francisco Counties, California*, NSF Grant No. BCS-9003785.
- California Geological Survey, *Earthquake Fault Zones, Oakland West Quadrangle*, Revised Official Map, Released: January 1, 1982.
- California Geological Survey, *Seismic Hazard Zones, Oakland West Quadrangle*, Official Map, Released: February 14, 2003.
- California Geological Survey, 2003. *Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle, Alameda County, California*, Seismic Hazard Zone Report 081.
- Holzer, Bennett, Noce, Padovani, and Tinsley, 2002, revised 2010. *Liquefaction Hazard and Shaking Amplification Maps of Alameda, Berkeley, Emeryville, Oakland, and Piedmont, California: A Digital Database*, USGS Open File Report 2002-02-296.
- Holzer, 1998. *The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction*, USGS Professional Paper 1551-B.
- Youd and Hoose, 1978. *Historical Ground Failures in Northern California Triggered by Earthquakes,* USGS Professional Paper 993.
- California Geological Survey, July 31, 2009. *Tsunami Inundation Map for Emergency Planning, Oakland West Quadrangle.*
- Federal Emergency Management Agency, *Flood Insurance Rate Map (FIRM), Panel 06001C0067H (12/21/18).*
- City of Oakland Community and Economic Development Agency, November 2004, *Safety Element, City of Oakland Safety Plan*.

2.2 Field Exploration

Fugro performed a geotechnical field exploration program that consisted of one (1) exploratory boring to a depth of about 76-1/2 feet and eight (8) CPTs (Cone Penetration Tests) to a maximum depth of about 75-1/2 feet on March 29, 2019, and January 2, 3, and 7, 2020. In addition, three (3) shallow hand auger borings to a maximum depth of about 6 feet were also performed at three (3) CPT locations (2019-CPT-1 through 2019-CPT-3). During the design of the Deep Mixing Method (DMM) ground improvement and to better define the bottom of Young Bay Mud (YBM) layer, we performed 10 additional CPTs to a maximum depth of about 100 feet on November 17, 18, and 22, 2022. The new CPTs are used to develop cross sections for the DMM Design and Recommendations, **Appendix J**. The approximate locations of the borings and CPTs are shown on the Site Plan (**Plate 3**). The locations were determined by pacing or tape measurement from field landmark references; and should be considered accurate only to the degree implied by the method used.

Drilling permits were attained from Alameda County Public Work Agency (ACPWA) for the subsurface explorations. Underground Service Alert (USA) was notified, and a private utility locating company, Bess Testlab, Inc. (BTL) of Hayward, California, was retained to clear the boring and CPT locations prior to explorations. In addition, a hand auger was also used to clear the top 5 to 6 feet of soils for utilities below existing ground surface at some of the boring and CPT locations.

The boring was performed by a State of California C-57 licensed driller, Geo-Ex Subsurface Exploration (GeoEx) of Dixon, California, using a track-mounted CME 75 drill rig equipped with a mud rotary wash system and a 140-lb automatic trip hammer. According to a hammer calibration report provided by Geo-Ex, the 140-pound automatic trip hammer used at the site for soil sampling had been rated as having an average energy transfer ratio of about 91 percent (calibrated on December 18, 2018).

CPTs were performed by both Fugro and Gregg Drilling, LLC (Gregg) of Martinez, California, in general accordance with ASTM D5778. Fugro used a 25-ton truck-mounted rig with an electronic piezocone penetrometer that has a tip area of 15 cm^2, a friction sleeve area of 225 cm^2, and a tip end area ratio of 0.59. Gregg used a 20 and 25-ton truck-mounted rig and a self-anchoring mini track-mounted rig with an electronic piezocone penetrometer that has a tip area of 15 cm^2, a friction sleeve area of 225 cm^2, and a tip end area ratio of 0.8. The cones were advanced at a standard rate of 2 cm/sec into the ground to measure tip resistance, sleeve friction, and excess pore pressure. Pore water pressure dissipation tests were also performed at selected depths. In addition, in-situ soil shear wave velocity measurements were performed at an approximate 5-foot interval at the 2020-CPT-07 location. The CPT logs and interpretations are presented in **Appendix A**.

Our field engineer continuously logged soils encountered in the borings in the field. The soils are classified in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). The logs of the borings as well as a key for the classification of the soils are included in **Appendix A**. Upon completion of our field explorations, the borehole and CPT holes were backfilled with neat cement grout in accordance with ACPWA requirements. All drilling derived soil cuttings and fluids from mud rotary wash drilling were containerized in 55-gallon metal drums and transported to appropriate facilities for disposal by Geo-Ex.

Representative soil samples were obtained during drilling using a Modified California split-barrel drive sampler (outside diameter of 3.0 inches, inside diameter of 2.5 inches) and a Standard Penetration Test (SPT) split-barrel drive sampler (outside diameter of 2.0 inches, inside diameter of 1.375 inches). Soil samples were transmitted to laboratories for evaluation and appropriate testing. The sampler types are indicated in the "Sampler" column of the boring log as designated in **Plate A-1**.

Resistance blow counts were obtained with the drive samplers by dropping a 140-pound automatic trip hammer through a 30-inch free fall in general accordance with ASTM D1586. The samplers were driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches. When the SPT split spoon sampler was used, these blow counts are the standard penetration resistance values (N values). However, due to the large diameter of the Modified California sampler, the blow counts recorded for this sampler are not standard penetration resistance values. These values were multiplied by a conversion factor of 0.63 for the Modified California Sampler and the calculated approximate equivalent N values are presented on our logs within parenthesis. No hammer energy correction had been applied on the N values presented on the logs.

Previously, several exploratory borings and CPTs were performed in 2002 by Fugro and in 1965 by Woodward-Clyde-Sherard and Associates (WCS) at the site and vicinity. The approximate locations of these previous explorations are also shown on the Site Plan (**Plate 3**). Logs of these previous explorations and laboratory testing results are included in **Appendix C** for reference. The results of these previous explorations and laboratory testing are also incorporated into this report.

2.3 Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. This program included:

- Fifteen (15) moisture content and dry unit weight determinations per ASTM D2937,
- Eight (8) hydrometer, sieve, and percent passing #200 sieve analyses per ASTM D422 and D1140,
- One (1) plastic and liquid limits per ASTM D4318,
- Two (2) unconsolidated undrained triaxial shear strength tests (TXUU) per ASTM D2850,
- One (1) incremental consolidation test per ASTM D2435, and
- Three (3) organic content determinations per ASTM D2974.

All tests were performed by Fugro's geotechnical laboratory in Ventura, California and Cooper Testing Laboratory in Palo Alto, California. Our laboratory testing results are included in **Appendix B**. Some of the test results are also presented on the boring logs (**Appendix A**) at the corresponding sample depths.

Corrosivity tests that include redox, pH, chlorides, sulfates, and resistivity were performed by CERCO Analytical, Inc. in Concord, California, on two representative onsite near-surface soil samples (from 2019-CPT-01 at about 2-1/2 feet and 2019-CPT-03 at about 4 feet). The test results and a brief evaluation report prepared by CERCO regarding the onsite near-surface soil corrosivity are also included in **Appendix B**.

3. Geologic and Seismic Setting

This section summarizes the regional geologic and tectonic setting, the local geologic setting, the site geology, and regional active faults and seismicity.

3.1 Regional Geologic and Tectonic Setting

The project site is located near the east shore of the San Francisco Bay in the Coast Ranges geomorphic province (CGS, 2002). The Coast Ranges are northwest-trending mountain ranges, typically rising to 2,000 to 4,000 ft. in elevation, with intervening elongated valleys. The oldest rocks in the range were formed from the subduction of the Farallon Plate beneath the North American Plate during the Jurassic and Cretaceous periods. The Franciscan Formation in the San Francisco Bay area is a complex of graywacke sandstone, shale, and other lithologies that accumulated in the offshore trench in the subduction zone, then were pushed up onto the continent. Later, Tertiary continental sediments and volcanic rocks were deposited over the Franciscan Formation.

Subduction was followed by strike-slip faulting along the San Andreas fault system starting in southern California about 28 million years ago as subduction gradually consumed the Farallon Plate, and the Pacific Plate and the North American Plate boundary migrated northward. The strike-slip motion along faults in the San Francisco Bay Area developed over the past 5 to 10 million years (Atwater, 1970; Wallace, 1990; Atwater & Stock, 1998). At the present time, the San Andreas fault and sub-parallel faults such as the Hayward fault form the boundary zone between the Pacific and North American plates in the San Francisco Bay area. Deformation over the past few million years along various faults of the San Andreas fault system has produced a series of northwest-trending valleys and mountain ranges, including the East Bay Hills, the San Francisco Peninsula, and the intervening San Francisco Bay.

The Hayward fault extends along the western front of the East Bay Hills along the east side of San Francisco Bay and forms an approximate boundary between two distinctly different geologic and physiographic provinces. Based on work by Radbruch (1969), basement rocks underlying the area west of the Hayward fault are primarily those of the Jurassic to Cretaceous Franciscan Complex (about 200 to 80 million years old). East of the Hayward fault, the basement rocks are Jurassic to Cretaceous sedimentary rocks of the Great Valley Sequence (about 140 to 65 million years old). These Mesozoic rocks are overlain by Tertiary volcanic and sedimentary rocks (65 to 2.5 million years old) in the East Bay hills. The San Francisco Bay Area experienced several episodes of uplift and faulting during late Tertiary time (about 25 to 2 million years ago).

The surficial deposits of the flatlands that lie between the hills and the bay are derived from erosion of the Mesozoic and Tertiary rocks in the hills. They are Quaternary in age, or less than about 2 million years old, and consist primarily of alluvial deposits laid down by streams draining the hills. These deposits form and underlie the wide, gently sloping East Bay Plain and provide the relatively level building sites for most of the development in the East Bay. Sediments that reach the bay are deposited as estuarine deposits in the tidal marshes, mud flats, and the floor of the bay.

The position of the Bay shoreline varied throughout the Quaternary as sea level rose and fell in response to glacial cycles. During peak of the last major glaciation, around 15,000 years ago, sea level was about 330 feet lower than it is today and the San Francisco Bay was a wide valley with streams flowing across the valley floor, joining together to flow out the Golden Gate and finally meeting the sea near the Farallon Islands. As the ice from the great continental glaciers melted, sea level began to rise, with the sea entering the Bay about 10,000 years ago. The present sea level was reached within the Bay about 6,000 years ago (Atwater et al., 1977).

As a result of these sea level fluctuations, the thick sequence of sediments in and adjacent to the bay includes layers of estuarine silts and clays deposited during interglacial periods, alternating with layers of sandy alluvial deposits laid down during glacial periods (Atwater et al., 1977; Sloan, 2006). Borehole data to depths of 300 feet in the central part bay show strata from as many as four glacial-interglacial cycles.

3.2 Local Geologic Setting

In the area of the site, thick Quaternary deposits overlie the basement rocks. The Quaternary deposits represent several stages of deposition, which have taken place over the last 2 million years or so. The combined thickness of the sediments above the Franciscan bedrock is estimated to be on the order of 500 feet based on deep boreholes drilled in downtown Oakland (Rogers & Figuers, 1991).

Structurally, the project site is in an area dominated by the active San Andreas Fault system that includes from west to east, the San Gregorio, San Andreas, Hayward-Rodgers Creek, Calaveras, Concord-Green Valley, and Greenville faults, as well as many other minor faults. The Hayward fault borders the western margin of the East Bay Hills in the eastern San Francisco Bay Area. The site lies about 2-1/2 miles southwest of the toe of Oakland Hills, which are part of the Diablo Range that separates the San Francisco Bay from the San Joaquin Valley. The nearest bodies of surface water are the Oakland Inner Harbor, located about 1/2 mile to the south, and Lake Merritt, located 1/4 mile to the north.

3.3 Site Geology

According to Witter et al. (2006), and as shown on the Quaternary Geologic Map (**Plate 4**), the site is located bayward of the historical shoreline, on former tidal flats adjacent to the Lake Merritt Channel that were filled to make land. The site is roughly in the middle of the estimated

500- to 1,400-foot-wide natural outlet channel of Lake Merritt, which had been dramatically reduced in width with development of the region after the 1860s. Filling of this area occurred between 1894 and 1915 based on the study by Rogers and Figuers (1991).

The historical artificial fill overlies Holocene estuarine mud (afem), which is known locally as Young Bay Mud. According to Helley and Graymer (1997), most of the fill placed before 1965 in San Francisco Bay Area was not compacted and consists of dumped or hydraulically emplaced materials. Based on the results of subsurface geotechnical explorations, the site is generally underlain by about 8 to 25 feet thick of heterogenous man-made fills that locally contain various amounts of concrete, brick, and wood debris.

The Young Bay Mud is a water-saturated estuarine deposit, predominantly gray, green and blue clay and silty clay deposited in tidal marshlands and mud flats of San Francisco Bay. The mud generally contains a few lenses of well-sorted, fine sand and silt, a few shelly layers, and peat. The Young Bay Mud was deposited during the post-Wisconsin rise in sea-level, about 12,000 years to present, and interfingers with and grades into fine-grained alluvial deposits at the distal edge of Holocene alluvial fans.

3.4 Regional Faulting and Seismicity

The San Francisco Bay Area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. As described in Section 3.1 and as shown on the Regional Fault and Seismicity Map (**Plate 5**), numerous major fault zones cross through the San Francisco Bay Area, generally trending northwest-southeast. These faults and other local faults have produced many strong earthquakes, magnitude 6.0 and greater, over the last two centuries within about 60 miles (100 km) of the site, as detailed in Section 3.5.

As shown on **Plate 5**, the site is located about 3.5 miles southwest of the Hayward fault zone. The Alquist-Priolo (AP) Earthquake Fault Zone Map of the Oakland West Quadrangle (**Plate 6**) shows that the site is not located within an earthquake fault zone, as designated by the State of California (California Geological Survey (CGS), 1982).

The Hayward fault exhibits typically geomorphic evidence of Holocene (less than 11,000 years) displacement such as shutter ridges, offset drainages, and aligned topographic sags and scarps. The Hayward fault zone varies in width, from relatively narrow traces of 5 to 10 meters in width, to a zone of subparallel strands several hundred meters wide, or more in fault stepovers. Fault creep occurs at the ground surface along most of the Hayward fault, with average measured creep rates of about 4 to 5 mm/year in the Oakland-Berkeley area (WGCEP, 2003).

Active faults located within about 60 miles (100 km) of the project site, and their generalized fault rupture parameters from the U.S. Geological Survey (USGS) are summarized in Table 3.1**.**

Table 3.1: Regional Active Faults and Generalized Rupture Parameters

Fault	Approximate Closest Distance from Site to Fault (miles)	Direction from Site to Fault	Estimated Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
Hayward-Rodgers Creek	3.4	NE	7.3	150	9
Mount Diablo	13.4	N	6.7	25	2
Calaveras	13.9	NE	7.0	123	15
San Andreas	14.6	SW	8.0	472	17
Green Valley	16.5	NE	6.8	56	4.7
San Gregorio	18.9	SW	7.5	176	5.5
Greenville	24.2	E	7.0	50	2
Monte Vista-Shannon	24.6	S	6.5	45	4
West Napa	25.5	N	6.7	30	1
Great Valley 5 Pittsburg Kirby Hills (Closest Section)	27.4	NE	6.7	32	1
Point Reyes	32.9	NW	6.9	47	0.3
Hunting Creek-Berryessa	45.5	N	7.1	60	6
Zayante-Vergeles	51.2	SE	7.0	58	0.1
Maacama-Garberville	58.7	NW	7.4	221	9
Sources: *USGS, 2008. National Seismic Hazard Maps – Source Parameters* website					

Earthquakes on the faults in Table 3.1 or on smaller, mapped or unmapped faults could cause strong ground shaking at the site. A USGS Fact Sheet (Aagaard et al., 2016) indicates there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2014 and 2043. Earthquake intensities will vary throughout the San Francisco Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site, and other factors.

According to 2019 CBC and ASCE 7-16, and based on an average soft clay soil site condition (Site Class E), the site geometric mean peak ground acceleration (PGA_M) from the Maximum Considered Earthquake (MCE) is estimated to be about 0.80g. The MCE peak ground acceleration has a 2 percent probability of being exceeded in 50 years (a mean return period of 2,475 years), except where deterministically capped along highly active faults.

3.5 Historical Seismicity

Major earthquakes have been recorded along the San Andreas Fault system and across California since the late 1700s. Table 3.2 presents large magnitude ($M \geq 6.0$) regional earthquakes within about 60 miles (100 kilometers) of the site from 1800 to 2018, arranged in chronological order. The Northern California Earthquake Data Center (NCEDC) and National Atlas of United States database was accessed to obtain the historical seismicity information presented in Table 3.2. The epicenter locations are shown on the Regional Fault and Seismicity Map (**Plate 5**).

Table 3.2: Large Magnitude (M≥6.0) Earthquakes Within About 60 Miles (100 km) of the Site

Epicenter Location	Date	Magnitude	Distance (mi)	Direction from Site to Epicenter
Near San Francisco	6/21/1808	6.0	13.1	W
In the San Francisco Bay Area	6/10/1836	6.8	3.4	E
In the San Francisco Area	6/1838	7.0	15.5	SW
North of San Jose	11/26/1858	6.1	28.4	SE
In the Santa Cruz Mountains	10/8/1865	6.3	45.6	SSE
Near Hayward	10/21/1868	6.8	11.0	SE
West of Antioch	5/19/1889	6.0	24.3	NE
Near Vacaville	4/19/1892	6.4	44.4	NNE
Near Winters	4/21/1892	6.2	52.5	NNE
Near Mare Island	3/31/1898	6.2	28.9	NNW
Near San Francisco	4/18/1906	7.8	14.8	SW
Near Coyote Hills	7/1/1911	6.6	46.9	SE
Near Morgan Hill	4/24/1984	6.2	45.1	SE
Loma Prieta	10/17/1989	6.9	56.3	SSE
Napa	8/24/2014	6.0	29.1	N

Several of these events were strong enough to cause structural damage to buildings in Oakland. The estimated M6.8 1868 Hayward earthquake ruptured the Southern Hayward fault from Hayward northward to Oakland and damaged or destroyed numerous buildings in Hayward, San Leandro, Oakland, and San Francisco (Lawson, 1908). The 1868 Hayward earthquake apparently resulted in damage in Oakland corresponding to Modified Mercalli Intensity (MMI) VIII (partial damage to buildings, walls); however, there is little reported information on ground shaking and damage in Oakland, largely because the area was sparely populated at the time of these earthquakes (Toppozada & Park, 1982; Toppozada, 2000).

During the moment magnitude (M_W) 7.8 1906 San Francisco earthquake, the San Andreas fault ruptured over a distance of about 296 miles (474 km) from Shelter Cove near Cape Mendocino southward to near San Juan Bautista. Maximum lateral displacements of 15 to 20 feet (4.6 to 6.1 meters) occurred north of the Golden Gate at Olema in Marin County (Lawson, 1908). Landslides, liquefaction, and ground settlement occurred throughout the Bay Area and in the vicinity of the surface rupture as a result of this earthquake. The ground shaking in Oakland during the 1906 earthquake is characterized as MMI VII to IX (minor to major damage to and collapse of structures; Lawson, 1908; Boatwright & Bundock, 2005). Significant damage occurred to masonry buildings across the city (Lawson, 1908). Ground failure effects, including liquefaction, lateral spreading, and settlement occurred in several areas along the Oakland-Alameda Estuary and at the southern end of Lake Merritt, (Youd & Hoose, 1978; Knudsen et al., 2000).

The most significant recent seismic event to occur in the San Francisco Bay Area was the October 17, 1989, Loma Prieta earthquake. The epicenter of this earthquake was located approximately 56 miles southeast of the site. This moment magnitude 6.9 earthquake ruptured a 22-mile (35-km) section of a splay of the San Andreas fault. The 1989 Loma Prieta earthquake produced MMI VII to VIII effects in the vicinity of the site (McNutt & Toppozada, 1990). Specific ground failure effects near the project site at Lake Merritt and the estuary channel resulting from the 1989 Loma Prieta and 1906 San Francisco earthquakes are described in the Section 5.2.1.

In addition to the damage from liquefaction near the site, the 1989 Loma Prieta Earthquake caused minor to significant damage to structures in the vicinity of the project site, including collapse of the elevated Cypress Structure on the west side of Oakland, and damage to buildings in downtown Oakland. The recorded peak ground acceleration from strong ground motion stations near the site is listed in Table 3.3; the nearest sites (within one mile of the project site) had PGAs of 0.18 to 0.26g.

Table 3.3: Strong Ground Motion Recordings from the 1989 Loma Prieta Earthquake

Station Number and Name	Site Conditions	Peak Ground acceleration (g)	Station Distance from Site (Miles)
58483 – Oakland – 24 story residential building	Alluvium	0.18 (ground)	0.35 NE
58224 – Oakland Title Ins. & Trust – 2 story building	Alluvium	0.26 (ground) 0.21 (revised NGA)	0.8 NNW
58334 – Piedmont – 3 story school office building	Serpentinite	0.08 (ground) 0.18 (structure)	2.4 NE
58338 – Piedmont Junior High School grounds	Weathered serpentinite	0.08 (ground)	2.5 NE
58472 – Oakland-Outer Harbor Warf	Fill/Bay Mud	0.29 (ground)	3.4 WNW
1662 – Emeryville – 6363 Christie	Alluvium	0.25 (ground, revised NGA)	3.8 NW
Data from U.S. National Center for Engineering Strong Motion Data (URL: http://www.strongmotioncenter.org/), and the Pacific Earthquake Engineering Research (PEER) Center Next Generation Attenuation (NGA) Database (http://peer.berkeley.edu/nga/earthquakes.html)			

4. Site Conditions

This section describes historical and present land use and topography at the project site, subsurface soils and geologic strata, and groundwater conditions based on project geotechnical data.

4.1 Surface Conditions

At the time of our study and as shown on the attached Site Plan (**Plate 3**), the proposed Library Learning Resource Center site is in the southeast corner of the Laney College main campus and is bounded by 7th Street on the southwest, Lake Merritt Channel on the east, a cooling tower structure and Building E on the northeast, and a handicap parking lot on the northwest.

The site is occupied by several portable classroom buildings, a small bathroom structure, a small storage shed, and associated concrete walkways and landscaping. Several large and small diameter trees were located around the perimeter of the site. Short retaining walls up to about 3 feet high are located to the northeast of the classroom buildings, which retained the existing generally level building pad. Based on available aerial photographs of the site, these existing improvements appeared to be installed between August 2007 and September 2008.

According to site survey information provided by CSW/Stuber-Stroeh Engineering Group, Inc. (April 2019), the existing surface elevations at the proposed building location varies from Elevations of +18 feet to +21 feet (NAVD 88). The areas to the east of the proposed building location sloped gently downward toward the Lake Merritt Channel with inclinations of about 6:1 (horizontal to vertical) to 10:1. The top of the adjacent channel bank is at about Elevation of 7 feet.

Comparing the topographic information contained on the site plan Figure 1 of the 2002 Fugro report, the current site grade appears to have been modified to create the generally level pad for the portable classroom buildings. We estimated minor cut and fill grading of up to about 2 to 3 feet had been performed at the site during the portable classroom development in 2007 or 2008. The actual details of the previous grading are unknown. We recommend any available previous grading and construction records be forwarded to us for further review.

In addition, based on our review of historical USGS topographic maps from 1915 to 1980 and aerial photographs of the site vicinity from 1993 to 2018, it is our understanding that the Lake Merritt Channel had been re-aligned and widened in 1970s to the current alignment. In the site area, the old channel west bank was located about 140 feet east of the current west bank.

4.2 Subsurface Conditions

The subsurface soil conditions encountered by our borings and CPTs at the proposed Library Learning Resource Center site are consistent with Quaternary geologic mapping of the project site vicinity that shows artificial fill overlying estuarine mud. Similar subsurface soil conditions were also reportedly encountered by previous borings and CPTs by Fugro and others in 1965 and 2002 at the site and vicinity. Our interpretations of the site subsurface soil conditions are presented on the Cross-Sections A-A' through E-E' (**Plates 7** through **11**, respectively).

The subsurface soils below the site generally consisted of predominately medium dense sandy fills that extended to depths of about 8 to 25 feet (Elevation of about +8 feet to -5 feet). Clayey fills of about 2 to 4 feet thick were also encountered in some areas. These fills are heterogenous and locally contain various amounts of concrete, brick, and wood debris. An unknown obstruction was also previously encountered at about 5 feet deep at the 2002-CPT-1 location. Most of these fills appear to be derived from the historical filling of the natural Lake Merritt outlet channel between 1860s and 1940s, and the later development of the Laney College campus in 1960s. Most likely these fills were not compacted to current acceptable geotechnical engineering standards.

Below the surficial fill layer, very soft to soft, high moisture content, and low shear strength Young Bay Mud was encountered to a depth of about 30 feet (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet (Elevation of about -30 feet) at the southeast side of the proposed building location. Some thin loose to medium dense sand lenses about 2 to 6 feet thick were also encountered within the Young Bay Mud layer. About 15-feet of loose to medium dense sands were also encountered between the surficial fill and the Young Bay Mud layers in 2019-CPT-3. These sands could be either historical fills placed in the natural Lake Merritt outlet channel or natural sand deposits that existed within the channel.

Underlying the Young Bay Mud layer, medium dense to very dense sands and stiff to hard clays were encountered to the maximum depth explored of about 76-1/2 feet (or elevation of about -60 feet).

The thin surficial layers of clayey fills are considered to have a low to medium plasticity and low to moderate expansion potential; the sandy fills are non-expansive. Our logs and interpretations of borings and CPTs are presented in **Appendix A**. Our laboratory testing results of the onsite soil samples are included in **Appendix B**. Logs of historic explorations and results of lab testing are included in **Appendix C** for reference.

4.3 Groundwater

Based on CPT pore pressure dissipation tests at selected depths, the site groundwater table is estimated to be at depths of about 5 to 18 feet (Elevations of about 0 to +9 feet). In addition, groundwater was reportedly encountered at 2002-CPT-2 location at a depth of about 11 feet (Elevation about +8 feet). The previous borings (2002-EB-1 through 2002-EB-3) also reportedly encountered groundwater at depths of about 15 to 45 feet (Elevations of about +5 to -27 feet). It should be noted that these borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, tidal effects, and other factors. According to CGS Seismic Hazard Zone report for the Oakland West Quadrangle (CGS, 2003), as shown on **Plate 13**, historically high groundwater in the site region had been reported at a depth of about 10 feet.

We recommend a design groundwater Elevation of +8 feet be used for the project designs, which generally corresponds to both the top elevation of Young Bay Mud layer within the project area and the top elevation of the adjacent Lake Merritt Channel bank.

5. Geologic Hazards Evaluation

Site geologic hazard evaluations were performed in accordance with guidance from the CGS Note 48 (CGS, 2019). The opinions, conclusion, and recommendations in the following sections were based on the results of our review of available information relating to geotechnical, geologic, and seismic data within the vicinity of the site, project field exploration and laboratory programs, and site-specific engineering analyses.

Hazard evaluations are grouped into five sections, addressing: 1) fault rupture, 2) seismic ground shaking effects (liquefaction, dynamic densification, and lateral spreading, 3) slope stability, 4) compressible soils (settlement of non-engineered fills and young sediments), and 5) other hazards (expansive soils, corrosive soils, volcanic eruptions, flooding and dam inundation, tsunami and seiche, naturally occurring asbestos, and hydrocompaction).

5.1 Fault Rupture Hazard Evaluation

Surface fault rupture occurs when an earthquake results in displacement of the ground surface along the trace of an active fault. Based on existing geologic maps and literature, there are no known active fault traces within, adjacent to, or trending towards the project site. The closest known active fault is the Hayward Fault, located approximately 3.6 miles (5.8 kilometers) to the northeast. The site is not located within a Fault-Rupture Hazard Zone, as shown on the Earthquake Fault Zone Map for the Oakland West Quadrangle (CGS, 1982). No other faults are mapped or know to occur near the project site. Based on this information, the potential for surface fault rupture at the site is very low.

5.2 Seismic Ground Shaking Effects

Strong ground shaking at the project site is anticipated during a moderate to severe earthquake occurring anywhere in the Bay Area. Strong ground shaking can cause direct damage to structures; and has the potential of inducing other phenomena that can cause indirect damage to structures. These phenomena include soil liquefaction, dynamic densification of dry soils, lateral spreading, and ground cracking, seismically induced waves, such as tsunamis and seiches, inundation due to dam or embankment failure, and landsliding.

Detailed discussions of liquefaction, dynamic densification and lateral spreading with respect to the site are presented in the subsequent paragraphs of this section. Discussions of landsliding, both static and seismically induced, are presented in Section 5.3, and discussions of tsunami, seiche, and flooding due to dam or embankment failure are presented in Section. 5.5.

5.2.1 Liquefaction and Dynamic Densification

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts. In addition, dynamic densification may occur within loose to medium dense, dry sand layers located above groundwater level.

According to Witter et al. (2006) and the Association of Bay Area Governments (ABAG) Resilience Program Liquefaction Susceptibility Map, the site (as shown on **Plate 12**) is located in an area that has been characterized as having a very high liquefaction susceptibility. The Seismic Hazard Zones Map of the Oakland West Quadrangle (CGS, 2003) indicates the site is located within a liquefaction seismic hazard zone (as shown on **Plate 6**), as designated by the State of California.

Our site liquefaction evaluations, which included liquefaction history review and liquefaction triggering and post-liquefaction deformation analyses, are presented in the following sections. In addition, potential for dry sand dynamic densification was also evaluated.

5.2.1.1 Historical Liquefaction in Site Region

According to the seismic hazard zone report of Oakland West Quadrangle (CGS, 2003), several historical liquefaction events had been documented from past earthquakes. Youd and Hoose (1978) compiled observed ground failures caused by earthquake shaking in northern California, including the 1906 San Francisco and 1868 Hayward earthquakes. Following the 1906 earthquake, a 24-inch steel pipe crossing 12th Street at Lake Merritt dam (Site 175 as indicated on **Plate 13**) was reportedly snapped from the settling of the flood gate. The foundation of Lake Merritt dam was also reported as “cracked and broken”. Along the west shore of Lake Merritt, the bank had been cracked and broken, and caved off into the lake.

In addition, liquefaction related ground failures caused by earthquake shaking occurred during the 1989 Loma Prieta earthquake throughout the San Francisco Bay Area and are summarized by Tinsley et al. (1998) . Ground settlement and several sand boils (Site 43) were observed along Lake Merritt Channel Park and Peralta Park, adjacent to the Laney College campus. The ground settlement resulted in the rupture of 6-, 12-, and 36-inch diameter main pipelines. Lateral spreading apparently occurred on the western bank of Lake Merritt during the 1906 event, but this bank was not distressed during the 1989 earthquake.

It is also our understanding damage to the original Laney College swimming pool, located to the north of Building E, was reported after the 1989 earthquake (Kaldveer, 1991), probably as the result of soil liquefaction. A replacement swimming pool was constructed in mid-1990s.

5.2.1.2 Liquefaction Evaluation Methodology

We performed both CPT- and SPT-based liquefaction triggering and post-liquefaction deformation analyses for the site generally in accordance with the guidelines listed in the CGS Special Publication 117A (2008) and the recommended procedures by Southern California Earthquake Center (SCEC, 1999).

Our analyses were based on a peak ground acceleration from a Maximum Considered Earthquake (MCE) event. A geometric mean MCE peak ground acceleration (PGA_M) of 0.81g (adjusted for a Site Class E soil condition) with a mean earthquake magnitude of Mw 7.0 and a modal magnitude of Mw 7.5 were determined for the site per ASCE 7-16 and seismic hazard deaggregation (USGS 2014 model). Our recommended project design groundwater level, at Elevation of +8 feet, was used in the analyses to assess its impacts on liquefaction and liquefaction induced ground surface damage potential.

For comparison and sensitivity evaluation purposes, both methodologies described by NCEER (2001) and by Boulanger and Idriss (BI, 2014) were used for CPT-based analyses. Post-liquefaction deformations were calculated for all layers by using Ishihara and Yoshimine procedures (1992) for NCEER method and EERI Monograph 12 procedures (Idriss & Boulanger, 2008) for BI 2014 Method. Sensitivity analysis was performed with changing earthquake Magnitude to Mw 7.6. Sensitivity analyses show the estimated settlements are not sensitive to earthquake Magnitude as the volumetric strain models saturate at the already low Factors of Safety estimated in this study. Further discussion is provided in Response to CGS Comments, Appendix I.

The SPT-based analyses generally followed the methodology described in the EERI Monograph 12 (MNO-12, Idriss & Boulanger, 2008). Per CGS Note 48 requirements, post-liquefaction deformations were calculated for soil layers that have a factor of safety against liquefaction less than 1.3.

5.2.1.3 Liquefaction Evaluation Results and Conclusions

Our results from both CPT- and SPT-based analyses generally indicate that the saturated, loose to medium dense sand layers of various thicknesses located both above and within the Young Bay Mud layer have a high potential for liquefying when they are subjected to an MCE earthquake event. The majority of these sand layers were encountered in borings and CPTs at the site within depths of about 30 to 40 feet (above Elevation of about -15 feet). The extent of the potentially liquefiable soils, factors of safety against liquefaction triggering, and calculated liquefaction-induced cumulative ground settlements at each boring and CPT location are presented in **Appendix D**.

We calculated that the MCE earthquake-induced liquefaction in these sand layers would result in residual volumetric strains varying from about 1 to 4 percent and total ground surface settlements (without reduction associated with the depth of occurrence) ranging from as little as 1 inch to up to about 6-1/2 inches. The table below summarizes the calculated liquefaction-induced settlement using the three different methods referenced above for the site boring and CPT locations. It should be noted the actual ground settlements may differ from our estimates due to uncertainties in the current liquefaction triggering and settlement analysis methodology. In addition, it is a generally accepted idea that the contribution of liquefiable soil layers to surface settlement diminishes as the depths of the layers increase.

Table 5.1: CPT- and SPT-Based Liquefaction Analysis Results

Location	Liquefiable Soil Elevation (ft)	Calculated Cumulative Ground Settlement (inches)		
		MNO-12 SPT Method	NCEER 2001 CPT Method	BI 2014 CPT Method
2019-CPT-01	+8 to +1.5 -5 to -7.5	-	3-1/4	3-1/2
2019-CPT-02	+7 to -2.5 -26.5 to -31	-	2-1/2	3
2019-CPT-03	+7 to +3.5 +2 to -14 -37.5 to -39	-	5	6-1/2
2020-CPT-04	+8 to +6 -9 to -13	-	1-1/2	1-3/4
2020-CPT-05	+7 to +5 -12.5 to -14.5 -17 to -19 -24 to -29	-	2-1/4	2-3/4
2020-CPT-06	+8 to +4.5 -0.5 to -3 -38 to -40 -43 to -45.5	-	2	2-3/4
2020-CPT-07	+8 to +7 -33 to -35 -38 to -40.5	-	1	1
2020-CPT-08	+3.5 to 2 -12.5 to -16 -27 to -31	-	2	2-1/4
2002-CPT-2	+7 to +6 -9.5 to -12	-	1	1-1/2
2020-B-01	+7 to +0.5 -13 to -18.5 -31 to -34	3-1/2	-	-
2002-EB-1	-12 to -17 -22 to -27	3-1/4	-	-
2002-EB-2	-	0	-	-
2002-EB-3	-10 to -16 -28 to -33	2-1/2	-	-

Based on our review of available maps and literature, and the results of our site evaluations, it is our opinion, when the site is subjected to a Maximum Considered Earthquake (MCE) event, the likelihood of liquefaction occurring at the site is high.

5.2.2 Dynamic Densification Evaluations

We performed both CPT-based and SPT-based dynamic densification evaluations based on procedures developed by Tokimatsu and Seed (1987) and Robertson and Shao (2010). A geometric mean MCE peak ground acceleration (PGA_M) of 0.81g, a mean earthquake magnitude of 7.0, and the project design groundwater level at Elevation of +8 feet were used in our analyses. The potential dynamic densification settlements of the near-surface unsaturated sandy fills of about 8 to 13 feet in thickness at the site are estimated to be on the order of 1/4 to 1/2 inch after the MCE event. The detailed results of each boring and CPT location are presented in **Appendix E**. It is our opinion that the potential for soil dynamic densification to impact the site is low.

5.2.3 Lateral Spreading

Lateral spreading occurs when soils liquefy during an earthquake event and the liquefied soils, along with the overlying soils, move laterally toward a free face or unconfined space, such as the west bank of the Lake Merritt Channel. Lateral spreading can result in significant horizontal ground displacements.

Our site lateral spreading evaluations generally followed methodology described in the EERI Monograph 12 (MNO-12, Idriss and Boulanger, 2008) to estimate the maximum shear strain of each liquefiable soil layer and calculate the Lateral Displacement Index (LDI) (Zhang et al., 2004) at each CPT and boring location. The detailed results are included in **Appendix D**.

In addition, empirical correlations developed by Youd et al. (2002) were also used to identify the potential soil layers that are prone to trigger ground lateral spreading and to provide estimates for possible ground lateral displacement. According to Youd et al. (2002), saturated cohesionless soil sediments with SPT $N_{1,60}$-value equal or more than 15 are considered as not likely to have significant displacement during earthquakes smaller than magnitude 8. Our calculated LDIs and order of ground lateral displacements (from soil layers having $N_{1,60}$-value less than 15) at the site CPT and boring locations are summarized in the table below. It should be noted these values should be considered as an index due to the limitations of the current engineering knowledge and analysis methodology. The Table 5.2 lateral displacement values are for Mw 7.0 earthquake. The calculated lateral displacement for the Mw 7.6 earthquake is presented in Response to CGS Comments, **Appendix I**.

Table 5.2: CPT- and SPT-Based LDI and Lateral Displacement Analysis Results

Location	Liquefiable Soil Elevation (ft)	Calculated Lateral Displacement Index - LDI (inches)	Potential Lateral Spreading Triggering Soil Elevation (ft)	Estimated Ground Lateral Displacement (inches)
2019-CPT-01	+8 to +1.5 -5 to -7.5	42	-	0
2019-CPT-02	+7 to -2.5 -26.5 to -31	30 to 33	+7 to -2.5	12 to 24
2019-CPT-03	+7 to +3.5 +2 to -14 -37.5 to -39	56 to 59	+7 to -8	18 to 36
2020-CPT-04	+8 to +6 -9 to -13	18	-	0
2020-CPT-05	+7 to +5 -12.5 to -14.5 -17 to -19 -24 to -29	22 to 24	+7 to -2	12 to 24
2020-CPT-06	+8 to +4.5 -0.5 to -3 -38 to -40 -43 to -45.5	25 to 27	+8 to -5	12 to 30
2020-CPT-07	+8 to +7 -33 to -35 -38 to -40.5	10	+8 to -5	12 to 24
2020-CPT-08	+3.5 to 2 -12.5 to -16 -27 to -31	22	-	0
2002-CPT-2	+7 to +6 -9.5 to -12	12	-	0
2020-B-01	+7 to +0.5 -13 to -18.5 -31 to -34	23	+7 to +3.5	6 to 18
2002-EB-1	-12 to -17 -22 to -27	30	-	0
2002-EB-2	-	0	-	0
2002-EB-3	-10 to -16 -28 to -33	23	-	0

Our results generally indicate the loose to medium dense sand layers encountered by CPTs and borings in the area adjacent to the Lake Merritt Channel at Elevations between +8 and -5 feet have a high potential to trigger ground surface lateral spreading during soil liquefaction from an

MCE event. The other onsite liquefiable sand layers are considered as having low potential to trigger ground lateral spreading due to their presence in isolated thin pockets and/or being located at deeper depths in relation to the bottom of the Lake Merritt Channel. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

5.3 Slope Stability Analysis

The project proposed building is located about 130 to 160 feet away from the edge of the west bank of the channel. Our evaluations are only meant to assess the global stability of the proposed development and the potential lateral extents of ground failures caused by the possible lateral spreading of the channel bank during an MCE event (if it occurs). Detailed stability evaluation of the existing channel west bank is beyond our scope of work, since soil stratigraphy below the bank and channel were extrapolated from data developed for the project area. The Seismic slope stability analysis is further discussed in Response to CGS Comments, **Appendix I**.

The global site slope stability was evaluated using a two-dimensional, limit equilibrium computer program, SLOPE/W (GeoStudio 2016, Ver. 8.16.1.13452), and Spencer analysis method. The recommended analysis procedures by South California Earthquake Center (SCEC, 2002) were generally followed. The representative Cross-Sections A-A', D-D' and E-E' (**Plates 7, 10** and **11**) were used in our analyses to evaluate the following four (4) design loading cases:

- Case 1: Long Term (Static)
- Case 2: Seismic Event Yield Acceleration (Pseudo-static)
- Case 3: Seismic Event k = 0.15g (Pseudo-static); Fixed Slip Surface at Edge of Building
- Case 4: Post-Liquefaction (Static)

Factors of safety against slope stability failures were calculated for the Cases 1, 3, and 4. Pseudo-static yield acceleration (ky to achieve a factor of safety equals 1.0) was calculated for Case 2.

5.3.1 Subsurface Soil Engineering Properties

Soil engineering properties were developed based on the field exploration and laboratory testing results by Fugro and others, and typical engineering correlations. The table below summarizes the soil properties used in our analyses.

Table 5.3: Soil Engineering Properties Used in Site Slope Stability Analyses

Material	Unit Weight (pcf)	Material Shear Strength	
		Cohesion c′ (psf)	Friction Angle Φ′ (degree)
Sandy Fill	120	0	35
Young Bay Mud with Sand Lenses	90	0.35 x Effective Overburden Stress (psf)	0
Interbedded Clays and Sands	130	0	40
Highly Liquefiable Sands	110	0	33
Post-Liquefaction Sands (Residual Strength)	110	100 + 20 x Depth (ft)	-

5.3.2 Slope Stability Analysis Results and Conclusions

The results of our slope stability analyses are presented in the table below. Our interpreted cross-section stratigraphic profiles, soil engineering properties used in the analyses, and the detailed results of the analyses are presented on the computer program printouts in the attached **Appendix F**.

Table 5.4: Slope Stability Analysis Results

Cross-Section	Case 1 Long Term	Case 2 Seismic Event Yield Acceleration	Case 3 Seismic Event k = 0.15g; Fixed Slip Surface at Edge of Building	Case 4 Post-Liquefaction
	Factor of Safety	ky	Factor of Safety	Factor of Safety
A-A′	2.8	0.12	0.9	2.6
D-D′	2.2	0.12	0.9	2.0
E-E′	1.7	0.11	0.9	1.5

The results of our slope stability analyses generally indicate that the factors of safety against slope failures for the Case 1 (Long Term, Static) are 2.8, 2.2, and 1.7, respectively, for Sections A-A′, D-D′ and E-E′, which exceed the generally accepted minimum allowable value of 1.5 for long term conditions.

For the Case 2 (Seismic Event Yield Acceleration, Pseudo-static), the yield accelerations (ky) are determined to be 0.12g, 0.12g, and 0.11g, respectively, for Sections A-A′, D-D′, and E-E′. Using the Bray (1998) procedure as recommended by the SCEC publication (2002), we calculated slope displacements on the order of about 15 to 24 inches (38 to 61 centimeters) may occur during an MCE event (with a maximum horizontal acceleration of 0.81g from a mode magnitude 7.5

causative earthquake located at 6.8 kilometers from the site). These calculated displacements exceed the threshold of 6 inches (15 cm) defined by the SCEC publication (2002), which likely distinguishes conditions in which small to moderate displacements are likely from conditions in which large displacements are likely. However, as indicated on the result printouts in **Appendix F**, the most critical slip surfaces along these cross-sections do not daylight within the proposed building location.

In addition, by fixing the slip surface daylight location at the edge of the proposed building location, factors of safety against slope failures for the Case 3 (Seismic Event k = 0.15g, Pseudo-Static) are all 0.9 for Sections A-A', D-D' and E-E', which also fail to meet the commonly accepted minimum value of 1.15 for seismic performance (Seed, 1979)[34]. It should also be noted, due to the low undrained shear strength of Young Bay Mud used in the Case 2 and Case 3 analyses (pseudo-static), the calculated low factors of safety and the estimated large and deep slip surfaces (35 to 45 feet deep below the top of channel bank) may not fully represent the seismic global slope stability at the proposed building location (which is about 130 to 160 feet away from the edge of the channel bank). Seismic slope stability of site is most likely governed by the extent of possible ground lateral spreading during major liquefaction events.

In Case 4 (Post-Liquefaction, Static), post-liquefaction residual shear strength was used for the highly liquefiable sands. The factors of safety against slope failures are 2.6, 2.0, and 1.5, respectively, for Sections A-A', D-D' and E-E', which exceed the generally accepted minimum value of 1.3 for short term conditions after major liquefaction events.

Due to the high degree of uncertainties on site subsurface conditions, seismic characteristics of the triggering earthquake, and analysis methodology, the results of our seismic slope stability and lateral spreading analyses should be considered as an index of site performance during major earthquake events. It is our opinion that the potential for slope instability during an MCE event and/or after major liquefaction event to impact the proposed building location is low to moderate. However, extensive slope failures may occur for the areas immediately adjacent to the Lake Merritt Channel if soil liquefaction and ground lateral spreading do occur at the site region during major earthquake events. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

5.4 Compressible Soils

The site is blanketed by historical sandy or clayey fills that extend to depths of about 8 to 25 feet (Elevation of about +8 feet to -5 feet). Most of these fills appear to be derived from the historical filling of the natural Lake Merritt outlet channel between 1860s and 1940s, and the later development of the Laney College campus in 1960s. These fills are heterogenous and locally

[34] Seed, 1979. *Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams*, Geotechnique, V. 29 (3), p. 215-263.

contain various amounts of concrete, brick, and wood debris. These historical fills were most likely not compacted to the current acceptable geotechnical engineering standards and are potentially compressible. In addition, we estimated minor cut and fill grading of up to about 2 to 3 feet had been performed at the site during the portable classroom development in 2007 or 2008. The actual details of the previous grading are unknown.

Below the surficial fill layer, Young Bay Mud was encountered to about 30 feet deep (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet deep (Elevation of about -30 feet) at the southeast side of the proposed building location. This 15- to 35-foot-thick layer of slightly over-consolidated to normally consolidated Young Bay Mud is very soft to soft, has a high moisture content and a low shear strength, and is highly compressible. Under additional new loads, such as weights of the new fills and structures, the Young Bay Mud will consolidate while the induced excess pore water pressures are dissipating, which may cause detrimental total and differential settlements to the imposing structures and improvements.

We estimate the primary consolidation settlement due to the historical fills placed prior to 1960s at the site should have been completed. Additional settlements from the recent fill placement during the portable classroom development in 2007 or 2008 may be still ongoing. We recommend any available previous grading and construction records be forwarded to us for further review.

No significant raising of the existing site grade is anticipated for the project. If new fills will be placed to raise the existing grade, we anticipate that additional settlement will occur in the future. Our analyses indicate that for every foot of new fills that will be placed, it would induce an additional ultimate settlement of about 2 to 3 inches over the next 10 to 30 years. This additional settlement will also likely affect the integrity of the existing and/or new utility lines. In addition, this settlement will also cause downdrag forces to the pile-supported structure.

5.5 Other Geologic Hazards

Below we briefly review other geologic hazards identified by the CGS (2019) Note 48 as exceptional geologic hazards or adverse site conditions that do not occur statewide. This section addresses expansive soils, corrosive soils, volcanic eruptions, flooding and dam inundation, tsunami and seiche, naturally occurring asbestos, and hydrocompaction.

5.5.1 Expansive Soils

The near-surface soils encountered at the site were predominately man-made fills that consist of silty sands and lean clays. The expansion potential of the near-surface soils at this site is considered low to moderate. The potential expansive soil hazard can be further reduced provided our recommendations in the report are followed.

5.5.2 Corrosive Soils

Corrosivity tests, that include redox, pH, chlorides, sulfates, and resistivity were performed by CERCO Analytical, Inc. on two representative onsite near-surface soil samples (from Boring 2019-CPT-01 at about 2-1/2 feet and 2019-CPT-03 at about 4 feet). The test results and a brief evaluation report prepared by CERCO regarding the onsite soil corrosivity are also included in **Appendix B**. According to the evaluation report, the onsite near-surface soils should be considered as "moderately" and "slightly" corrosive based on resistivity and redox potentials measurements, respectively.

5.5.3 Volcanic Eruption

The hazards of volcanic eruption include impact and inundation by lava flows, volcanic mudflows, or pyroclastic flows, and the effects of airborne volcanic ash and gases. No active volcanoes occur in the San Francisco Bay area. The nearest active volcano is the Clear Lake Volcanic Field, located about 90 miles north of the site. Volcanic flows would not extend far enough to affect the site. Airfall ash, which is known to travel great distances, would likely travel eastward based on prevailing winds. Potential hazards associated with volcanic activity in the site region are estimated to be very low (Miller, 1989).

5.5.4 Flooding and Dam Inundation

In this section we provide a brief discussion of flooding and dam inundation hazard based on a review of readily available information. A detailed risk evaluation of flooding and inundation at the site was not performed because the initial screening evaluation did not identify any significant flooding or inundation hazards.

According to the FEMA (2018) flood insurance rate map for Oakland, the project building area is located outside a 100-year flood zone. The site is adjacent to the Lake Merritt Channel, which serves as the outlet for Lake Merritt and whose level is controlled by tide level in the Oakland Inner Harbor and the level of Lake Merritt, which is partially tidal. Tide gates at the 7th Street bridge regulate the water flows into and out of Lake Merritt. The elevation of the site is 18 to 21 ft (NAVD88) and the highest astronomical tide (HAT) in the Oakland Inner Harbor is about 8 feet (NAVD88) (https://tidesandcurrents.noaa.gov/datums.html?id=9414764), at least 10 feet lower than the site. Runoff into Lake Merritt from heavy storms could raise the water level in the tidal channel if lake waters were released through the tide gates, but this is not likely to exceed the HAT.

The City of Oakland notes that there are 13 active dams, reservoirs, and clearwells that, in case of failure, would cause flooding in Oakland. These facilities include:

- Central, Claremont, Dingee, Dunsmuir, Estates and 39th Avenue reservoirs, the dams at Lake Chabot and at Upper San Leandro reservoir, and the Upper San Leandro filtration plant no. 1 and no. 2 clearwells (owned by the East Bay Municipal Utility District, EBMUD);
- Lake Temescal dam (owned by the East Bay Regional Park District);
- Lower Edwards and Upper Edwards reservoirs (owned by the Mountain View Cemetery Association); and
- Lower and Upper Edwards reservoirs, owned by the Mountain View Cemetery Association.

However, according to Figure 6.1 of the City of Oakland General Plan Safety Element (2004), the site is not located within any of the dam failure inundation areas of any of these above facilities. Based on this information, the potential for flooding or inundation of the project site by dam failure is judged to be very low.

5.5.5 Tsunami and Seiche

During a major earthquake, strong waves such as tsunamis or seiches may be generated in large bodies of water and may cause damage to structures at or near the shoreline. Tsunamis are large waves generated by displacement of the seafloor by earthquakes, coastal or submarine landslides, or volcanoes. Damaging tsunamis are a potential hazard along the California coast. Most historical California tsunamis were associated with distant earthquakes (such as those in Alaska or Pacific Ocean), not with local earthquakes. However, they may occur, especially along the far northern coast of California where seafloor displacement is associated with major subduction zone earthquakes. Devastating tsunamis have not occurred in historic times in the San Francisco Bay Area.

The existing surface elevations at the project building area are about +18 feet to +21 feet (NAVD 88) and the site is located about 1/4 mile from the Oakland Inner Harbor, bounded by the Alameda Island and the Oakland bay shore. According to the Tsunami Inundation Map for Emergency Planning of the Oakland West Quadrangle (CGS, 2009), the project building area is located adjacent to but outside the mapped boundary of an identified potential tsunami inundation area. It appears the mapped boundary lies approximately at Elevation of +15 feet. In our opinion, the potential inundation hazard by a tsunami at the project building area is low.

A seiche is a wave that occurs in an enclosed basin as a result of displacement in the basin bottom, large landslides into the basin, or periodic oscillation or sloshing of the water in the basin. According to City of Oakland General Plan Safety Element (2004), the nearby by Lake Merritt, with depths greater than 2 to 3 feet only near its center, is likely too shallow to be able to generate devasting seiches. In our opinion the potential for damage due to a seiche is negligible.

5.5.6 Naturally Occurring Asbestos (NOA)

Inhalation of asbestos fibers may cause cancer. Most commonly, asbestos occurrences are associated with serpentinite and partially serpentinized ultramafic rocks.

Asbestos occurs naturally in certain geologic settings in California. Exposure and disturbance of rock and soil that contains asbestos can result in the release of fibers to the air and consequent exposure to the public. Asbestos most commonly occurs in ultramafic rock that has undergone partial or complete alteration to serpentine rock (proper rock name serpentinite) and often contains chrysotile asbestos. In addition, tremolite, another form of asbestos, can be found associated with ultramafic rock, particularly near faults. Sources of asbestos emissions include:

- Unpaved roads or driveways surfaced with ultramafic rock,
- Construction activities in ultramafic rock deposits or soils, or
- Rock quarrying activities where ultramafic rock is present.

The bedrock underlying the site is estimated to be on the order of 500 feet below the surface. In addition, no serpentinite gravels were reportedly encountered in the previous borings at the site, and no serpentinite outcrops or serpentine derived soils are identified in the hills that drain into Lake Merritt. Therefore, we consider the possibility of NOA at the site to be very low.

5.5.7 Hydrocompaction

Hydrocompaction; also referred to as hydro-collapse, is a process of settlement and resulting volume change that occurs in, low density, fine sand with minor amounts of silt and clay. Near-surface soils above groundwater encountered at the site predominately consist of medium dense silty sands and gravels or medium stiff clays; therefore, the potential for hydrocompaction or hydrocollapse is very low.

6. Discussion and Conclusions

It is our opinion that the project is feasible from a geotechnical and engineering geologic standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections.

6.1 Seismic and Geologic Hazards

The site is in a seismically active region of California. Significant earthquakes in the San Francisco Bay Area have been associated with movements within the fault zones. Earthquakes occurring along faults in the area have the potential to produce strong ground shaking at the site. Structures within the San Francisco Bay Area will experience similar shaking effects during a moderate to strong earthquake. Details discussions regarding the site geologic hazards are presented in **Section 5.0**.

Based on the results of our review and evaluation, geologic hazards at the project site consist of the potential for strong ground shaking, liquefaction, lateral spreading, landsliding, compressible fills and soils, corrosive soils, and expansive soils. Detailed measures to mitigate these geologic hazards are incorporated in our recommendations presented in **Section 7.0**.

However, the potential for surface fault offset, dynamic densification, seismically induced waves, flooding, dam inundation, hydrocompaction, NOA, and volcanic eruption at the project building area appear to be low to negligible.

6.2 Liquefaction, Lateral Spreading, and Slope Instability

As described previously, the results of our site liquefaction evaluations generally indicate the saturated, loose to medium dense sand layers of various thicknesses located both above and within the Young Bay Mud layer have a high potential for liquefying when they are subjected to an MCE earthquake event. The majority of these sand layers were encountered by borings and CPTs at the site within depths of about 30 to 40 feet (above Elevation of about -15 feet). We calculated that the MCE induced liquefaction in these sand layers would result in residual volumetric strains varying from about 1 to 4 percent and total ground surface settlements (without reduction associated with the depth of occurrence) ranging from as little as 1 inch to up to about 6-1/2 inches.

Our lateral spreading analysis results generally indicate the loose to medium dense sand layers encountered by CPTs and borings in the area adjacent to the Lake Merritt Channel at Elevations between +8 and -5 feet have a high potential to trigger ground surface lateral spreading during soil liquefaction from an MCE event. The other onsite liquefiable sand layers are considered as having low potential to trigger ground lateral spreading due to their presence in isolated thin pockets and/or being located in deeper depths in relation to the bottom of the Lake Merritt Channel.

In addition, it is our opinion the potential for slope instability during an MCE event and/or after major liquefaction event to impact the proposed building location is low to moderate. However, extensive slope failures may occur for the areas immediately adjacent to the Lake Merritt Channel if soil liquefaction and lateral spreading do occur at the site region during major earthquakes.

We recommend the proposed new building be supported on a deep foundation system that provides proper bearing support during the potential soil liquefaction events. The deep foundation should be designed to resist downdrag loads that would be imposed upon the foundations due to soil liquefaction.

In addition, the southeast side of the proposed new building foundation should also include a permanent shoring system, or a ground improvement technique should be used to mitigate the detrimental impacts from the potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

Based on the proposed building layout, we recommend the permanent shoring system along the southeast side of the proposed building (estimated lateral spreading/slope instability lateral extent) be designed to retain a 12-foot-high column of soils, assuming the loss of adjacent ground support due to slope failure. The small portion of the shoring system located further east of the area of estimated lateral spreading/slope instability should be designed to retain an 18-feet high column of soils. Our recommended lateral pressures for the shoring system designs are shown on **Plates 14** and **15**. Recommendations for ground improvement technique are presented in **Appendix I**.

The site and any new improvements not supported on deep foundations may experience total areal ground surface settlements on the order of about 1 to 4 inches with locally up to about 6-1/2 inches of settlement. In the area immediately adjacent to the channel bank, the ground settlements may be larger than the above estimates if lateral spreading occurs. Underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed to accommodate for the settlement caused by the liquefaction of the underlying supporting soils. Consideration should be given to using flexible pipe connections to mitigate potential damage from the estimated potential liquefaction-induced settlement of 4 inches at locations where the pipes are connected to pile-supported structures.

It should be noted that after a major liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as roadways and utilities may be observed and may require repair.

Alternatively, soil liquefaction ground improvement options that involve densification, drainage, reinforcement, mixing, or replacement of the liquefiable soils can be used to mitigate the site liquefaction, lateral spreading, and slope instability potentials. If needed, we can provide additional recommendations during project design, once the building and development layouts are finalized.

6.3 Compressible Soils

As described previously, the site is blanketed by sandy or clayey fills that extended to depths of about 8 to 25 feet (Elevation of about +8 feet to -5 feet). Below the surficial fill layer, Young Bay Mud was encountered to about 30 feet deep (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet deep (Elevation of about -30 feet) at the southeast side of the proposed building location. This 15- to 35-foot layer of slightly over-consolidated to normally consolidated Young Bay Mud is very soft to soft, has high moisture content and low shear strength, and is highly compressible under new additional loads. Besides the areas of the recent fills placed during the portable classroom development in 2007 or 2008, we estimated the site primary consolidation settlement due to the historical fills placed prior to 1960s should have been completed.

In our 2020 report, we recommend the proposed new building be supported on a deep foundation system that extends to a depth of at least 70 feet (or to a pile tip Elevation of -50 feet) to transfer bearing loads to the sand and clay layers below the Young Bay Mud layer. Either precast pre-stressed concrete driven piles or drilled piles, such as Case-in-Drilled-Hole (CIDH) piers and auger cast piles, can be used at the site. We note that 70- to 110-foot long, 14-inch square, precast, pre-stressed concrete driven piles were used to support the existing Art Building (built in 2005) that is also located adjacent to the Lake Merritt Channel and is about 500 feet northeast of the proposed Library Learning Resource Center site. Furthermore, the new Building Efficiency for a Sustainable Tomorrow (BEST) Center built in 2016 also is reportedly supported by 95- to 105-foot long, 14-inch square, precast, pre-stressed concrete driven piles.

The design team has decided to use a DMM ground improvement technique in combination with a shallow foundation for the LLRC building. The details of the design and specifications are presented in **Appendix J**.

In addition, to reduce the soil consolidation-induced downdrag forces on the pile foundations, we recommend the proposed project site grading activities, construction of the new surface improvements (such as exterior flatwork), and backfill for deeply buried pipelines (if any) be designed so "zero net load" will be imposed on the underlying Young Bay Mud. A "zero net load" condition can be achieved by over-excavating the fills (and possibly a portion of the Young Bay Mud if necessary) and backfilling the excavation with lightweight fill materials. Lightweight fills or concrete materials should also be used to backfill deep pipe trenches. The weight combination of new fills, at-grade new improvements, and new lightweight fills and/or concrete materials should not exceed the weight of the soils removed.

Our recommended unit weights of the fills and Young Bay Mud to be used in the "zero net load" analyses are shown in the table below. The site grade prior to the portable classroom development in 2007 or 2008 should be used in the analyses as the base line. We also recommend a groundwater level at Elevation of +8 feet be used in the analysis.

Table 6.1: Recommend Fill and Young Bay Mud Unit Weight

Soil Unit	Elevation	Unit Weight (pcf)
Existing Fill and Soil Above Groundwater	Above +8 Feet	110
Young Bay Mud Below Groundwater	Below +8 Feet	30

Alternatively, lightweight concrete materials such as Elastizell and Geofoam can be used as lightweight fills. We note that with the use of these lightweight materials below the ground water level would likely require dewatering of the excavation until sufficient weight from fills and/or structure loads are imposed to prevent potential uplift water pressures from lifting the lightweight fill materials.

6.4 Deep Mixing Method (DMM)

The design team decided to use shallow foundation and ground improvement technique in lieu of deep foundation and retaining wall system to create a more competent bearing layer for the shallow foundation and reduce the ground displacements due to lateral spreading. DMM ground improvement is one of the many techniques that is an in-situ soil treatment in which native soils or fills are mixed and blended with cement or other binders and water. The final mixed soil-binder product has enhanced engineering properties such as increased strength, lower permeability, and reduced compressibility. Two types of DMMs are used in the United States: wet mixing and dry mixing. Wet mixing involves injecting binders in slurry (wet) form to blend with the soil. Primarily single-auger, multi-auger, or cutter-based mixing processes are used with cement-based slurries to create isolated elements, continuous walls or blocks for large-scale foundation improvement, earth retaining systems, hydraulic barriers, and contaminant/fixation systems. Dry mixing uses binders in powder (dry) form that react with the water already present in the soil. Primarily single-auger dry mixing processes are used with lime and lime-cement mixtures to create isolated columns, panels, or blocks for soil stabilization as well as reinforcement of cohesive soils.

Soils best suited to DMM include cohesive soils with high moisture contents and loose, saturated, fine granular soils. DMM has also been used successfully in a wide range of less cohesive soils and fills, but it is typically not feasible in very dense or stiff materials or in ground with obstructions such as cobbles or boulders. The treated soil properties obtained by DMM reflect the characteristics of the native soil, binder characteristics, construction variables, operational parameters, curing time, and loading conditions. The generic term DMM is inclusive of other terms such as deep soil mixing (DSM) and cement deep soil mixing (CDSM). A detailed design and recommendations are presented in DMM Design Recommendations, **Appendix J.**

6.5 Preliminary Corrosion Evaluation

Corrosivity tests that include redox, pH, chlorides, sulfates, and resistivity were performed by CERCO Analytical, Inc. in Concord, California, on two representative onsite near-surface soil samples (from Boring 2019-CPT-01 at about 2-1/2 feet and 2019-CPT-03 at about 4 feet). The test results and a brief evaluation report prepared by CERCO regarding the onsite soil corrosivity are also included in **Appendix B**. We recommend these test results and the report be forwarded to the project underground contractors, pipeline designers, and foundation designers and contractors, so that they can design and install corrosion protection measures for buried concrete structures and ferrous metal. We also recommend additional testing be performed if the test results in **Appendix B** are deemed insufficient by the designers of the corrosion protection.

6.6 Construction Considerations

Excavations will be required to construct building foundations and elevator pit (if any), install utilities, and to remove locally weak or unsuitable soils. All excavations that will be deeper than 5 feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards.

If earthwork is performed during the dry season, moisture conditioning will be required to raise the onsite soil moisture contents to the engineered fill placement and compaction recommendation presented in this report. If earthwork is performed during or shortly after wet weather conditions, the moisture content of the soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

7. Recommendations

7.1 Seismic Design

The proposed new building should be designed to resist the lateral forces generated by earthquake shaking in accordance with Chapter 16 of the 2019 California Building Code (CBC). This section presents seismic design criteria according to 2019 CBC, which has adopted the seismic hazard assessment procedures provided by ASCE 7-16, Minimum Design Loads for Buildings and Other Structures. Per Section 11.6 of ASCE 7-16, structures of Risk Category I, II, and III (defined in ASCE 7-16 Table 1.5-1) should be designed according to Seismic Design Category "D".

Our liquefaction triggering hazard assessment indicated that the soils at the site are potentially liquefiable. Therefore, according to ASCE 7-16, the site is classified as Site Class F, and site response analyses, as defined in Section 21.1 of ASCE7-16, are required to calculate the design ground motions at the ground surface. Additionally, due to the large ground motion amplitudes expected at the site, ASCE 7-16 also requires the performance of a site-specific seismic hazard assessment according to Section 21.2 of ASCE 7-16. Detailed discussions of these site-specific ground motion analyses are included in **Appendix G**.

Table 7.1 tabulates the spectral ordinates of the recommended site-specific MCE_R and design response spectra per ASCE 7-16 for the ground surface. The corresponding design acceleration parameters S_{MS}, S_{M1}, S_{DS}, S_{D1}, S_S, and S_1 are tabulated in **Table 7.2**. The MCE_R and design response spectra per ASCE 7-16 at the base of the Young Bay Mud layer is provided in **Appendix G** and in Responses to CGS Comments, **Appendix I**.

Table 7.1: MCE$_R$ and Design Response Spectra per ASCE 7-16 at the Ground Surface, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)	
	Site-Specific MCE$_R$	Design Response Spectrum
0.01 (PGA)	0.584	0.389
0.03	0.639	0.426
0.05	0.694	0.463
0.075	0.763	0.508
0.1	0.831	0.554
0.15	0.969	0.646
0.2	1.11	0.738
0.25	1.24	0.829
0.3	1.38	0.921
0.304	1.39	0.927
0.4	1.39	0.927
0.5	1.39	0.927
0.75	1.39	0.927
1	1.39	0.927
1.5	1.39	0.927
1.52	1.39	0.927
2	1.06	0.704
3	0.827	0.551
4	0.733	0.489
5	0.561	0.374
7.5	0.282	0.188
8	0.264	0.176
10	0.169	0.113

Table 7.2: Design Acceleration Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.39 g
S_{M1}	2.93 g
S_{DS}	0.927 g
S_{D1}	1.96 g
S_s	1.74 g
S_1	0.66 g

7.2 Earthwork

7.2.1 Site Clearing and Preparation

The site should be cleared of all obstructions, including any existing structures and their entire foundation systems, concrete slabs-on grade, existing utilities and pipelines and their associated backfill, designated trees and their associated entire root systems, landscaping, and debris. Concrete/asphalt concrete, baserock, and trench backfill materials can be reused as new fills provided debris is removed and concrete/asphalt concrete are broken up to meet the engineered fill size requirements presented in this report.

Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with engineered fills and compacted to the requirements presented in this report. We recommend backfilling operations for any excavations to remove underground obstructions be performed under observations and testing of the project Geotechnical Engineer. After clearing, areas containing heavy surface vegetation should be stripped to an appropriate depth to remove these materials. We estimate the stripping depth to be about 6 inches. The amount of actual stripping should be determined in the field at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

7.2.2 Subgrade Preparation

Following the site clearing and preparation, soil subgrades in areas to receive engineered fill, slabs-on-grade, or pavements be scarified to a depth of at least 12 inches, moisture conditioned to approximately 3 percent above optimum water content and compacted to the requirements for engineered fills. Locally weak fills and soils, if encountered, should also be excavated and replaced, or otherwise stabilized as recommended by the project Geotechnical Engineer at the time of earthwork operations.

The prepared subgrade surface should be firm, unyielding, and kept moist during construction. The subgrades should be protected from damage caused by weather and construction traffic. If the subgrades are left exposed to weather for extended periods of time or are disturbed by construction traffic, the project Geotechnical Engineer should be consulted on the need for subgrade moisture reconditioning and/or scarifying and recompacting to eliminate shrinkage cracks and disturbances.

7.2.3 Engineered Fill Materials

Any new fills placed at the site should consist of engineered fills that meet the requirements presented in this report, except for landscaping materials which are placed on level ground. All engineered fills should have an organic content of less than 3 percent by volume and should not contain rocks or lumps larger than 4 inches in greatest dimension with not more than 15 percent larger than 2.5 inches.

Onsite soils (except for Young Bay Mud) and fills can be used as new fills. Imported fills not used as non-expansive fills should be predominantly granular, have a liquid limit less than 40 percent, and have a plasticity index not exceeding 20. Imported, non-expansive fills should consist of sub-angular to angular particles, have a plasticity index not exceeding 12, and have a significant fine content. All imported fills should not contain environmental contaminants or debris and should be non-corrosive.

7.2.4 Fill Placement and Compaction

Within the upper 5 feet of the finished ground surface, we recommend engineered fills be compacted to at least 90 percent relative compaction, as determined by ASTM D1557. Engineered fills below a depth of 5 feet should be compacted to at least 95 percent relative compaction. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 inches in uncompacted thickness.

We recommend engineered fills be moisture conditioned to approximately 3 percent above optimum water content. To achieve satisfactory compaction of fill materials, it may be necessary to adjust the water content at the time of earthwork operations. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet. To achieve satisfactory compaction of onsite excavated soils from near or below the existing groundwater level will require drying at the time of construction.

7.2.5 Trench Backfill and Pipe Bedding

To prevent imposing additional load to the underlying soils and to reduce potential settlement along deeply buried pipelines, trench backfill materials should be properly selected so that the unit weight of backfill materials is less or equivalent to the unit weight of the removed onsite soil materials (zero net load). Considerations should be given to increasing the hydraulic gradient of gravity flow pipes to account for potential soil differential consolidation settlements below the pipes and also using flexible connections for all pipes.

Pipeline trenches should be backfilled with engineered fills placed in lifts of approximately 8 inches or less in uncompacted thickness. Thicker lifts can be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only; jetting is not permitted. Onsite soils, and onsite and imported fills when used for trench backfill should be compacted to at least 90 percent relative compaction. Imported sands and aggregate bases when used for trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction.

Sand or gravel backfilled trench laterals that extend from irrigated landscaped areas, such as lawns or planting strips, toward pavements, exterior slabs, and building foundations, should be plugged with onsite or imported clayey soils, low strength concrete, or sand-cement slurry mixture below the edges of pavements and exterior slabs, and under perimeters of the foundations. The plugs for the trench laterals should be at least 24 inches thick, extend at least 24 inches beyond the trench walls, and extend from the bottom of the trench to the top of the sand or gravel backfills.

Bedding material should consist of Caltrans Class 2 Aggregate Base or Aggregate Base Course (ABC) meeting the requirements of Section 26 of Caltrans Standard Specifications. All bedding material shall have 3/4-inch maximum aggregate size and be free from organic or vegetable matter, lumps, or balls of silt/clay, or any other deleterious matter. ABC material shall conform to the following gradations when tested in accordance with ASTM C136 or California Test 202.

Table 7.3: Aggregate Base Course Gradation Requirements

Sieve Size (Square Openings)	Percentage by Weight Passing Sieves
1 inch Screen	100
3/4 inch Screen	90 to 100
No. 4 Sieve	35 to 60
No. 30 Sieve	10 to 30
No. 200 Sieve	2 to 9

In addition to the above requirements, all material used shall conform to the following quality requirements:

- Resistance (R-Value) with the minimum test results of 78;
- Sand Equivalent with the minimum test result of 22; and
- Durability Index with the minimum test result of 35.

7.2.6 Exterior Flatwork

We recommend exterior slabs, such as sidewalks and patios, be placed directly on the properly prepared subgrades in accordance with the recommendations presented in this report. Eliminating aggregate base, gravel, or crushed rock base beneath exterior slabs will reduce the potential for landscape irrigation water to seep through the granular materials and cause the underlying soil subgrades to saturate or pipe. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 percent above laboratory optimum moisture (ASTM D-1557).

The expansive clayey soils and fills at the site could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as doors that open outward. We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, considerations should be given to installing of #4 bars spaced at approximately 18 inches on center in both directions. Both score joints and expansion joints can be used to control cracking and allow for expansion and contraction of the concrete slabs.

We recommend appropriate flexible, relatively impermeable fillers be used at all expansion and cold joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; if used, the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced.

It should be noted, movements or failures of the exterior slabs should be anticipated after major liquefaction events. Repair of the exterior slabs, as well as site regrading, may be needed after the events.

7.2.7 Surface Drainage and Landscaping

We recommend exaggerated positive surface gradients that take into account potential differential ground settlements be provided adjacent to structures and for pavements to direct surface water toward suitable discharge facilities. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge into appropriate facilities. Ponding of surface water must not be allowed adjacent to structure foundations and exterior slabs, adjacent to pavements, at the top or adjacent to retaining walls.

To reduce moisture changes in the soils below and adjacent to structure foundations and exterior slabs, landscaping and irrigation systems should be designed and installed in a uniform and systematic manner as equally as possible on all sides of the foundations and adjacent to exterior slabs. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the trees from improvements to reduce the adverse effects from the tree roots. We recommend that drought resistant plants and low flow/drip irrigation watering systems be used. All irrigation systems should be regularly maintained and inspected for leakage. Over-watering must be avoided.

For bio-retention swales and basins (if planned), where they are located within 10 feet of infrastructure improvements (such as structure foundations, exterior flatwork, and pavements), we recommend they be lined with a relatively impermeable membrane to reduce water seepage and the potential for damage to other infrastructure improvements (such as foundations, exterior slabs, and pavements). The membrane can consist of a layer of STEGO Wrap 15-mil or equivalent installing below and along the sides of these facilities to direct the collected water into subdrain pipes. The membrane should be lapped and sealed in accordance with the manufacturer's requirements, including sealing joints where pipes penetrate the membrane.

The bio-treatment soil mix materials within swales and basins should be considered as having no lateral load resistant. We recommend the sidewall slopes of the swales and basins not to exceed 2:1 (horizontal to vertical) to reduce potential vertical and lateral movements of surrounding ground surface. In addition, we recommend either improvements (foundations, exterior slabs, and pavements) be setback beyond an imaginary 1:1 (horizontal to vertical) plane projected upward from the bottom edges of the swales and basins or the affected areas of the improvements be supported by deepening foundations or edges. Alternatively, properly designed below-grade enclosure structures can be used to build the swales and basins and to retain surrounding ground and improvements.

7.2.8 Construction During Wet Weather Conditions

If construction proceeds during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation, placement of onsite soil as structural fill might not be possible. A geotechnical engineer can provide alternative wet weather construction recommendations in the field at the time of construction, if appropriate.

7.3 Building Foundation System

The proposed new building foundation should be designed to provide proper bearing supports during the potential soil liquefaction events. Two foundation system are proposed: 1) deep foundation in combination with a permanent shoring system on the southeast side of the proposed new building foundation and 2) shallow foundation in combination with DMM ground improvement technique to mitigate the detrimental impacts from the potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

Based on the proposed building layout, we recommend the permanent shoring system along the southeast side of the proposed building (estimated lateral spreading/slope instability lateral extent) be designed to retain a 12-foot-high column of soils, assuming the loss of adjacent ground support due to slope failure. The small portion of the shoring system located to further east of the estimated lateral spreading/slope instability lateral extent should be designed to retain an 18-foot high column of soils. Our recommended lateral pressures for the shoring system designs are shown on **Plates 14** and **15**.

We recommend the proposed new building be supported on a deep foundation system that extends to a depth of at least 70 feet (or to a pile tip Elevation of -50 feet) to transfer bearing loads to the sand and clay layers below the Young Bay Mud layer. Either precast pre-stressed concrete driven piles or drilled piles, such as Case-in-Drilled-Hole (CIDH) piers and auger cast piles, can be used at the site. The deep foundation should also be used to support any exterior elements that are considered essential parts of the building. Structural slabs should be designed to span between pile foundations. Detailed descriptions of the ground improvement technique are presented in DMM Design and Recommendations, **Appendix I**.

The deep foundation should be designed to resist downdrag loads that would be imposed upon the foundations due to soil liquefaction. Consideration should also be given to using flexible pipe connections to mitigate potential damage from the estimated potential liquefaction-induced settlement of 4 inches at locations where the pipes are connected to pile-supported structures.

Structures not supported on deep foundations may experience total areal ground surface settlements on the order of about 1 to 4 inches with locally up to about 6-1/2 inches of settlement. In the area immediately adjacent to the channel bank, the ground settlements may be larger than the above estimates if lateral spreading occurs.

7.3.1 Pile Axial Load Capacity

The new building can be supported by a deep foundation system that develops its load carrying capacity from soil friction/adhesion within the competent sand and clay layers below the Young Bay Mud. Either precast pre-stressed concrete driven piles or drilled piles, such as Case-in-Drilled-Hole (CIDH) piers and auger cast piles, can be used at the site

Piles should be at least 14 inches in square or diameter, extend to a depth of at least 70 feet (or to a pile tip Elevation of -50 feet), and have a center-to-center spacing of at least 3 times the pile dimension. The actual design lengths of the piles should also be determined using an ultimate skin friction of 1,500 psf (pounds per square feet) for the pile section located below the bottom of the Young Bay Mud layer. As indicated on **Plates 7** through **11**, the bottom of the Young Bay Mud layer is located at about 30 feet deep (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet deep (Elevation of about -30 feet) at the southeast side of the proposed building location. The pile section within and above the Young Bay Mud layer should be neglected in design for axial loading. The allowable axial capacity should be calculated by dividing the ultimate axial capacity by the factors of safety provided in the table below or the project structural design over strength factor (if applicable). Eighty percent (80 percent) of the skin friction value can be used to resist uplift.

Table 7.4: Recommended Factors of Safety for Axial Loading of Pile Foundation

Load Condition	Factor of Safety
Dead Load	3
Dead plus Live Loads	2
Total Loads (including wind or seismic)	1.5

The piles should also be designed to resist downdrag loads that would be imposed upon the foundations due to potential liquefaction of the isolated sand layers above Elevation of about -15 feet. We recommend an average negative skin friction of 650 psf be included along the upper 35 feet of the pile shaft to account for the potential liquefaction-induced downdrag forces from about 15 feet of fills, and 20 feet of Young Bay Mud with liquefied sand lenses. This value should be subtracted from the ultimate pile axial capacity.

A viscous bituminous coating can be applied on the upper 35 feet of pile shaft to reduce the downdrag loads. A fifty percent (50 percent) reduction is applicable to the above downdrag value when bituminous coating is used.

Static total and differential settlements of the pile supported structure are estimated to be insignificant (i.e., less than 0.5 inch) and within tolerable limits for the proposed structure. Seismic settlement of the pile is estimated to be less than 1 inch assuming the pile is designed to resist the downdrag force only using pile skin friction.

Regardless of the calculated pile lengths to meet axial capacity demands, a minimum of 35 feet of pile embedment is also needed to provide pile "fixity" to resist lateral loading based on the LPILE analysis results.

7.3.2 Pile Lateral Load Capacity

We evaluated pile lateral load capacities using the computer program LPILE (Ensoft, Ver. 2017.11.01) to model subsurface soils as a series of discrete springs with nonlinear behavior. Our analyses assumed a 70-foot long, 14-inch square elastic pile with a design concrete strength of 5,000 pounds per square inch (psi). The estimated flexural rigidity (EI) of the pile was reduced by fifty percent (50 percent) to account for an assumed twenty percent (20 percent) of pile section concrete crack in the direction of lateral loading. Pile axial loads were not included in our analyses.

Four (4) different soil profiles (1, 2, 3A & 3B) along the Cross-Section A-A' (**Plate 7**) were established in our analysis models based on the idealized subsurface soil conditions at the site. The locations of these profiles are shown on **Plate G-1** for reference, included in **Appendix G**.

Both Profiles 1 and 2 have the same soil stratigraphy, besides the thickness of the Young Bay Mud layer. An additional saturated highly liquefiable sand layer was also included in Profiles 3A and 3B between the surficial fill layer and the underlying Young Bay Mud layer. In Profile 3B, this sand layer was assumed to be liquefied during earthquake events. A design groundwater table at an elevation of +8 feet were used for all profiles. The detailed soil stratigraphy and engineering properties used in our analyses are in the tables below.

Table 7.5: Soil Engineering Properties for Profile 1

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psf)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus, k (pci)
				Top	Bottom	Top	Bottom		
0 to 12 feet	Sandy Fill	Reese (Sand)	120	-	-	-	-	32	90
12 to 30 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	668	0.02	0.01	-	-
Below 30 feet	Sand and Clays	Reese (Sand)	66	-	-	-	-	40	125

Table 7.6: Soil Engineering Properties for Profile 2

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psf)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus, k (pci)
				Top	Bottom	Top	Bottom		
0 to 12 feet	Sandy Fill	Reese (Sand)	120	-	-	-	-	32	90
12 to 43 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	786	0.02	0.01	-	-
Below 43 feet	Sand and Clays	Reese (Sand)	66	-	-	-		40	125

Table 7.7: Soil Engineering Properties for Profile 3A

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psi)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus, k (pci)
				Top	Bottom	Top	Bottom		
0 to 7 feet	Sandy Fill	Reese (Sand)	120	-		-	-	32	90
7 to 18 feet	Highly Liquefiable Sands	Reese (Sand)	46	-	-	-	-	33	60
18 to 41 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	668	0.02	0.01	-	-
Below 41 feet	Sand and Clays	Reese (Sand)	66	-		-		40	125

Table 7.8: Soil Engineering Properties for Profile 3B

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psi)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus , k (pci)
				Top	Bottom	Top	Bottom		
0 to 7 feet	Sandy Fill	Reese (Sand)	120	-		-	-	32	90
7 to 18 feet	Highly Liquefiable Sands	Liquefied Sand (Rollins)	46	-	-	-	-	-	-
18 to 41 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	668	0.02	0.01	-	-
Below 41 feet	Sand and Clays	Reese (Sand)	66	-		-		40	125

Both free and fixed pile head conditions were examined in our analyses. Our estimated lateral loads for 1/4-inch, 1/2-inch, and 1 inch of lateral displacements at pile heads for each pile head condition and loading case (1 through 6) are presented in the tables for each soil profile. The calculated pile head deflection, bending moment, and shear force versus embedment depth are presented in **Appendix H**. It should be noted that no factor of safety was applied to the estimated loads or deflections.

Table 7.9: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 1

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	9	25
2	Free	0.5	13	44
3	Free	1.0	21	77
4	Fixed	0.25	20	71
5	Fixed	0.5	33	125
6	Fixed	1.0	53	221

Table 7.10: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 2

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	9	25
2	Free	0.5	13	44
3	Free	1.0	21	77
4	Fixed	0.25	20	71
5	Fixed	0.5	33	125
6	Fixed	1.0	53	221

Table 7.11: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 3A

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	8	25
2	Free	0.5	13	43
3	Free	1.0	21	78
4	Fixed	0.25	20	70
5	Fixed	0.5	33	124
6	Fixed	1.0	53	220

Table 7.12: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 3B

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	8	23
2	Free	0.5	12	39
3	Free	1.0	19	67
4	Fixed	0.25	17	58
5	Fixed	0.5	26	94
6	Fixed	1.0	37	146

Where competent subgrade soils exist, a soil passive resistance equal to an equivalent fluid weighing 350 pcf (pounds per cubic foot), which acts against the vertical face of the pile cap and grade beam (assumes a deflection of approximately 1/2 inch), can also be used in conjunction with the above estimated pile shaft lateral load capacities. A higher soil passive resistance equal to an equivalent fluid weighing 450 pcf can be used for the portion of the surficial fills that is properly over-excavated and re-compacted as engineered fills. The upper 12 inches of soils should be neglected in passive resistance design unless they are confined by a pavement or slab. This value can be used without reduction if the pile shaft lateral load capacity is also based on a compatible 1/2 inch pile head displacement. Any portion of the pile cap, grade beam and shaft located above an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent utility trenches should be ignored in the passive resistance design.

For closely spaced piles, the shear planes in the soil overlap and the lateral resistance for a pile within the group is less than that of a single pile. We note that the leading piles are generally less impacted by group effects and tend to draw higher loads. To account for the reduction of soil resistance because of group effects, we recommend multiplying the lateral loads by the reduction factors provided in the table below**.** Reduction factors, or p-multipliers, are a function of center-to-center spacing where D is the pile diameter. P-multipliers should be applied to trailing piles in the direction of loading.

As an example, a 1 by 6 pile row with a center-to-center spacing of 6 diameters and loaded in the direction parallel to the pile row would use a p-multiplier of 1.0 for the lead pile and 0.7 for all trailing piles. The same group loaded perpendicular to the pile row would use a p-multiplier of 1.0 for all piles. Linear interpolation may be used for other pile spacing.

Table 7.13: Reduction Factors for Pier Lateral Load Capacity

Center-to-Center Spacing	p-Multiplier
8D	1.0
6D	0.7
4D	0.4
3D	0.3

7.3.3 Pile Construction

We recommend that the installation or excavation of all piles be performed under the direct observation of the project Geotechnical Engineer to confirm that the piles are founded in suitable materials and constructed in accordance with the recommendations presented herein. All piles should be installed or constructed vertically to their design tip elevations at the specified locations to develop adequate vertical pile capacities.

The pile driving hammer and the methods of handling, picking, and setting the piles should be properly selected by the contractor and reviewed by both the project Structural Engineer and Geotechnical Engineer. It is possible for a very large or very small hammer to cause damage to the pile it is driving. The pile driving criteria should be stablished by the Contractor in conjunction with the project Geotechnical Engineer by performing a wave equation analysis (WEAP) after selections of type and size of pile and pile hammer have been finalized, and prior to pile installation.

In addition, we recommend an indicator pile program be performed for the project, which consists at least 5 indicator piles and Pile Dynamic Analyzer (PDA) tests. The indicator piles should be performed in close proximity to the exploratory borings and CPTs to determine the lengths for production piles and driving resistance of the piles, as well as to verify the pile capacities and the anticipated soil profile across the site. The indicator piles should be at least 10 feet longer than the anticipated design length of the production piles. The indicator pile program should be conducted using the same equipment and same installation methods that will be used for installing the production piles. Due to the potential for encountering hard driving within dense sands below the Young Bay Mud layer, we recommend that the moment resisting reinforcement in the indicator piles be deepened 10 to 20 feet in anticipation of possible pile cutoffs.

The project Geotechnical Engineer should observe the driving of all indicator and production piles and in no case should driving be terminated without the approval of the project Geotechnical Engineer. The project Geotechnical Engineer should evaluate the allowable capacity of any piles driven shorter than their anticipated lengths.

We recommend predrilling through the existing fill layer be performed at driven pile locations to avoid obstructions and potential damage to the piles. The pre-drilled holes should have a diameter less than the 3/4 the diagonal width of the piles.

7.3.4 Deep Mixing Method (DMM) Ground Improvement

Several alternatives were considered for mitigating the lateral spread hazard at the planned building site, including installation of a retaining wall and the deep mixing method (DMM) beneath the building footprint. Considering the high seismic demand, presence of shallow liquefiable soils and soft Young Bay Mud, proximity to the Lake Merritt Channel, and constraints from the PG&E easement on the north side of the planned building, it is our experience and opinion that continuous grids of deep mixed shear walls are the most suitable, robust, and cost-effective technique to mitigate the lateral spread hazard at the planned building site. The grids of deep mixed shear walls will provide support for shallow foundation systems for seismic loading and transfer bearing loads deeper to the medium dense to very dense sands and stiff to hard clays, reducing total and differential building settlements. In addition, we recommend using structural slabs to span between DMM deep mixed shear walls, assuming that the untreated soils within the grid walls may still develop post-liquefaction reconsolidation settlements below slabs. The deep mixed shear walls will also affect the composite ground response to horizontal ground motions. This section presents a brief overview of the deep mixing method (DMM), our design approach, DMM design properties, and results of our evaluation process, including results of seismic stability analyses. Seismic design parameters incorporating the composite response of the deep mixed zone are presented in **Appendix I**, herein.

7.3.5 Building Ground Interior Slab

The interior ground slab should consist structural slabs that are designed to span between pile foundations. The slab should be underlain by an at least 12 inches of properly compacted engineered fills that extend at least 3 feet beyond the foundation footprints.

If migration of water vapor through interior slab is undesirable, we recommend a vapor retarder and an underlying 4-inch layer of ¾-inch, clean, crushed, uniformly graded gravel/drain rock be placed between the bottom of the slab and the recommended engineered fill layer. The gravel/drain rock layer can be considered as part of the non-expansive engineered fill layer. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent provided the equivalent satisfies the following criteria: a permeance less than 0.01 perms as guided by ACI 302.2R, Class A strength as determined by ASTM E1745, and a thickness of at least 15 mils. Installation of the vapor retarder, including protrusions where pipes or conduit penetrate the membrane, should conform to ASTM E1643 and the manufacturer's requirements. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane. The subgrade below the slab should be property prepared, firm, and non-yielding. All foundation excavations should be kept moist and free of loose soils and standing water prior to concrete placement.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. A proper water/cement ratio should be determined by the foundation designers for the slabs to reduce vapor transmitting if need. We recommend the foundation designer determine if corrosion protection is needed for the foundation concrete and reinforcing steel. The corrosivity test results of onsite soil samples and a brief evaluation report by others are included in **Appendix B**; the foundation designer should determine if additional testing is needed. In addition, the foundation designers should provide recommendations to reduce the potential for differential concrete curing if necessary.

7.4 Retaining Walls

Retaining walls can be supported on spread footing or pile foundations. Fill placed behind walls should conform to the engineered fill materials, and fill placement and compaction recommendations. If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced.

For retaining walls not to be supported on piles, a "zero net load" approach should be used for the wall design and construction to reduce the soil consolidation settlement below the walls. Detailed descriptions of the approach are provided in **Section 6.3**. It should be noted that walls located within the area of potential ground lateral spreading/slope instability (east of the dashed line) may potentially experience large vertical and lateral movements during major earthquake events.

7.4.1 Lateral Loads

Any walls that retain soils should be designed to resist both lateral earth pressures and any additional lateral loads caused by roadway surcharging, earthquake loading, and hydrostatic pressure if the walls are located below groundwater table. Considerations should be given to applying waterproofing to backside of the wall to reduce water/vapor transmission and efflorescence forming on the front wall face.

We recommend that any undrained unrestrained walls are free to deflect or rotate be designed to resist an equivalent fluid pressure of 85 pounds per cubic foot (pcf). Undrained restrained walls should be designed to resist an equivalent fluid pressure of 100 pcf. This assumes walls with level backfills. Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 1/3 the anticipated surcharge load for unrestrained walls, and 1/2 the anticipated surcharge load for restrained walls.

If back-drainage is provided behind the walls, we recommend that drained unrestrained walls be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot (pcf). Drained restrained walls should be designed to resist an equivalent fluid pressure of 75 pcf. These recommended drained lateral pressures assume walls are fully-back drained to prevent the build-up of hydrostatic pressures. This can be accomplished by using ½ to ¾ inch crushed, uniformly graded gravel entirely wrapped in filter fabric, such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 8 inches wide and extend from the base of the wall to within 12 inches of the finished grade at the top (Caltrans Class 2 permeable material (Section 68) may be used in lieu of gravel and filter fabric). A 4-inch diameter, perforated pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to suitable discharge facilities. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade and are able to function properly. As an alternative to using gravel, pre-fabricated drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal).

For walls that are higher than 6 feet, we recommend the walls also be designed to resist a uniform lateral pressure of 38H pcf for both unrestrained and restrained wall conditions based on the ground acceleration from a design basis earthquake (Seed and Whitman, 1970; Atik and Sitar, 2007), where H is the height of the retaining portion of the walls. This seismic induced earth pressure is in addition to the pressures noted above. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads.

7.4.2 Wall Footing Foundation

Retaining walls can be supported by conventional spreading footings that are designed for "zero net load" and bear on competent onsite fills. Over-excavation and re-compaction of any weak fills below the footings may be required due to the heterogenous nature of the onsite existing fills. The bottom of the footings should be at least 12 inches wide and founded at least 24 inches below lowest adjacent finished grade. Deeper embedment will be required for footings that are located adjacent to or near top of slopes. Portion of the footings located within 10 feet (as measured laterally) of the slope face should be ignored in both vertical and passive resistance design.

Footings located adjacent to other footings or utility trenches should also bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Alternatively, the foundation reinforcing could be increased to span the area defined above assuming no soil support is provided. Our recommended allowable spread footing bearing pressures are provided below. These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Table 7.14: Allowable Wall Spread Footing Bearing Pressures

Load Condition	Allowable Bearing Pressure (psf)	Factor of Safety
Dead Load	"Zero Net Load"	-
Dead plus Live Loads	"Zero Net Load"	-
Total Loads (including Wind or Seismic)	3,000	1.5

Resistance to lateral loads can be provided by friction along the base of footings and by passive pressures acting on the sides of footings. An allowable friction coefficient of 0.3 times the dead load (a factor of safety of 1.5) may be used to evaluate the allowable frictional resistance along the bottom of footings. Where the footing is poured neat against competent subgrade soils, a passive pressure equal to an equivalent fluid pressure of 350 pounds per cubic foot (pcf) can be used for lateral load resistance against the sides of footings perpendicular to the direction of loading. The upper 12 inches of soils should be ignored, unless they are confined by pavement or slab. This passive resistance should be considered as an ultimate value (a factor of safety of 1.0) and assumes a deflection of approximately 0.5 inch to fully mobilize the passive resistance.

7.5 Additional Geotechnical Services

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations. We recommend that Fugro be also retained to provide geotechnical services during earthwork operation and foundation installation to observe compliance with the design concepts, specifications, and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered.

8. Limitations

The opinions, conclusions, and recommendations presented in this report are based on our reviews of available geologic and geotechnical data, maps, reports, our site subsurface exploration and laboratory testing results, our engineering analysis results, and information provided by others. Our opinions, conclusions, and recommendations are solely professional opinions and were made in accordance with generally accepted local and current geotechnical engineering principles and practices. We make no warranty, either express or implied.

Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed and at the time when services were conducted; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

This report has been prepared for the exclusive use of Peralta Community College District and their consultants for specific application to the proposed Laney College Library Learning Resource Center in Oakland, California as described herein. If there are any changes in the nature, design or location of the project, as described in this report, or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing.

Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for use of segregated portions of this report without our prior consultation.

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List of Plates

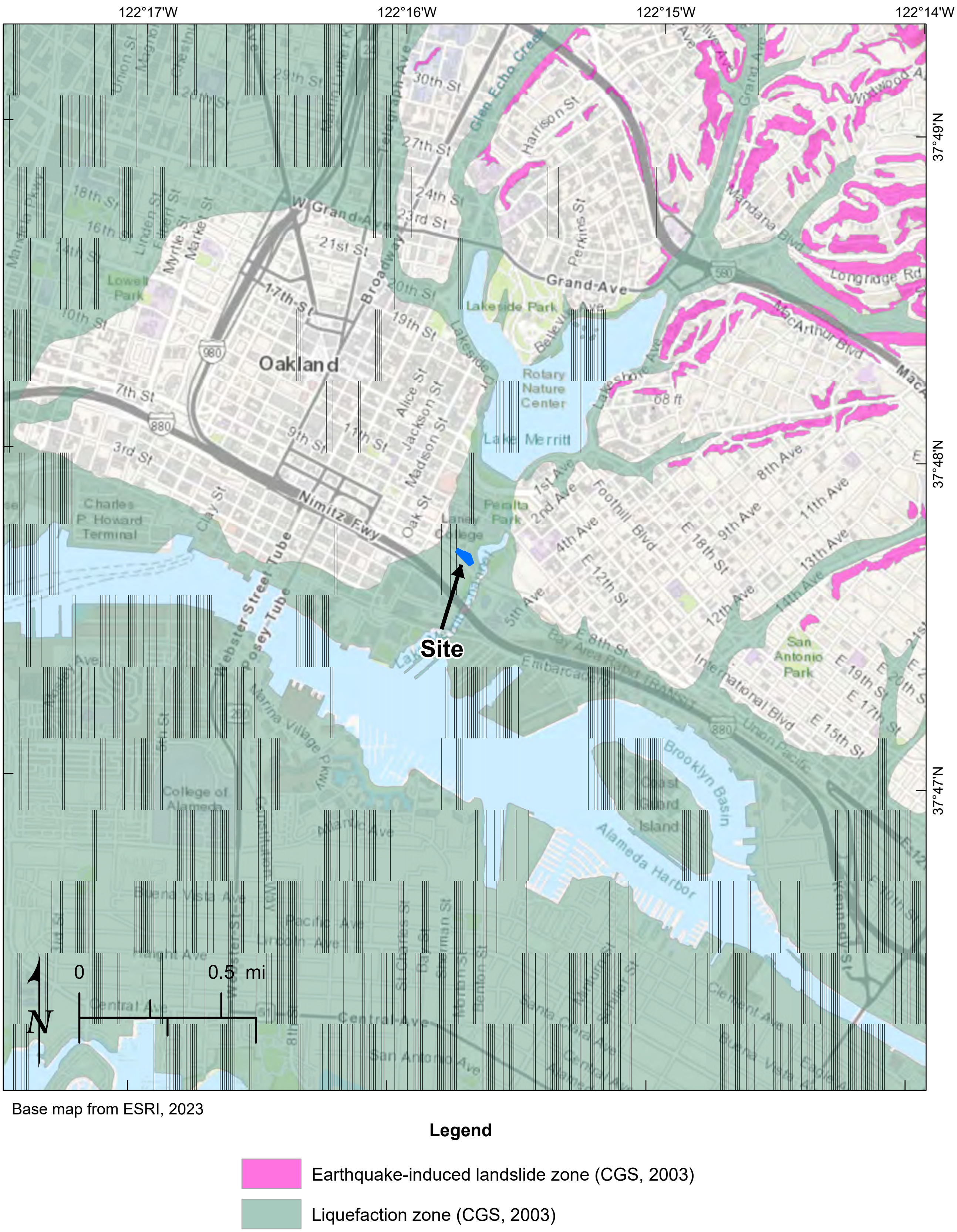
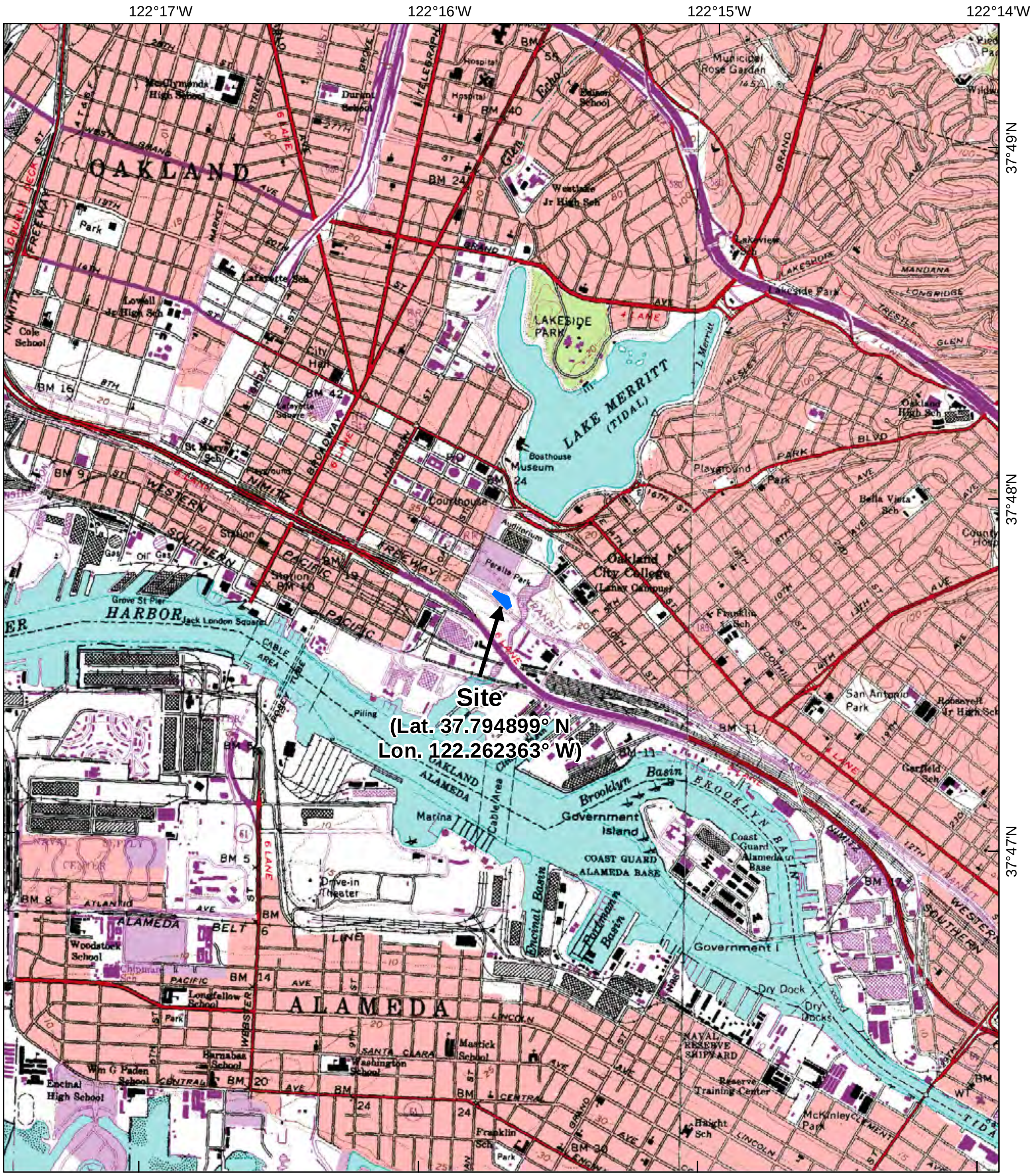


Plate-6: CGS Seismic Hazard Zone Map

Base map from Esri, 2023.

Plate-1: Vicinity Map



Base map USGS Oakland West and Oakland East 1:24,000-scale topographic quadrangles, 1980.

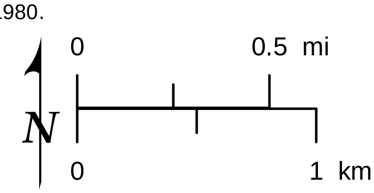
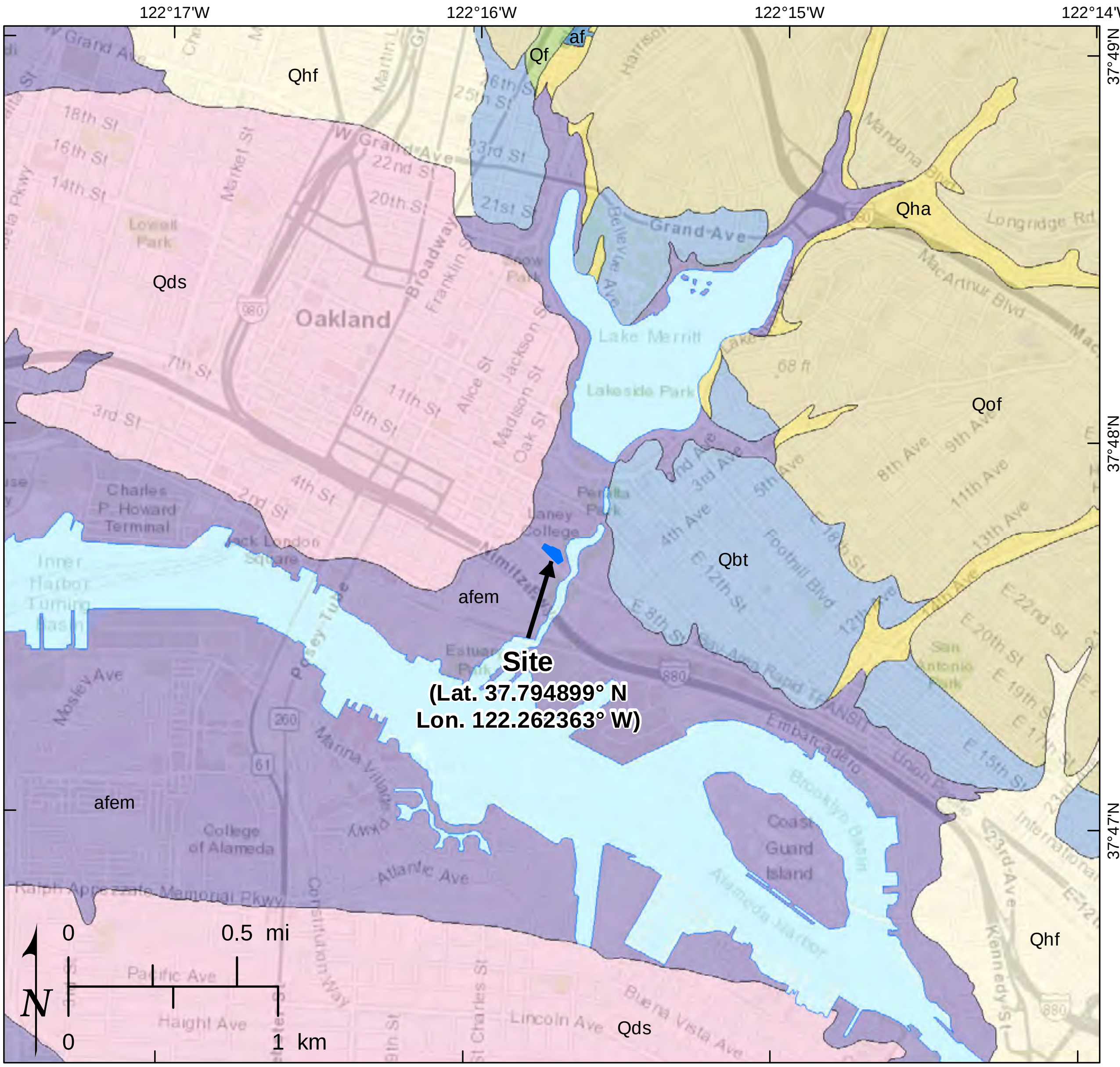


Plate-2: Topographic Site Map

Aerial imagery from Bing Maps. Topo contours provided by CSW/Stuber-Stroeh, April 2019. Proposed building location provided by Noll and Tam Architects, January 2020.

Figure 3: Site Plan

FUGRO



Base map USGS Oakland West and Oakland East 1:24,000-scale topographic quadrangles.Geologic map: Witter et al, 2006.

Legend

HISTORICAL

af	Artificial fill
afem	Artificial fill over estuarine mud

HOLOCENE

Qhf	Alluvial fan deposits
Qha	Alluvial deposits, undifferentiated

HOLOCENE TO LATEST PLEISTOCENE

Qds	Dune sand
Qf	Alluvial fan deposits

PLEISTOCENE

Qbt	Bay terrace deposits

EARLY TO LATE PLEISTOCENE

Qof	Alluvial fan deposits

Plate-4: Quaternary Geologic Map

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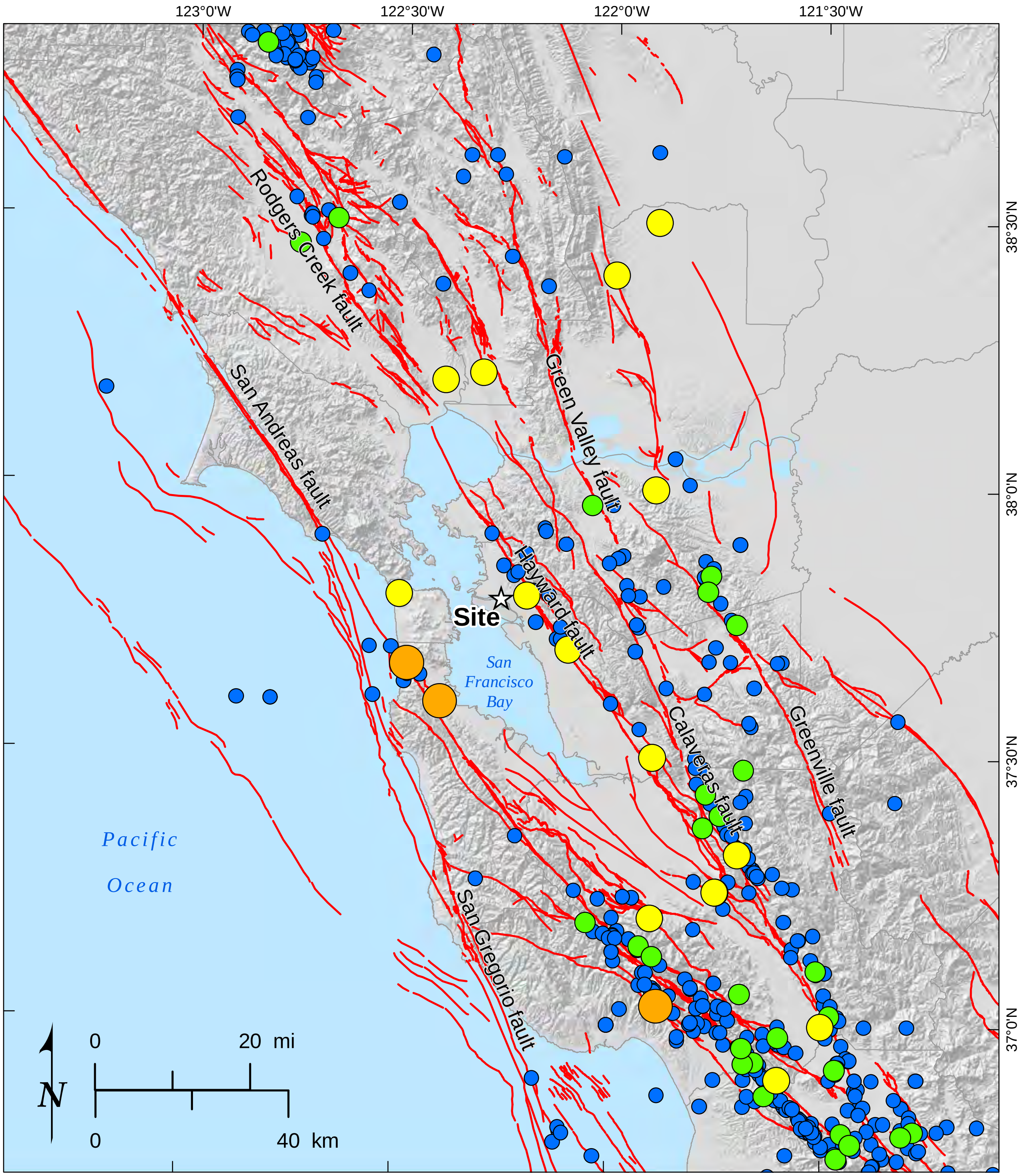

Legend

●	4.0 - 4.9
●	5.0 - 5.9
●	6.0 - 6.9
●	>=7.0

Seismicity by Magnitude (1910-2018 Earthquake Catalog from NCEDC and 1568-2009 Significant Earthquake data from USGS)

Faults (USGS, Quaternary Fault and Fold Database, 2017)

REGIONAL FAULT AND SEISMICITY MAP PLATE 5

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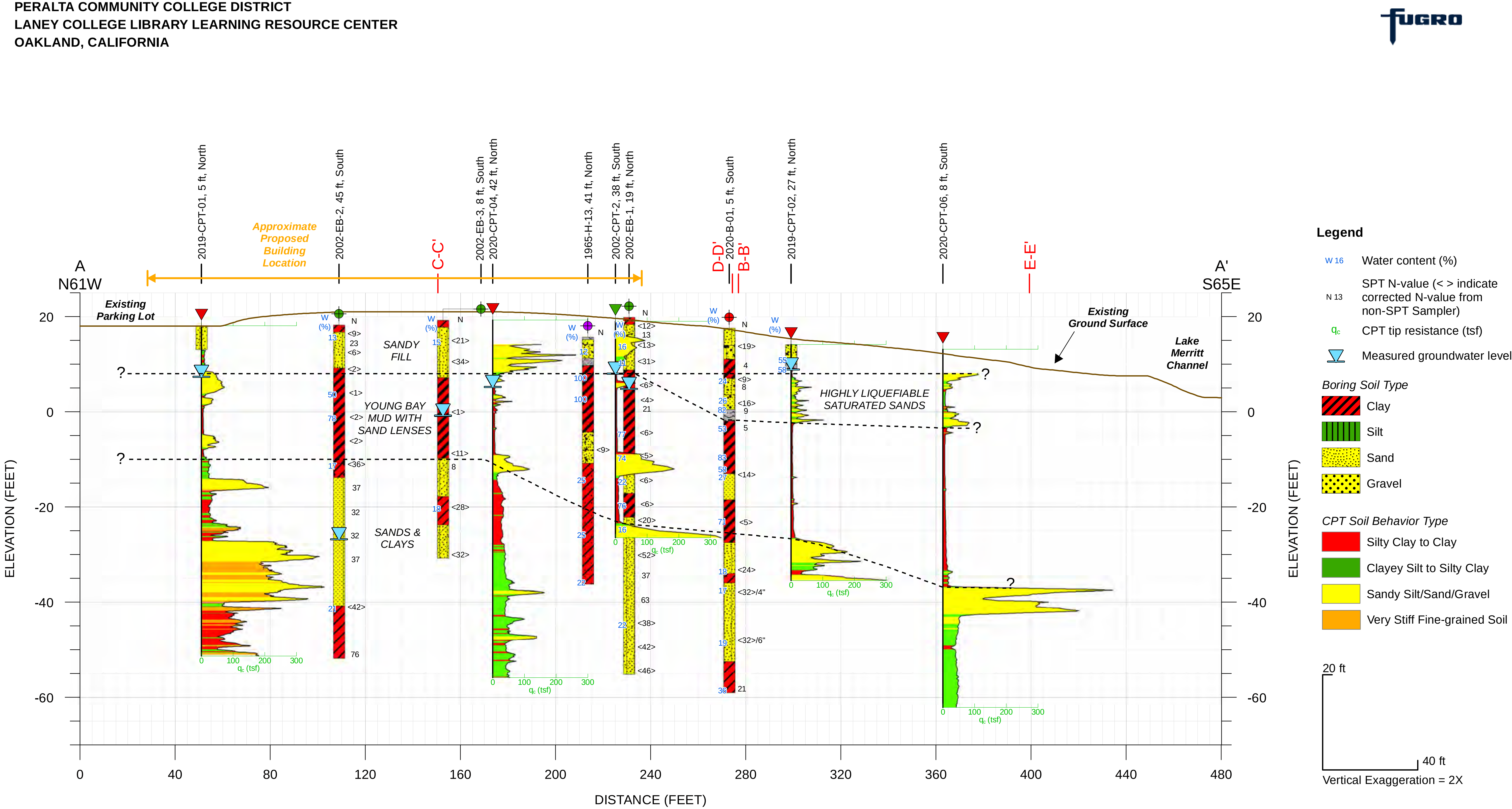

CROSS SECTION A-A'

PLATE 7

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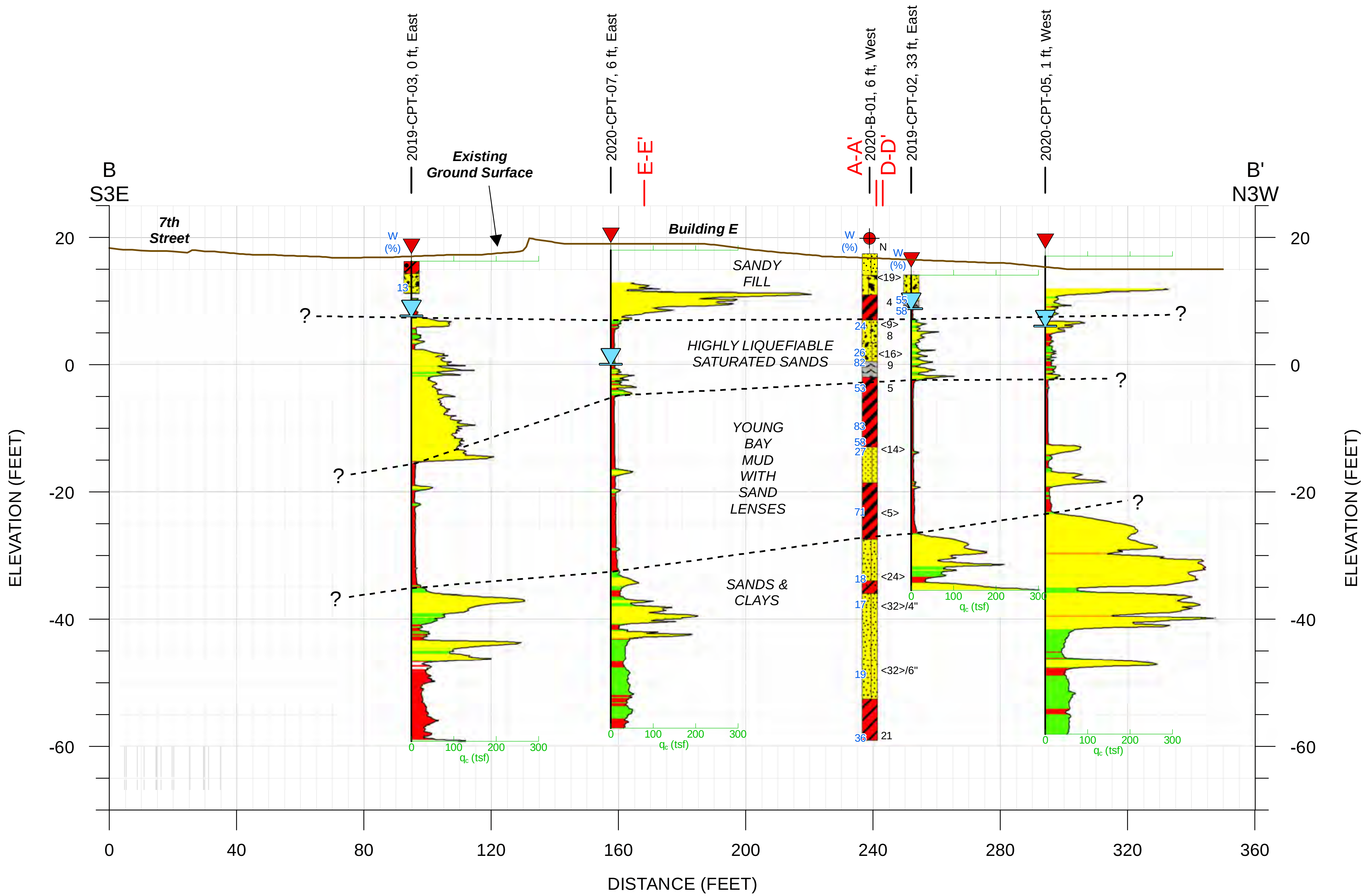

Legend

- W 16 — Water content (%)
- N 13 — SPT N-value (< > indicate corrected N-value from non-SPT Sampler)
- q_c — CPT tip resistance (tsf)
- Measured groundwater level

Boring Soil Type

- Clay
- Silt
- Sand
- Gravel

CPT Soil Behavior Type

- Silty Clay to Clay
- Clayey Silt to Silty Clay
- Sandy Silt/Sand/Gravel
- Very Stiff Fine-grained Soil

20 ft
40 ft
Vertical Exaggeration = 2X

CROSS SECTION B-B'

PLATE 8

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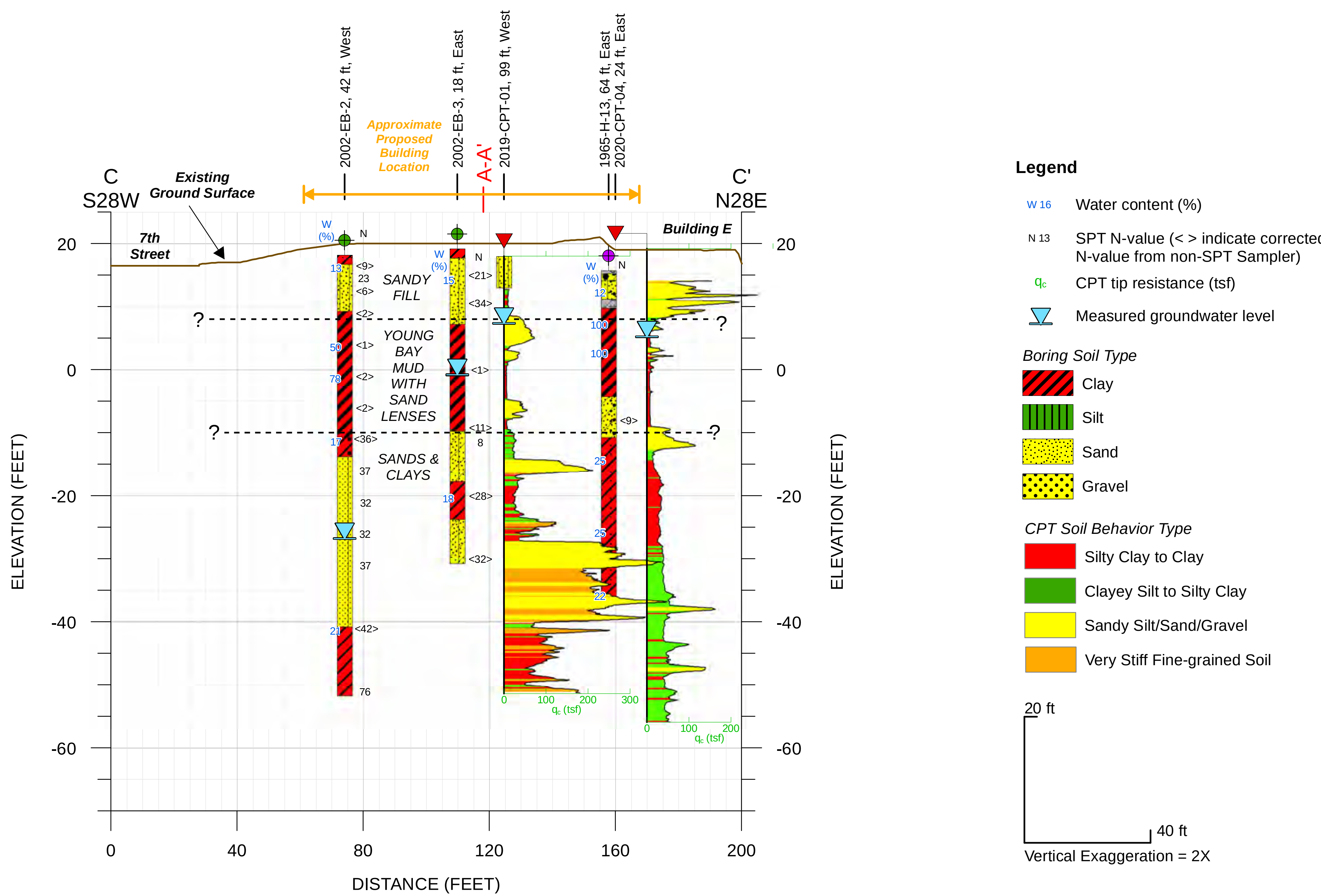

CROSS SECTION C-C'

PLATE 9

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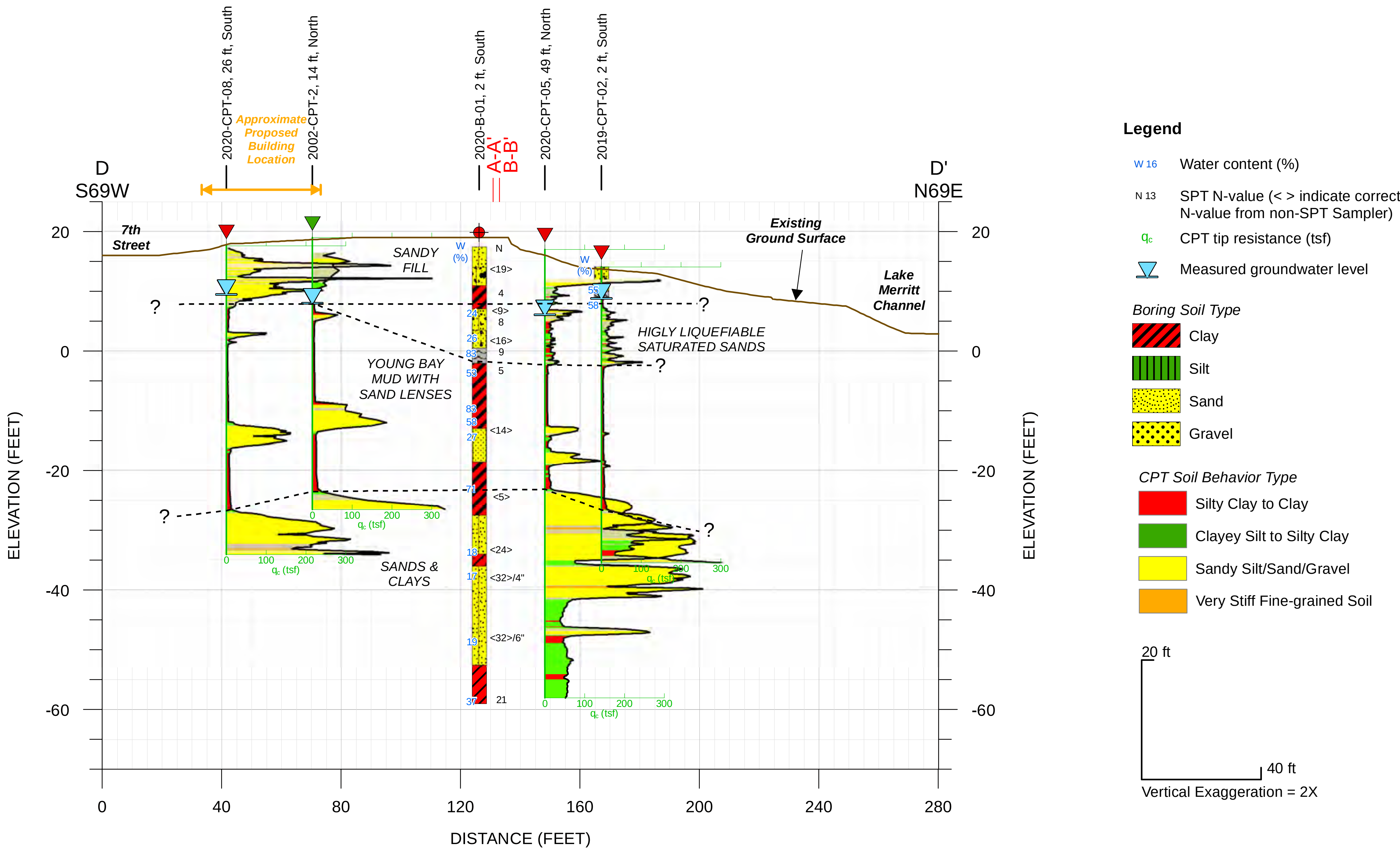

CROSS SECTION D-D'

PLATE 10

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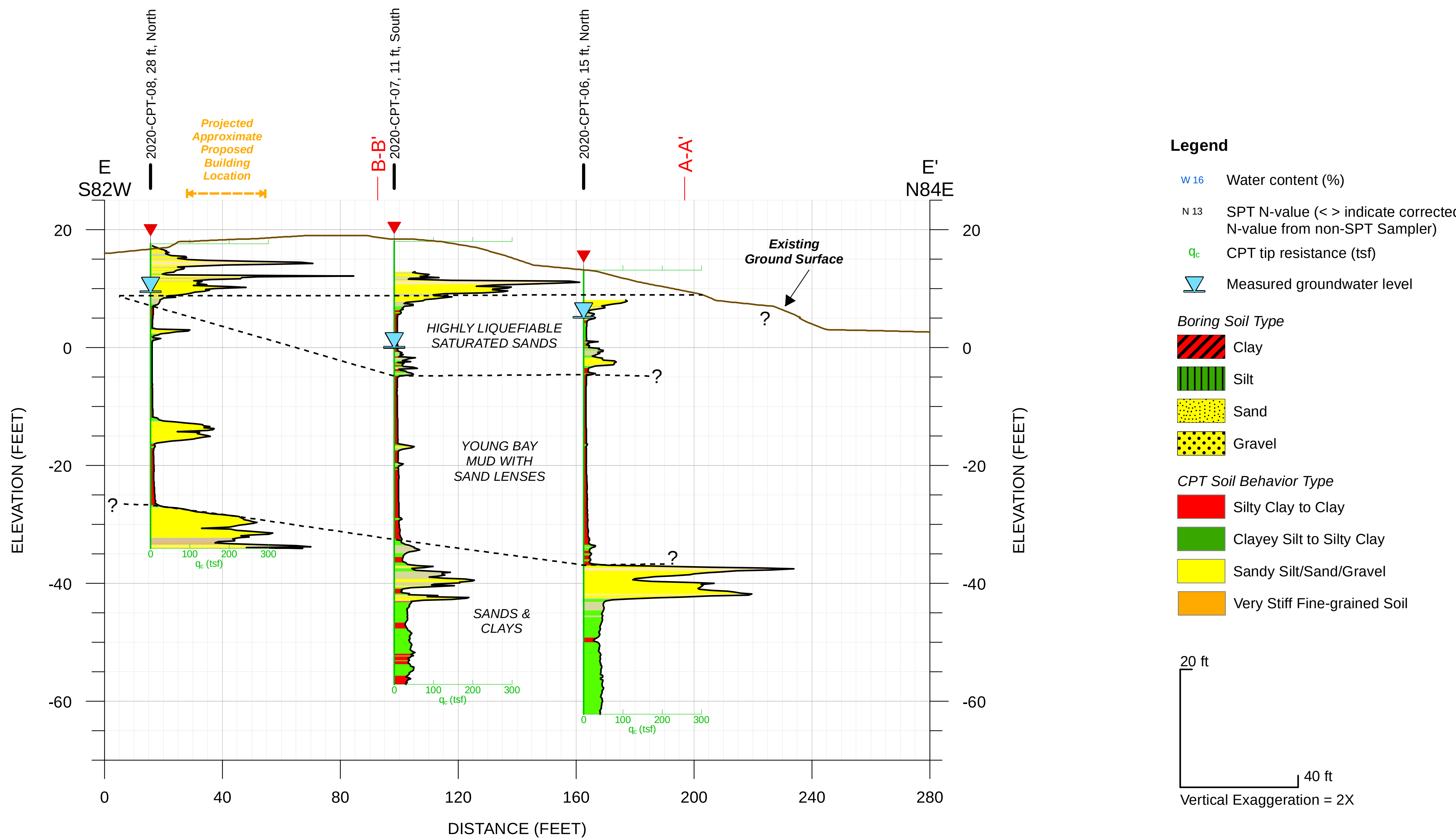

CROSS SECTION E-E'

PLATE 11

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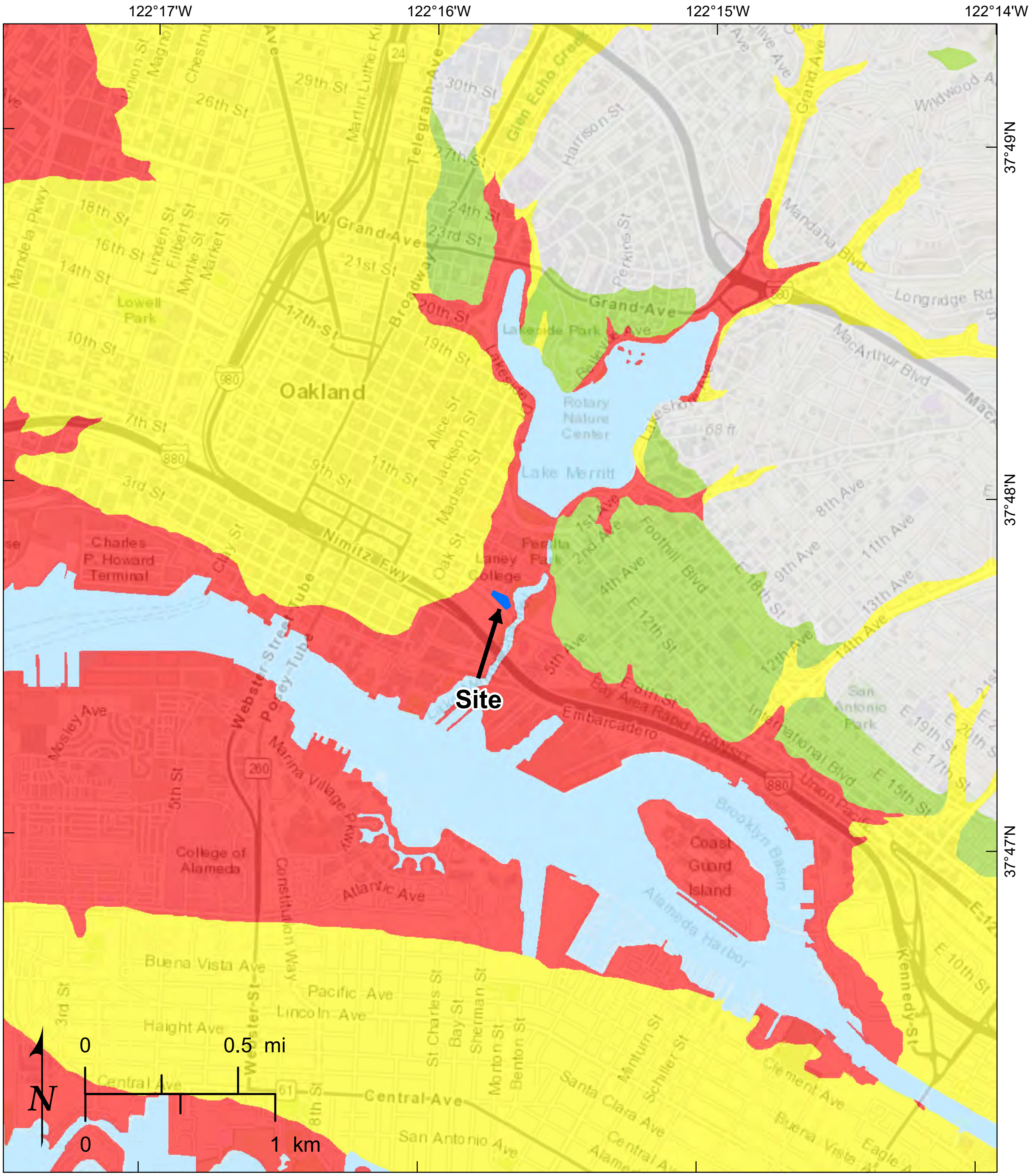

Legend

Liquefaction Susceptibility
(Witter et al., 2006)

- Very high
- High
- Moderate
- Low
- Very low

LIQUEFACTION SUSCEPTIBILITY

PLATE 12

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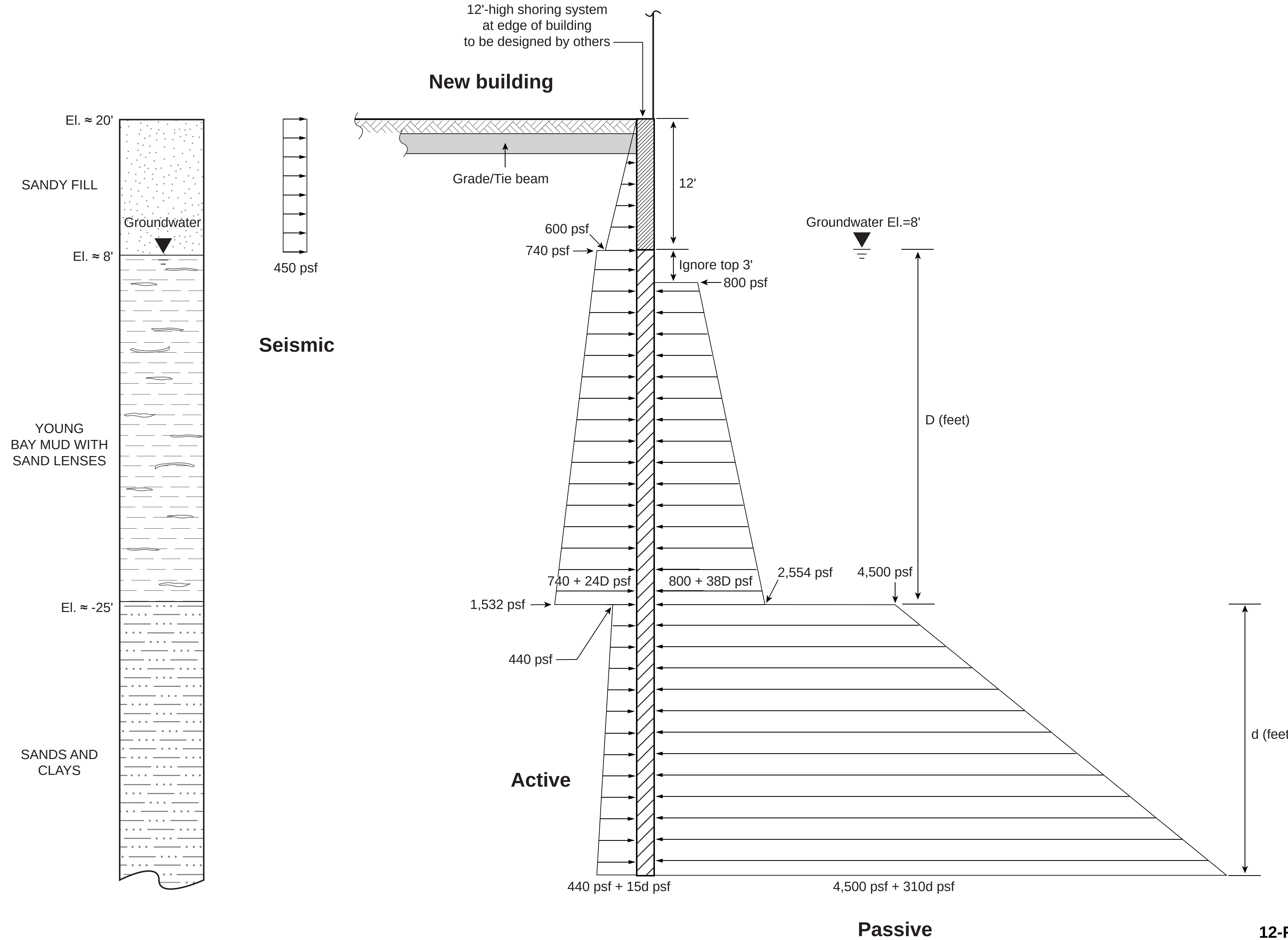
Legend

Historical Ground Failures (Knudsen et al., 2000)

	Miscellaneous effects
	Ground settlement
	Lateral Spread
	Sand boil
	Pipeline break
X	Cracks in streets or ground
x	Location of multiple ground effects (See corresponding symbols)
●	Geotechnical borings used in liquefaction evaluation
○	Groundwater level data
174	Number assigned to ground failure site - adapted from Youd and Hoose (1978), Tinsley and others (1998), and by Knudsen and others (2000)
—10—	Depth to historically high groundwater, in feet

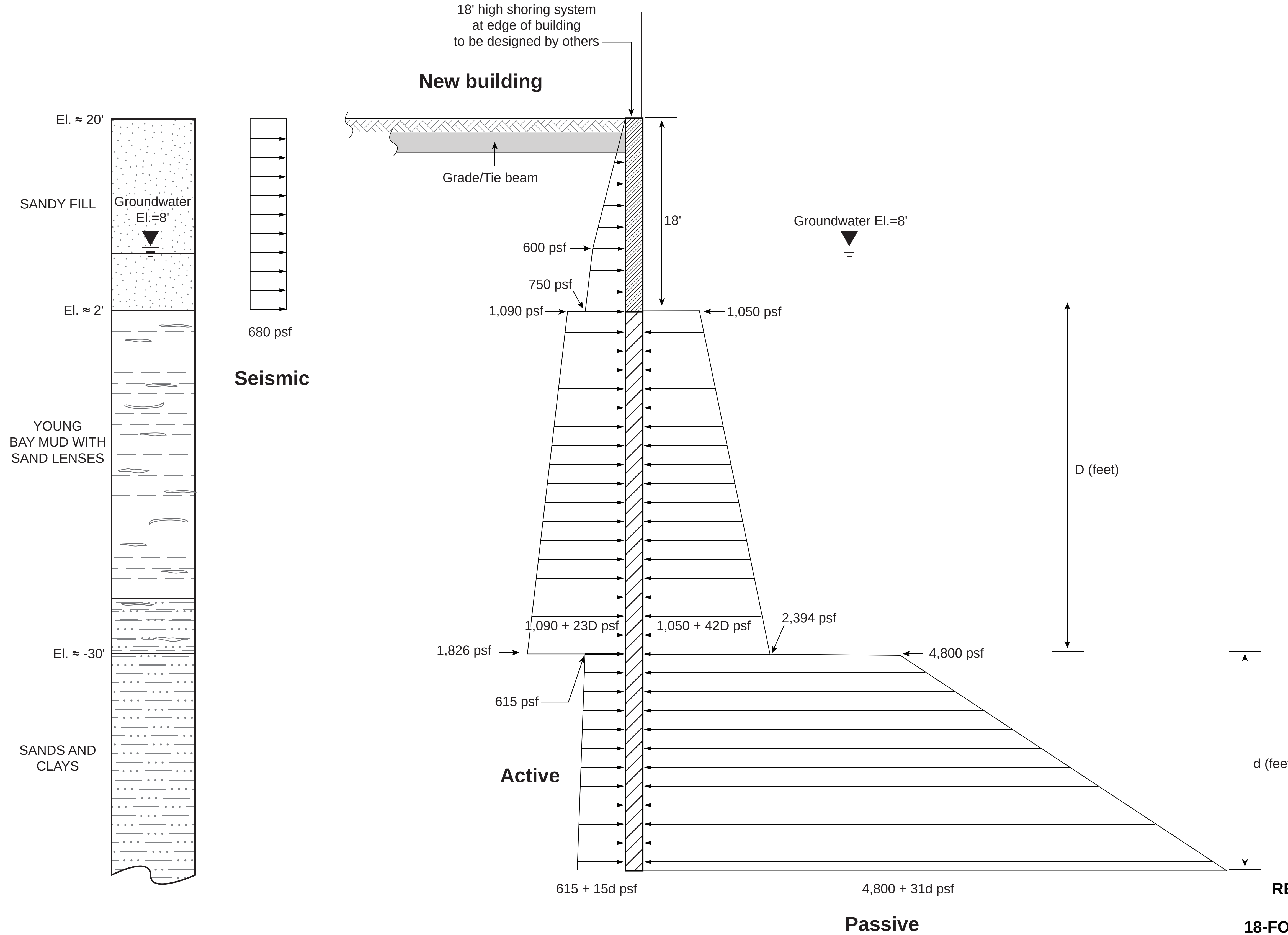
HISTORICAL LIQUEFACTION SITES AND HISTORICALLY HIGH GROUNDWATER TABLE

PLATE 13



RECOMMENDED LATERAL
PRESSURES FOR
12-FOOT HIGH SHORING SYSTEM

PLATE 14



RECOMMENDED LATERAL PRESSURES FOR 18-FOOT HIGH SHORING SYSTEM

PLATE 15

Appendix A

Field Explorations

CLASSIFICATION AND MATERIAL SYMBOLS

MAJOR DIVISIONS PER ASTM D2488-06			MAJOR GROUP NAMES AND MATERIAL SYMBOLS	
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW	Well-Graded GRAVEL
			GP	Poorly Graded GRAVEL
		Gravels with more than 12% fines	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	Clean sand less than 5% fines	SW	Well-Graded SAND
			SP	Poorly Graded SAND
		Sands with more than 12% fines	SM	SILTY SAND
			SC	CLAYEY SAND
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML	SILT
			CL	Lean CLAY
			OL	ORGANIC SILT
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH	Elastic SILT
			CH	Fat CLAY
			OH	ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	Peat or Highly Organic Soils

Notes:
Classification of soils on the boring logs is in general accordance with ASTM D2488, or D2487 if appropriate laboratory data are available.
The geologic formation is noted in bold font at the top of interpreted interval on the boring logs.

OTHER MATERIAL SYMBOLS

- Debris or Mixed Fill
- Pavement with Aggregate Base

SAMPLER TYPE

SPT (Driven) 1-3/8" ID 2" OD	Modified California (Driven) 2-3/8" ID 3" OD	Modified California (Driven) 1-7/8" ID 2-1/2" OD
Shelby Tube (Pushed) 2-7/8" ID 3" OD	Pitcher Barrel (Rotary-cut) 2-7/8" ID	Osterberg (Piston) 2-7/8" ID
101 Geobarrel (Rotary-cut) 2-7/8" ID	Rock Core (Rotary-cut) See log for size	Vibracore (Vibrated) See log for size
Push-core (Pushed) See log for size	Collected from Auger	Other See log for details

Note: Refer to text of report for additional details or other sampler types.

BLOW COUNT

Number of blows required to drive sampler each of three 6-in. intervals, as measured in the field (uncorrected). An SPT hammer (140 lb., falling 30-in.) was used unless otherwise noted on the boring log. For example:

Blow Count	Description
5 7 8	5, 7, and 8 blows for first, second, and third interval, respectively.
35 50/3"	35 blows for the first interval. 50 blows for the first 3 inches of the second interval. Lack of third value implies that driving was stopped 3 inches into the second interval.
WOH WOH 5	"WOH" indicates that the weight of the hammer was sufficient to advance the sampler over the first two intervals. 5 blows were required to advance the sampler over the third interval.

N-VALUE

The N-Value represents the blowcount for the last 12 inches of the sample drive if three 6-inch intervals were driven. N-value presented is independant of impact energy. If 50 hammer blows were insufficient to drive through either the second or the third interval, the total number of blows and total length driven are reported (excluding the first interval). "ref" (refusal) indicates that 50 blows were insufficient to drive through the first 6-inch interval.

Parenthesis indicate that an approximate correction has been applied for non-SPT drive samplers. For example, a factor of 0.63 is commonly used to adjust blow counts obtained using a 3-inch outside diameter modified California sampler to correspond to Standard Peneteration Test.

UNDRAINED SHEAR STRENGTH

A value of undrained shear strength is reported. The value is followed by a letter code indicating the type of test that was performed, as follows:

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

OTHER TESTS

Field or laboratory tests without a dedicated column on the boring log are reported in the Other Tests column. A letter code is used to indicate the type of test. For certain tests, a value representing the test result is also provided. Typical letter codes are as follows. Additional codes may be used. Refer to the report text and the laboratory testing results for additional information.

k - Permeability (cm/s)
Consol - Consolidation
Gs - Specific Gravity
MA - Particle Size Analysis
EI - Expansion Index
OVM - Organic Vapor Meter

WATER LEVEL SYMBOLS

- Initial water level
- Final water level
- Seepage encountered

INCREASING MOISTURE CONTENT

Dry
Moist
Wet

CONSISTENCY OF COHESIVE SOIL

CONSISTENCY	UNDRAINED SHEAR STRENGTH (KIPS PER SQUARE FOOT)
Very Soft	< 0.25
Soft	0.25 to 0.50
Medium Stiff	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	> 4.0

Note: In absence of test data, consistency has been estimated based on manual observation.

APPARENT DENSITY OF COHESIONLESS SOIL

APPARENT DENSITY	N-VALUE
Very Loose	0 to 4
Loose	5 to 9
Medium Dense	10 to 29
Dense	30 to 49
Very Dense	> 49

TERMS AND SYMBOLS USED ON BORING LOGS

PLATE A-1

Peralta Community College District
Project No. 04.72190021

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: N 37.795163+/- E 122.262754+/- WGS84 SURFACE EL: 18.0 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
						FILL: 0 TO 6 FEET SILTY SAND (SM): loose to medium dense, light brown, dry, fine-grained, silty							
5						Change color to mottled gray brown , trace coarse-grained, few gravel (fine, subangular to subrounded), few brick fragments and organics							
						NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 6.0 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: March 29, 2019
DRILLING METHOD: 3-in dia Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: Fugro
LOGGED BY: F De Paola
CHECKED BY: T Chen

LOG OF BORING NO. 2019-CPT-01
Laney College Library Learning Resource Center
Oakland, California

Peralta Community College District
Project No. 04.72190021

FUGRO

LOCATION:
N 37.794900+/- E 122.261959+/- WGS84
SURFACE EL: 14.1 ft +/- (rel. NAVD88 datum)

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
						FILL: 0 TO 6 FEET SILTY SAND with GRAVEL (SM): medium dense, light gray, dry, fine- to medium-grained, trace coarse-grained, silty, with gravel (fine to coarse, subangular to subrounded)							
						PEAT (PT): very soft to soft, black, dry, with organic odor.		55					
5						Fat CLAY (CH): soft, gray, moist, trace sand (fine-grained), trace small shell fragments, few organics, with strong organic odor		58					

NOTES:
1. Terms and symbols defined on Plate A-1.

BORING DEPTH: 6.0 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: March 29, 2019
DRILLING METHOD: 3-in dia Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: Fugro
LOGGED BY: F De Paola
CHECKED BY: T Chen

LOG OF BORING NO. 2019-CPT-02
Laney College Library Learning Resource Center
Oakland, California

Peralta Community College District
Project No. 04.72190021

LOCATION:

N 37.794463+/- E 122.262030+/- WGS84
SURFACE EL: 16.3 ft +/- (rel. NAVD88 datum)

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
						FILL: 0 TO 6 FEET Lean CLAY with GRAVEL (CL): soft to medium stiff, mottled gray brown, dry, with gravel (fine to coarse, subangular to rounded), few sand (fine- to coarse-grained)							
						CLAYEY GRAVEL with SAND (GC): loose, mottled gray brown, dry to moist, fine to coarse, subangular to rounded, clayey, with sand (fine- to coarse-grained)		13	20				
5						CLAYEY SAND (SC): loose to medium dense, dark brown, moist, fine- to coarse-grained, clayey , few gravel (fine, subangular to subrounded)							

NOTES:
1. Terms and symbols defined on Plate A-1.

BORING DEPTH: 5.0 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: March 29, 2019
DRILLING METHOD: 3-in dia Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: Fugro
LOGGED BY: F De Paola
CHECKED BY: T Chen

LOG OF BORING NO. 2019-CPT-03
Laney College Library Learning Resource Center
Oakland, California

LOCATION:

N 37.794856+/- E 122.262089+/- WGS84
SURFACE EL: 17.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
						FILL: 0 TO 19.5 FEET							
15						SILTY SAND with GRAVEL (SM): loose to medium dense, brown, dry, fine- to medium-grained, trace coarse-grained, silty, with gravel (fine to coarse, angular to subangular)							
		S1	18 17 13	(19)	14"/18"	SILTY GRAVEL with SAND (GM): medium dense, mottled gray brown, dry, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), silty, trace clay							
	5	S3	50	ref	6"/6"	with rock fragments up to 2", dry to moist at 5'							
10		S4	2 2 2	4	18"/18"	Fat CLAY with SAND (CH): medium stiff, mottled black green dark gray, dry, with sand (fine- to coarse-grained), trace organics, trace glass fragments, with organic odor							
	10	S5	14 10 5	(9)	18"/18"	SILTY SAND with GRAVEL (SM): medium dense, mottled brown gray, dry, fine- to coarse-grained, silty, with gravel (fine to coarse, angular to subangular), a large brick fragment at 11'	91	24	21				MA
5		S6	3 3 5	8	16"/18"	with abundant wood chips at 12' to 13', trace glass fragments, moist below 12.5'							
	15	S7	5 6 19	(16)	18"/18"	Poorly-graded SAND with SILT and GRAVEL (SP-SM): medium dense, mottled brown gray, moist, fine- to coarse-grained, with silt, with abundant wood chips, with brick and glass fragments, trace clay chunks	95	26	6				MA Organic = 5%
0		S8	10 4 5	9	16"/18"	samll rock fragments at 16.5' to 17'		82					Organic = 21.2%
						ORGANIC CLAY with SAND (OH): soft to medium stiff, mottled brown dark gray, moist, with peat, with sand (fine- to coarse-grained), trace gravel (fine, angular to subangular), few wood chips							
	20	S9	3 3 2	5	10"/18"	**NATIVE: 19.5 TO 76.5 FEET** Fat CLAY (CH): medium stiff, gray, moist, trace wood chips		53					Organic = 6.6%
-5													
	25	S10	50 psi 100 psi		30"/30"	very soft to soft, trace wood chips	52	83					Consol
-10													
	30	S11	5 13 9	(14)	18"/18"	soft to medium stiff, trace sand (fine-grained), trace rootlets, a 2" rock fragment at 30'	69	58	93	73	43	0.5 Q	MA
						Poorly-graded SAND with SILT (SP-SM): medium dense, gray, wet, fine- to medium-grained, with silt, trace small shell fragments	94	27	8				MA
-15													
	35					3" rock fragment at 35'							
		S12	100 psi		30"/30"	Fat CLAY (CH): soft to medium stiff, gray, moist							
-20													

Continued

BORING DEPTH: 76.5 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Estabilished
FIELDWORK DATE: January 7, 2020
DRILLING METHOD: 4-in. dia. Solid Stem Auger/Rotary Wash

HAMMER TYPE: Automatic Trip
RIG TYPE: CME 75 Track
DRILLED BY: Geo-Ex
LOGGED BY: T Chen
CHECKED BY: A Johan

LOG OF BORING NO. 2020-B-01
Laney College Library Learning Resource Center
Oakland, California

FUGRO

LOCATION:

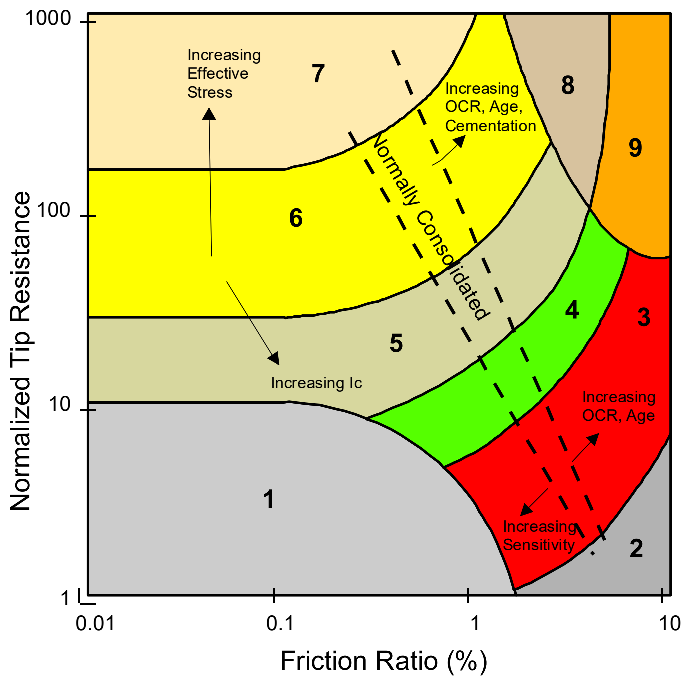
N 37.794856+/- E 122.262089+/- WGS84
SURFACE EL: 17.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
		S13	0 2 6	(5)	18"/18"	medium stiff	59	71				0.7 Q	
-25													
	45	S14	200 psi 650 psi		0"/15"	SILTY SAND (SM): medium dense to dense, gray, wet, fine- to medium-grained, silty							
-30													
	50	S15	26 24 14	(24)	18"/18"		112	18	16				MA
						SANDY Lean CLAY (CL): very stiff, mottled gray yellowish brown, moist, sandy (fine- to medium-grained)							
-35													
						SILTY SAND (SM): dense to very dense, gray, wet, fine- to medium-grained, silty, trace shell fragments							
	55	S16	49 50/4"	(32)/4"	10"/10"		116	17	17				MA
						very dense, fine- to medium-grained, with coarse-grained, with silt, few gravel (fine, angular to subangular)							
-40													
	60												
-45													
	65	S17	35 50	(32)/6"	12"/12"			19	19				
-50													
	70					Lean CLAY (CL): very stiff to hard, light brown, moist							
-55													
	75	S18	7 10 11	21	18"/18"			37					

NOTES:
1. Terms and symbols defined on Plate A-1.

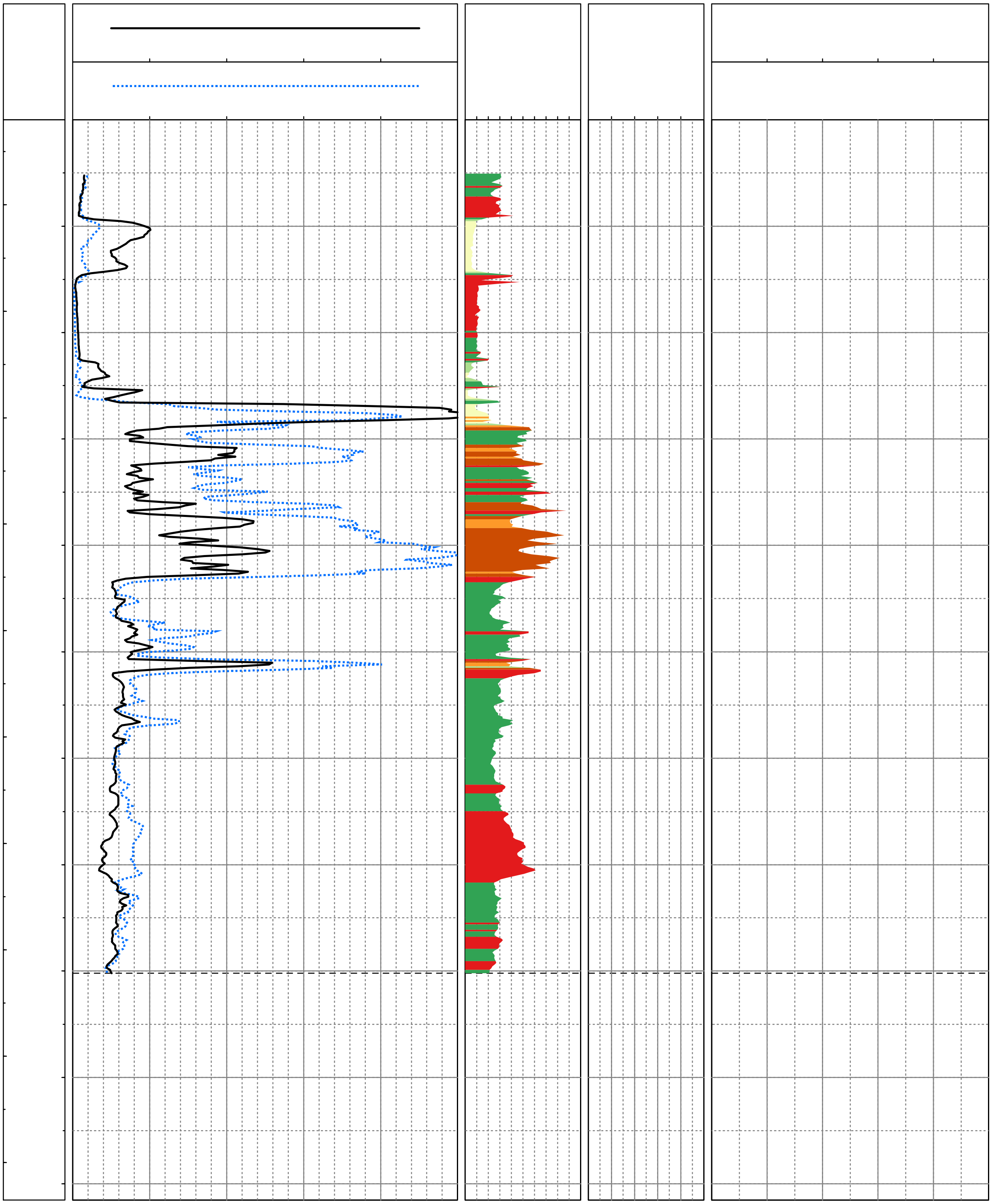
LOG OF BORING NO. 2020-B-01
Laney College Library Learning Resource Center
Oakland, California

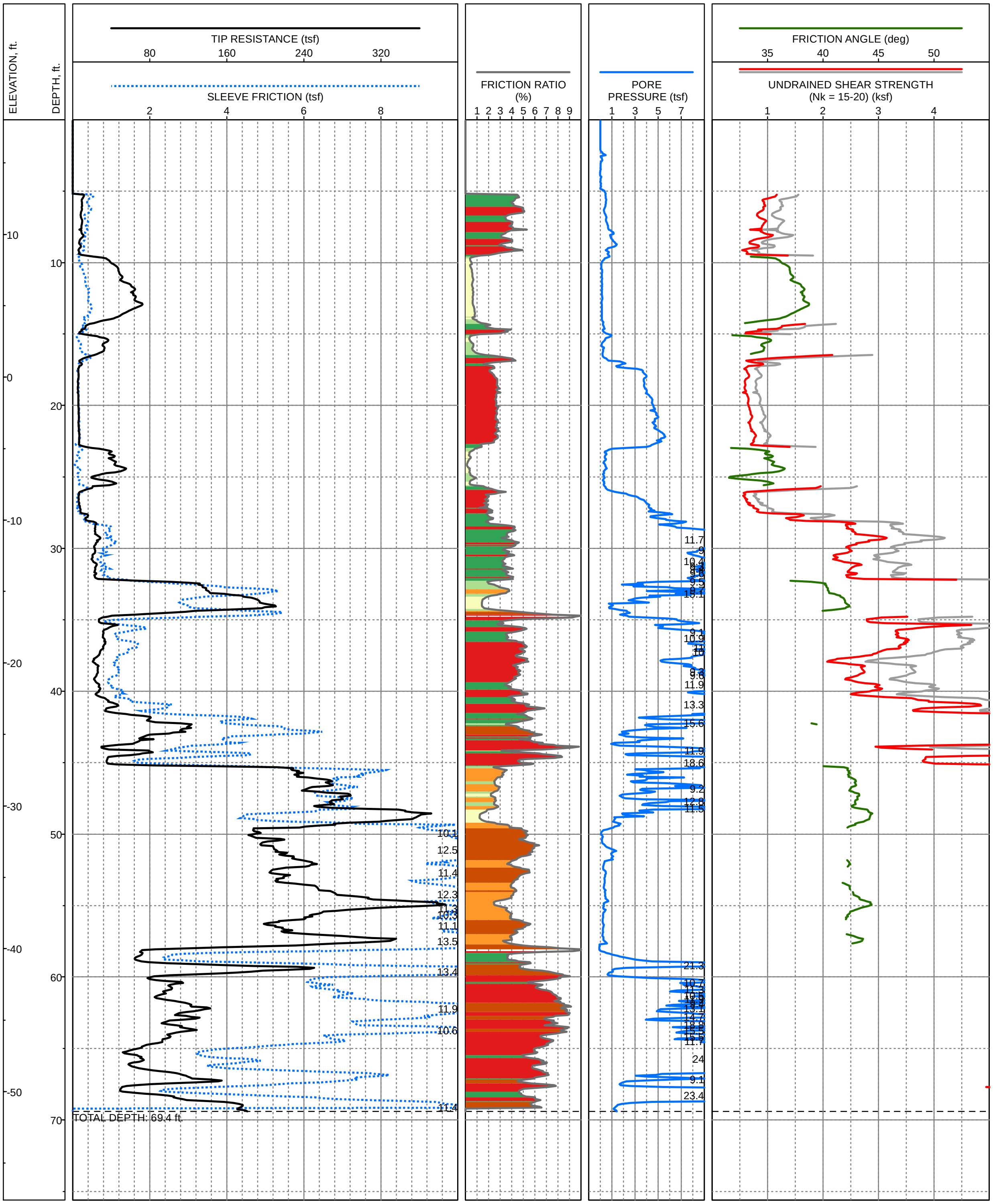
CPT CORRELATION CHART
(Robertson 1990)



Zone	Soil Behavior Type
1	Sensitive Fine-grained
2	Organic Soils, Peats
3	Clays - Clay to Silty Clay
4	Silt Mixtures - Clayey Silt to Silty Clay
5	Sand Mixtures - Silty Sand to Sandy Silt
6	Sands - Clean Sand to Silty Sand
7	Gravelly Sand to Sand
8	Very Stiff Sand to Clayey Sand
9	Very Stiff Fine-Grained

Plate A-7: Key to CPT Interpretation

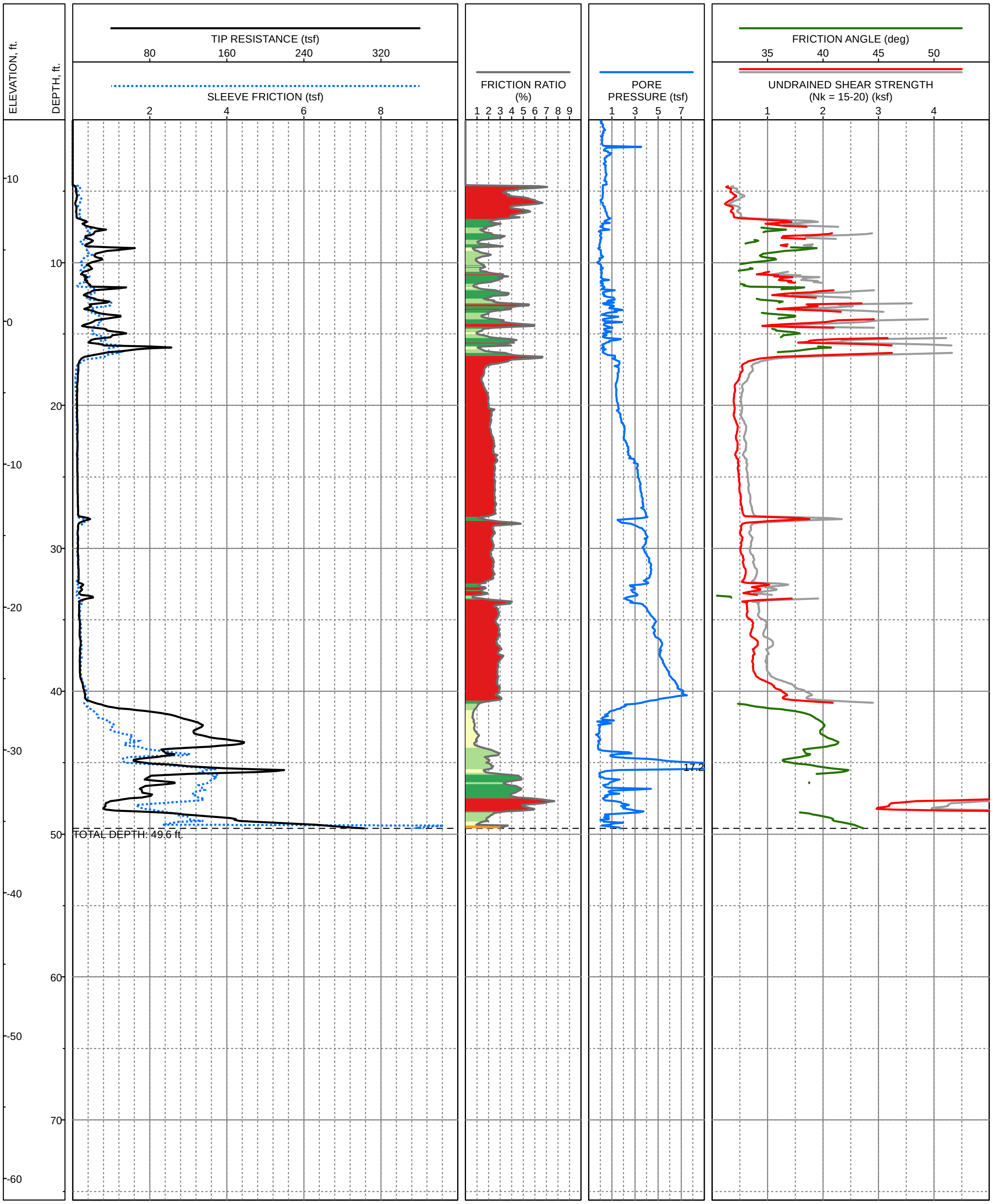




LOCATION: E6,052,365, N2,116,794, NAD83 SP CA Z3 FT
SURFACE EL: 18ft +/-
COMPLETION DEPTH: 69.4ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.59

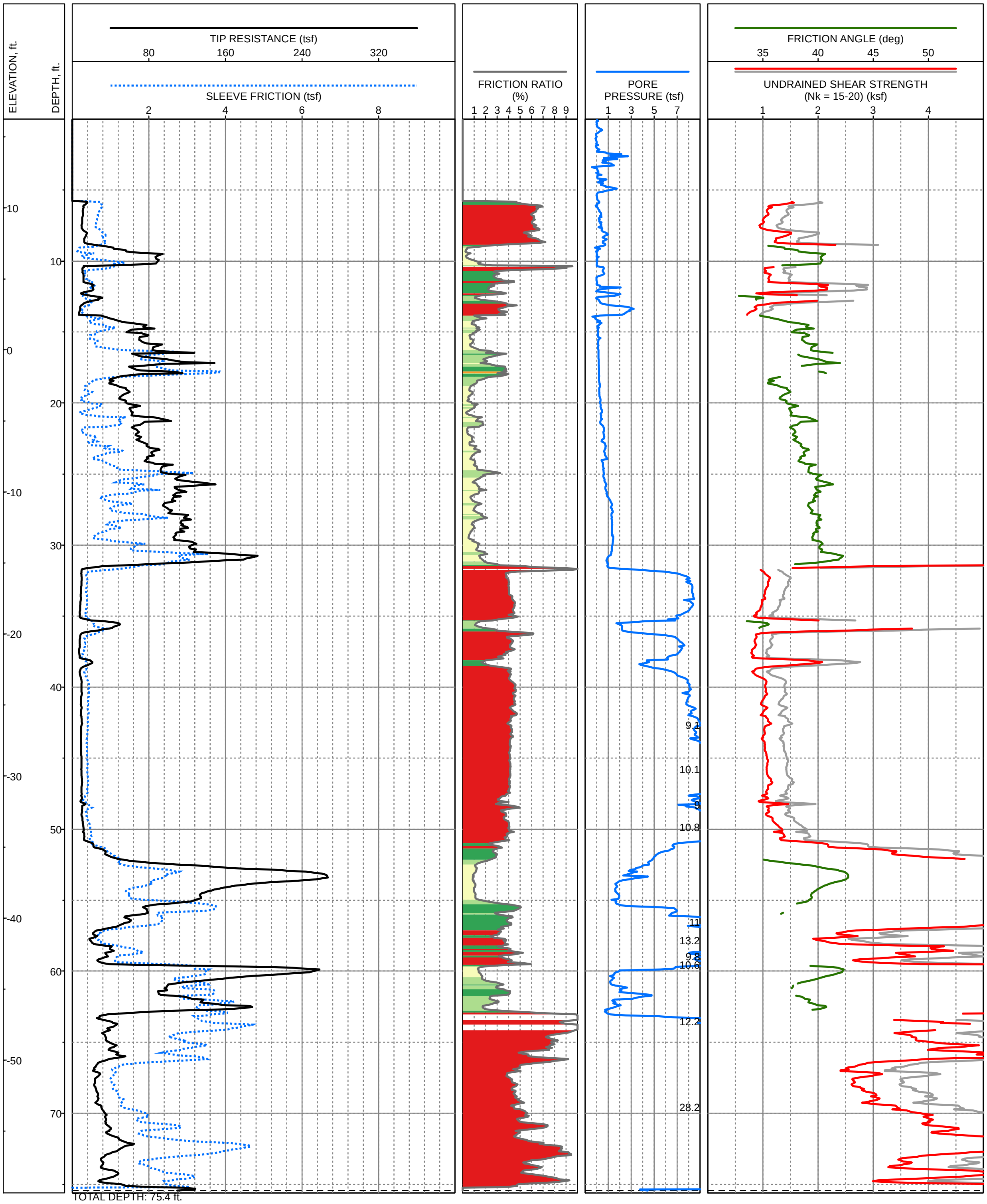
PLATE A-8: LOG OF 2019-CPT-01



LOCATION: E6,052,593, N2,116,694, NAD83 SP CA Z3 FT
SURFACE EL: 14ft +/-
COMPLETION DEPTH: 49.6ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.59

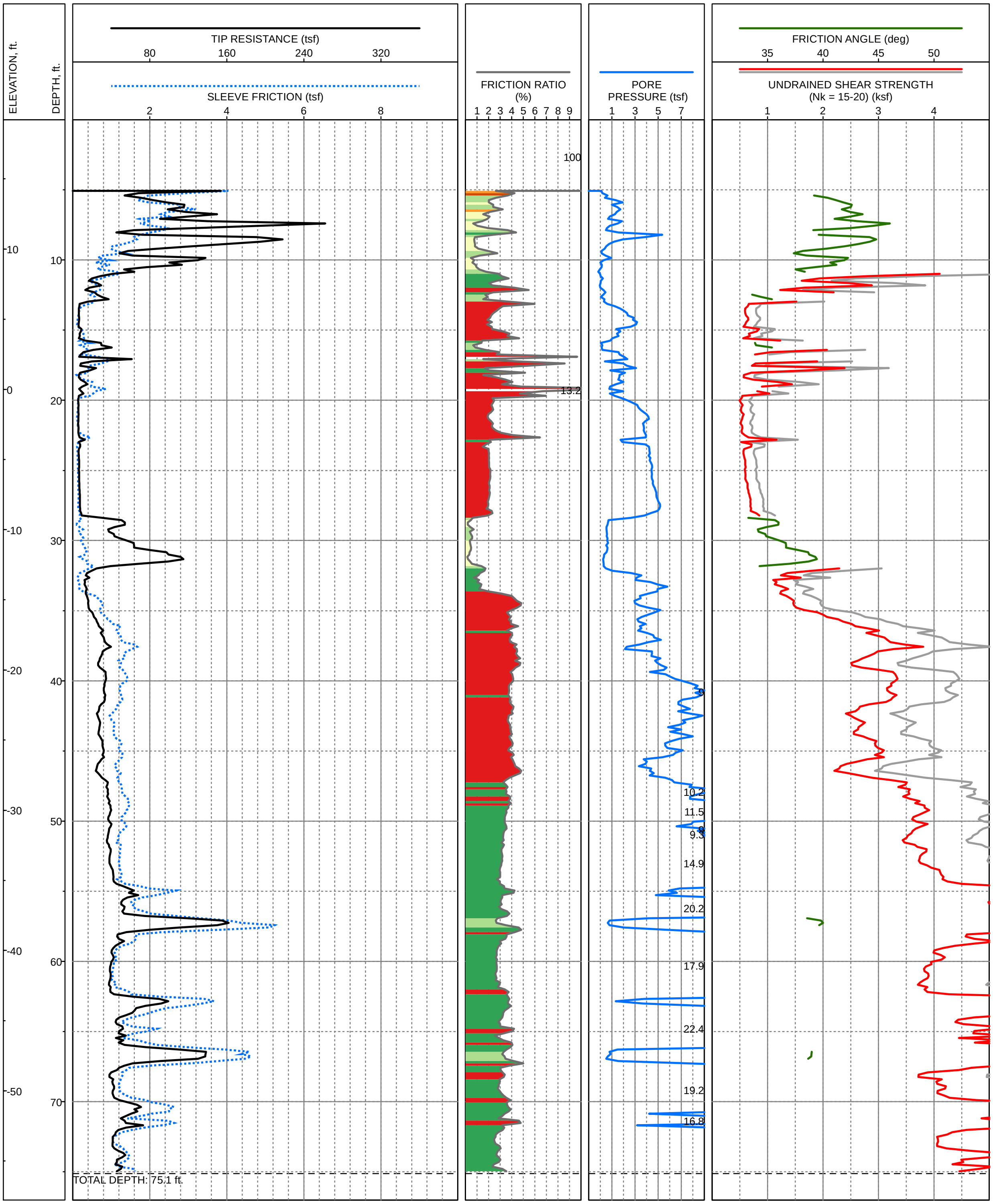
PLATE A-9: LOG OF 2019-CPT-02



LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16ft +/-
COMPLETION DEPTH: 75.4ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.59

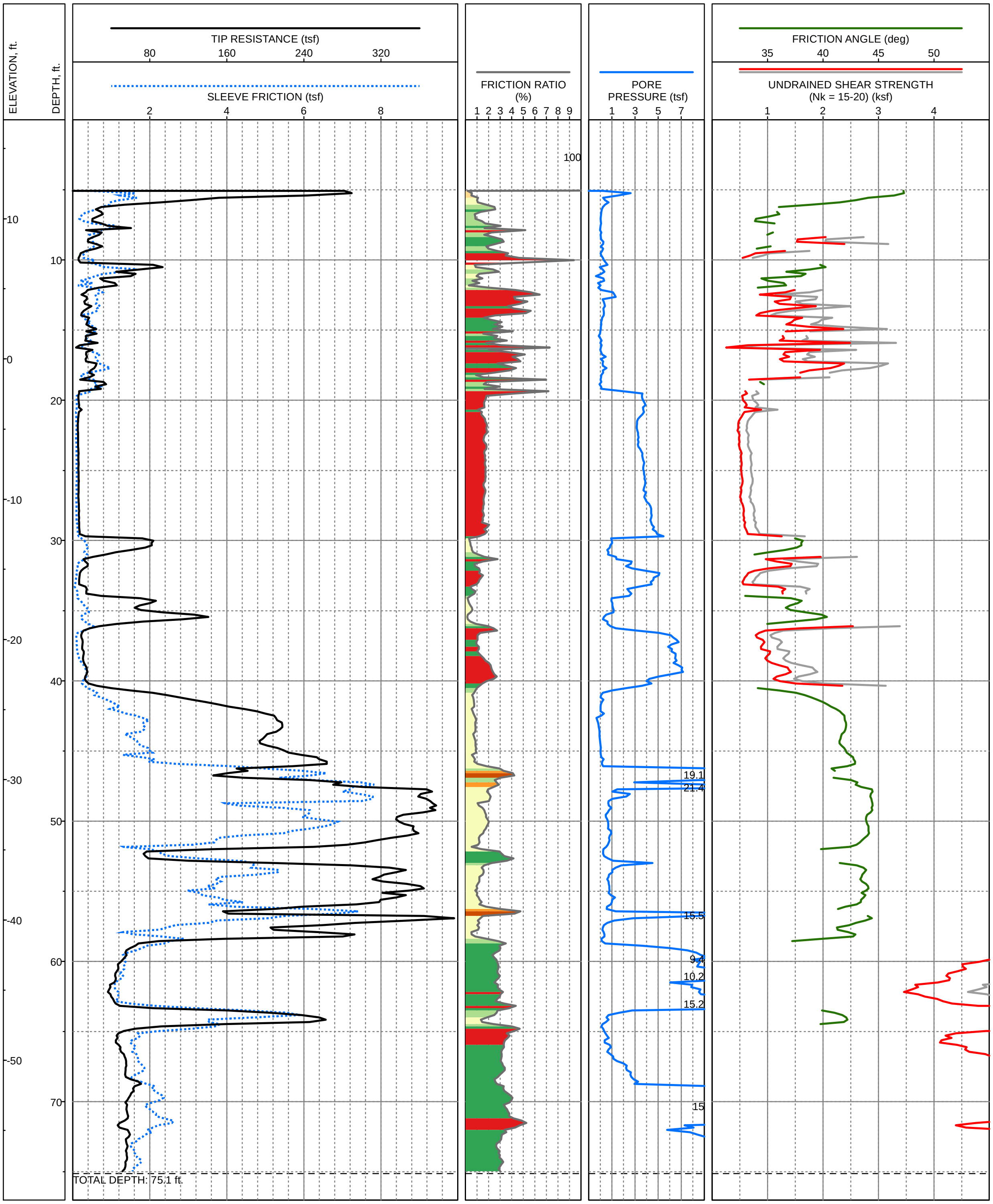
PLATE A-10: LOG OF 2019-CPT-03



LOCATION: E6,052,490, N2,116,767, NAD83 SP CA Z3 FT
SURFACE EL: 19.2ft
COMPLETION DEPTH: 75.1ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

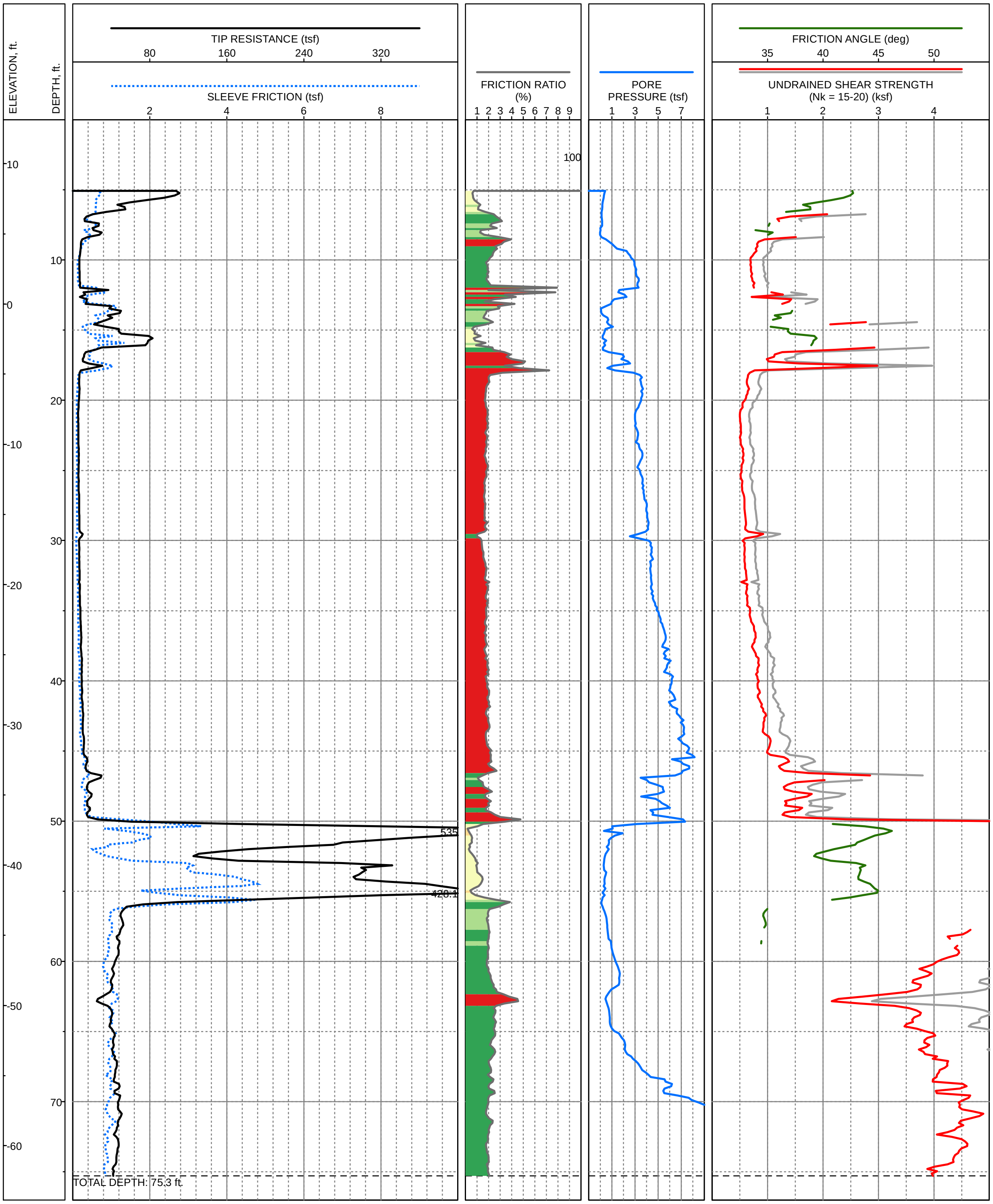
PLATE A-11: LOG OF 2020-CPT-04



LOCATION: E6,052,557, N2,116,734, NAD83 SP CA Z3 FT
SURFACE EL: 17.1ft
COMPLETION DEPTH: 75.1ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

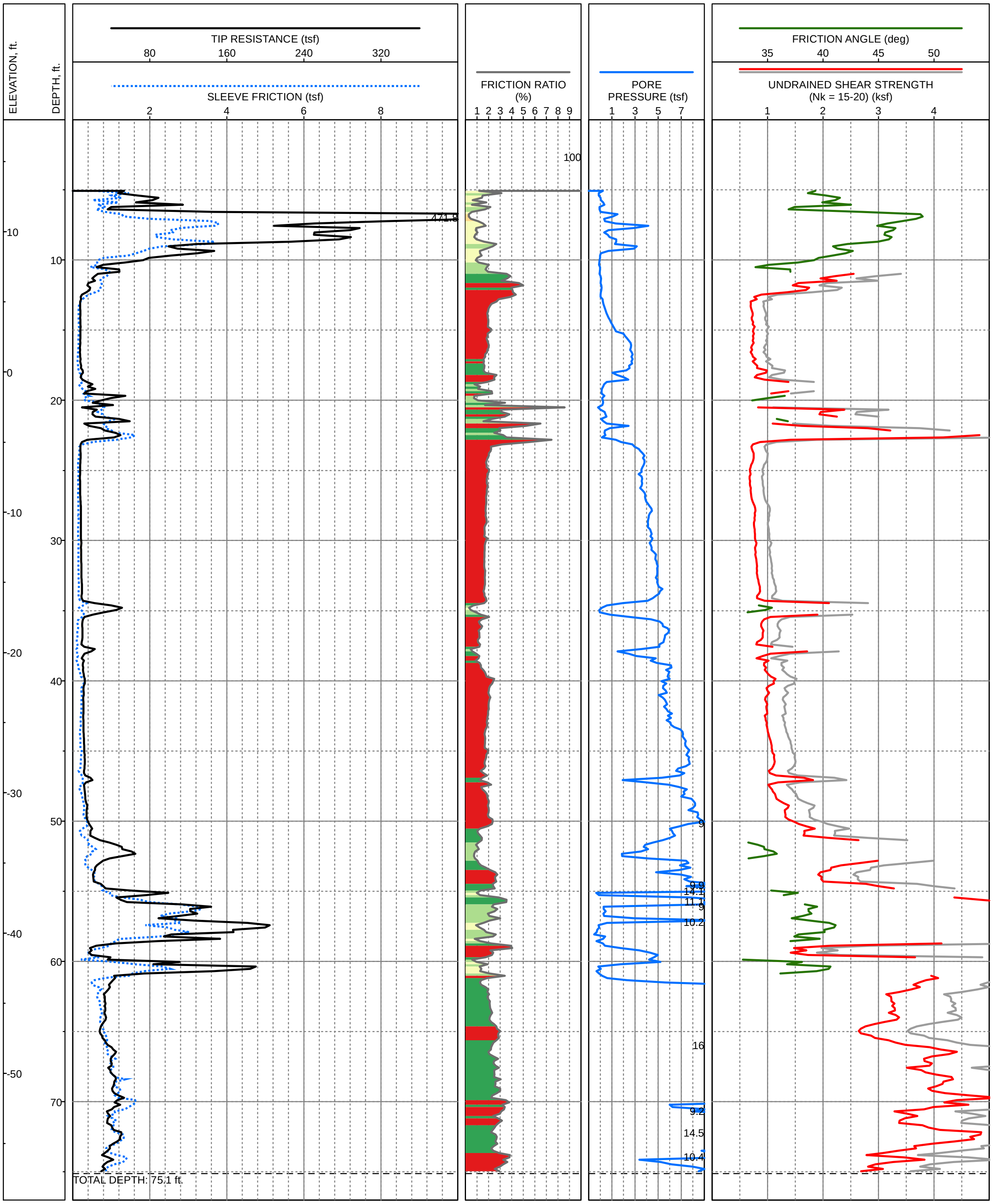
PLATE A-12: LOG OF 2020-CPT-05



LOCATION: E6,052,632, N2,116,632, NAD83 SP CA Z3 FT
SURFACE EL: 13.1ft
COMPLETION DEPTH: 75.3ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

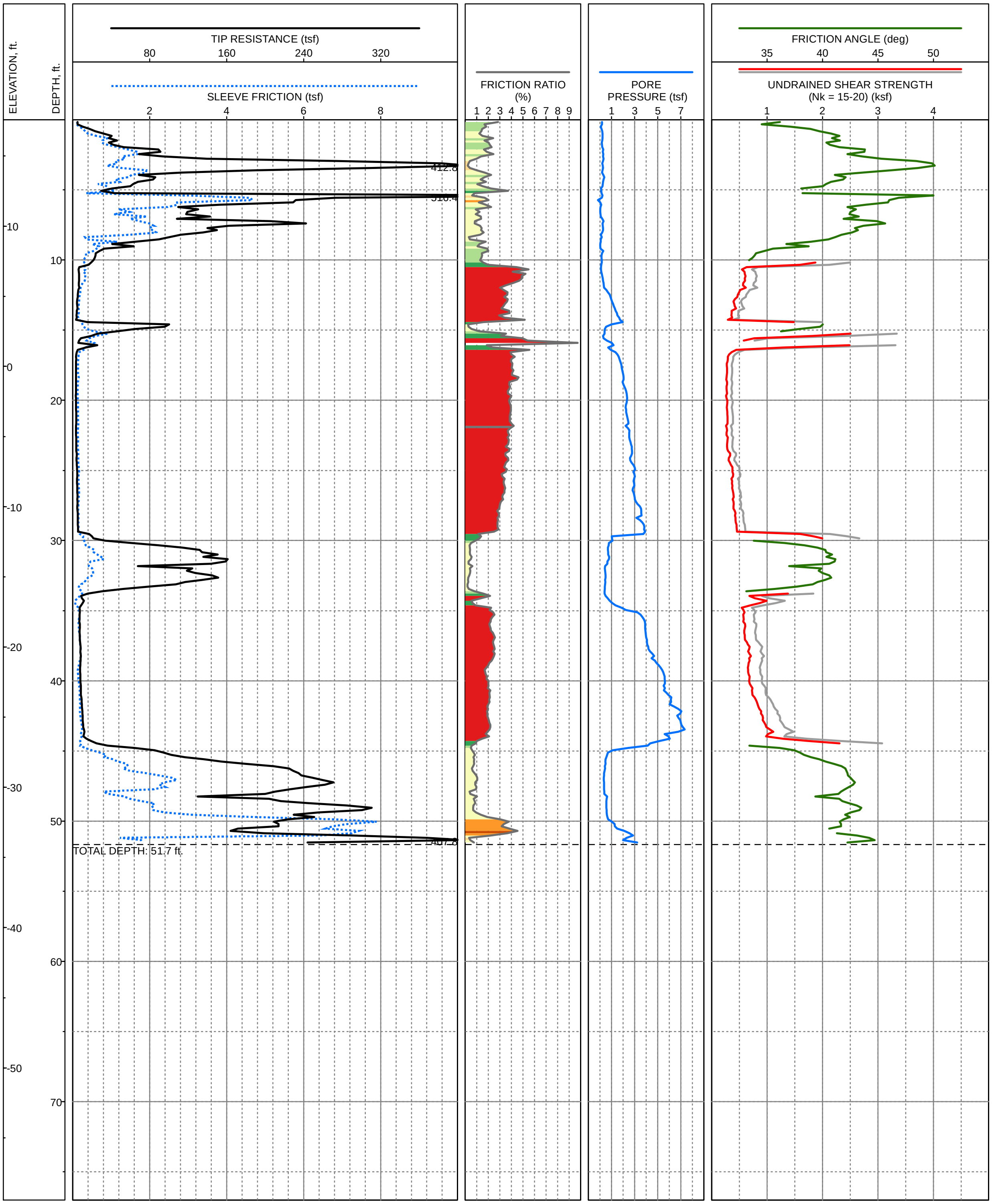
PLATE A-13: LOG OF 2020-CPT-06



LOCATION: E6,052,572, N2,116,598, NAD83 SP CA Z3 FT
SURFACE EL: 18.0ft
COMPLETION DEPTH: 75.1ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

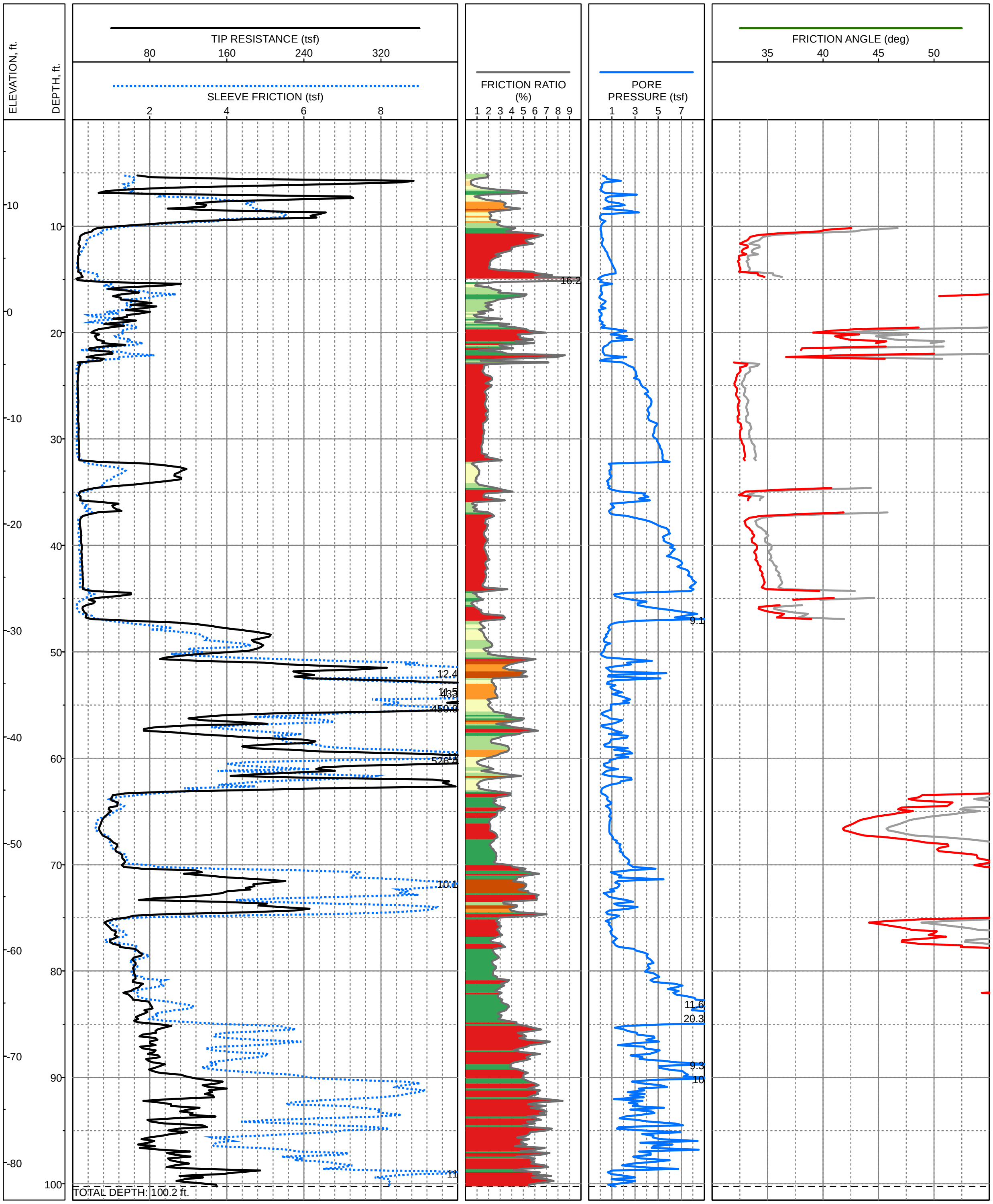
PLATE A-14: LOG OF 2020-SCPT-07



LOCATION: E6,052,485, N2,116,625, NAD83 SP CA Z3 FT
SURFACE EL: 17.6ft
COMPLETION DEPTH: 51.7ft
TESTDATE: 1/2/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

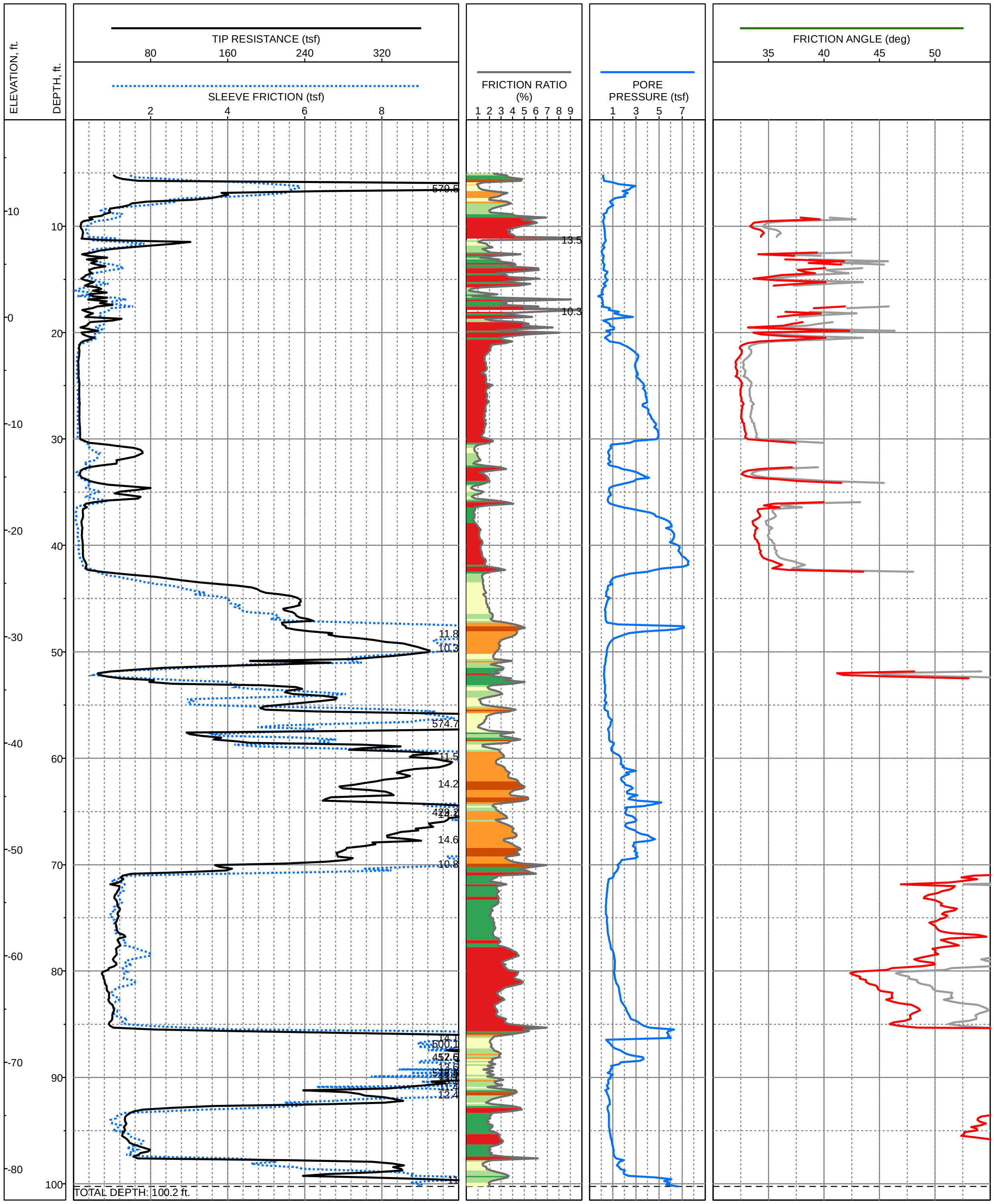
PLATE A-15: LOG OF 2020-CPT-08



LOCATION: E6,052,530, N2,116,637, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.0ft
COMPLETION DEPTH: 100.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

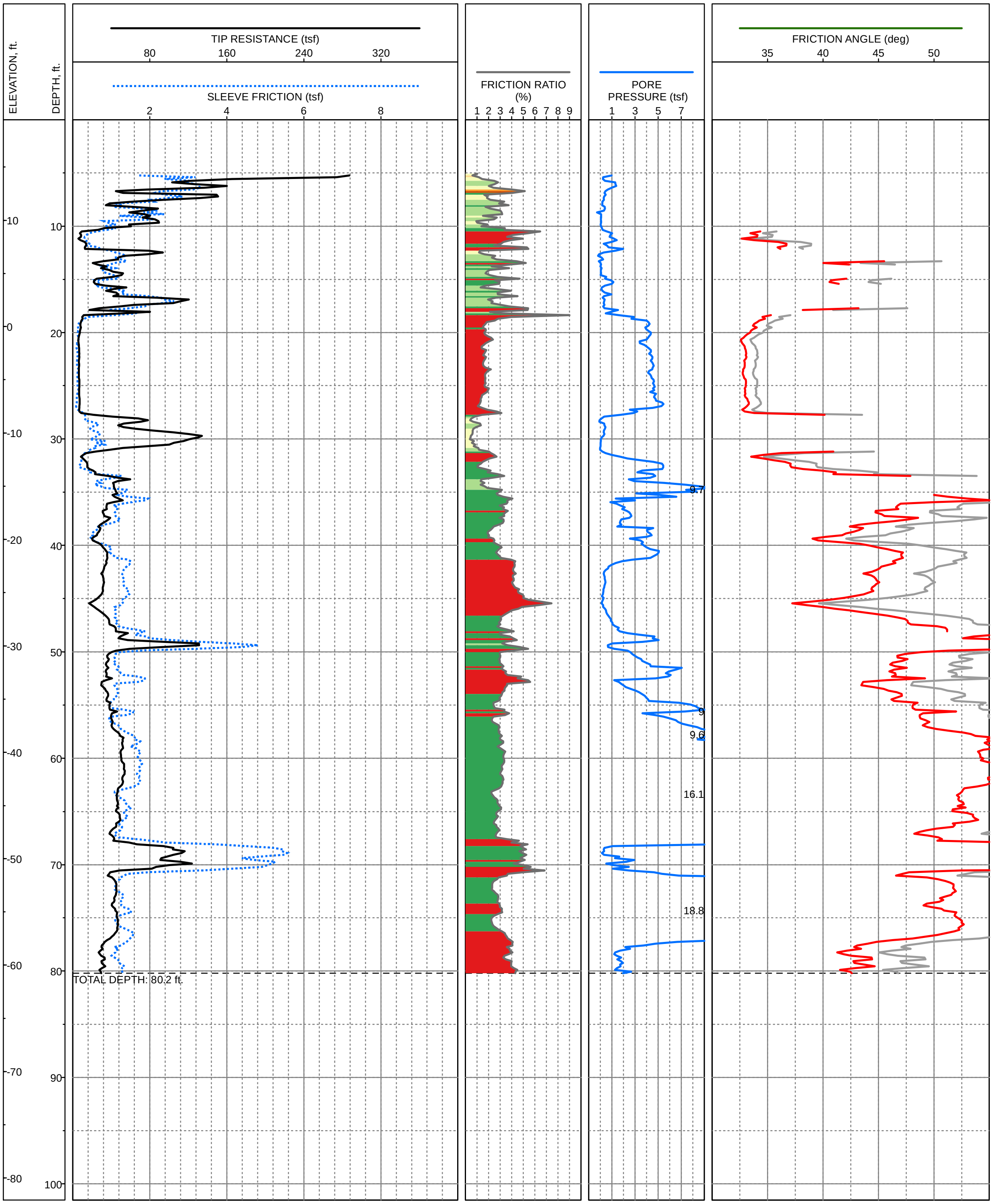
PLATE A-16: LOG OF 2022-CPT-16



LOCATION: E6,052,534, N2,116,701, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.6ft
COMPLETION DEPTH: 100.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

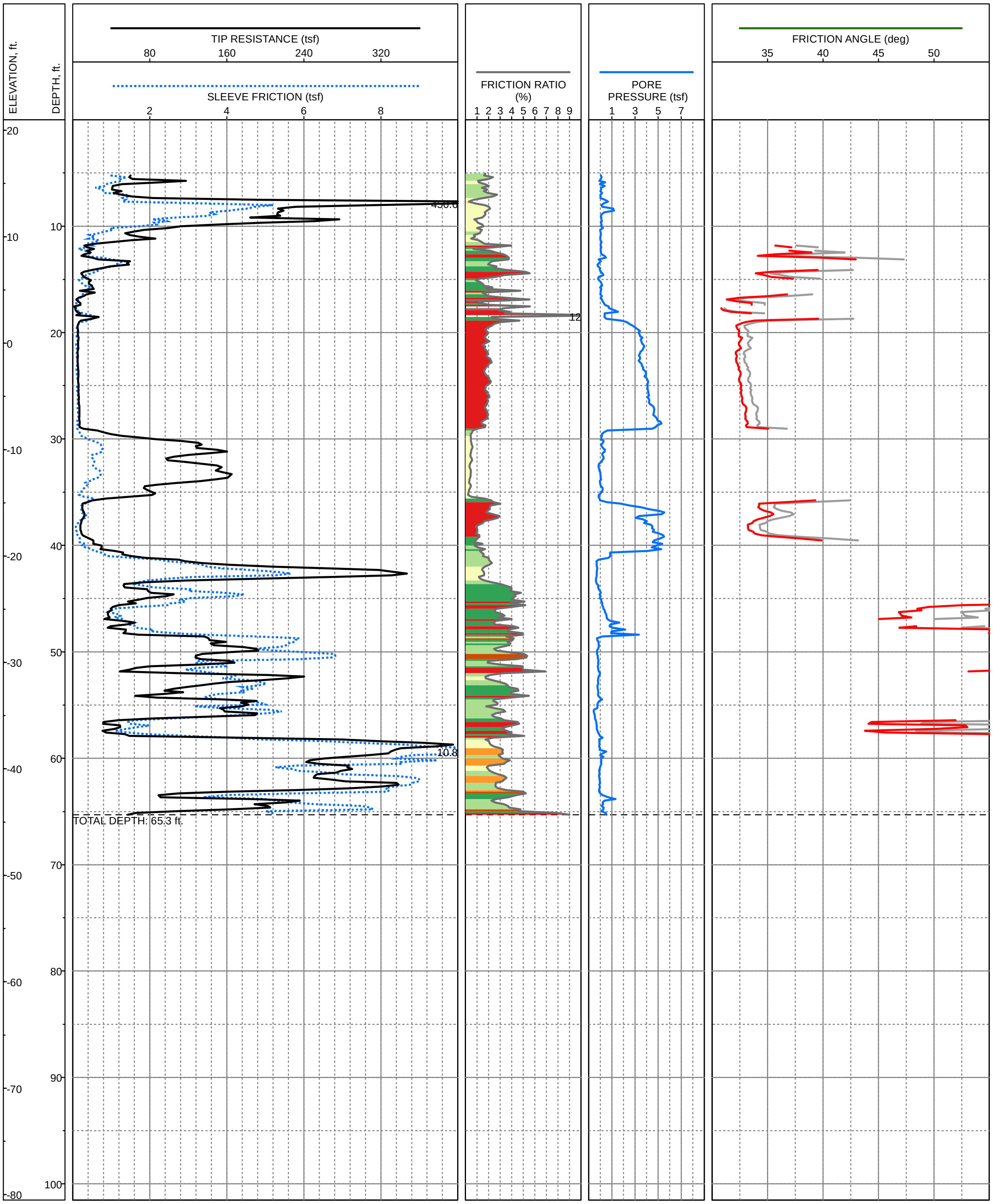
PLATE A-17: LOG OF 2022-CPT-17



LOCATION: E6,052,503, N2,116,793, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 19.4ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

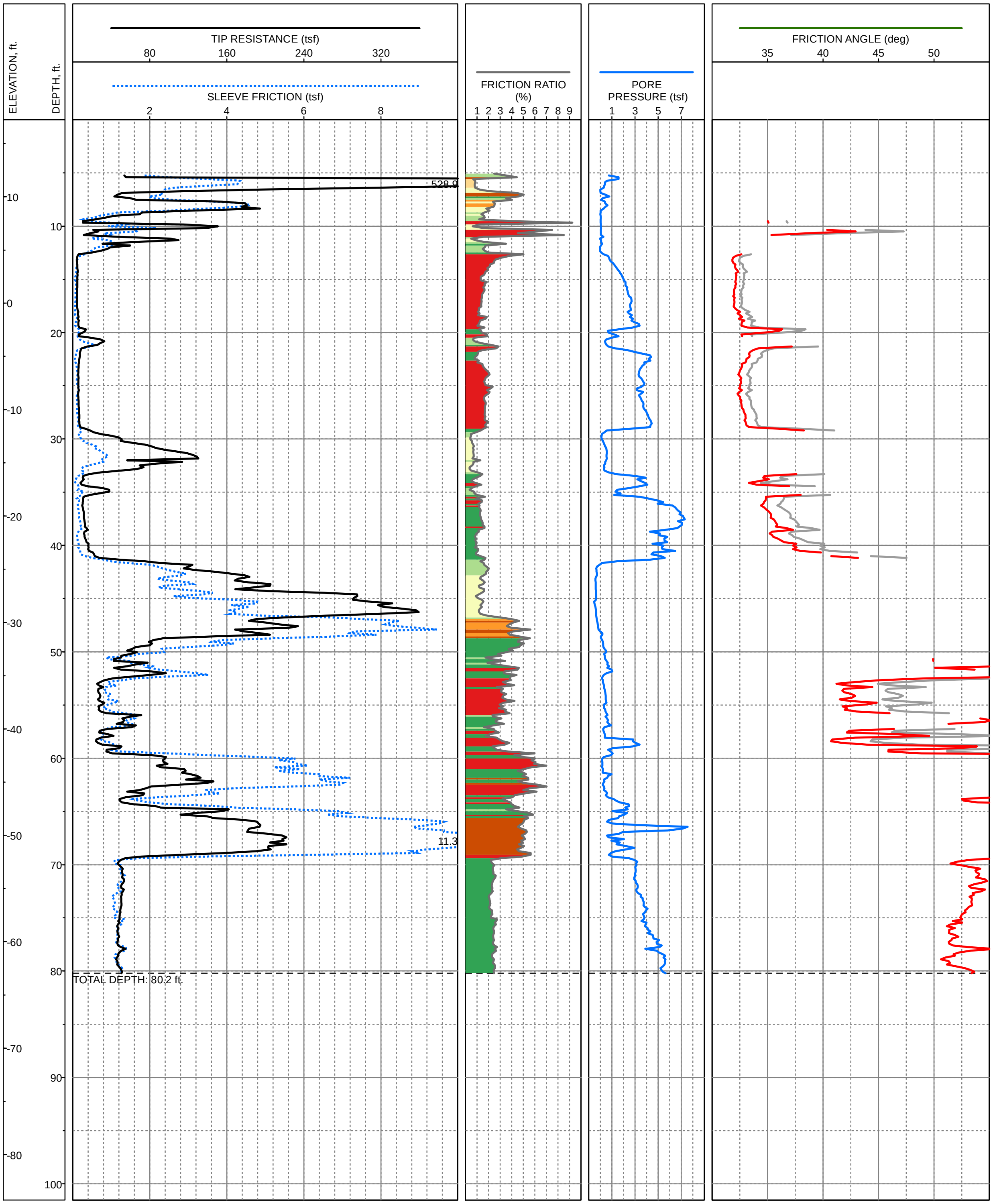
PLATE A-18: LOG OF 2022-CPT-18



LOCATION: E6,052,485, N2,116,730, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 21.0ft
COMPLETION DEPTH: 65.3ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

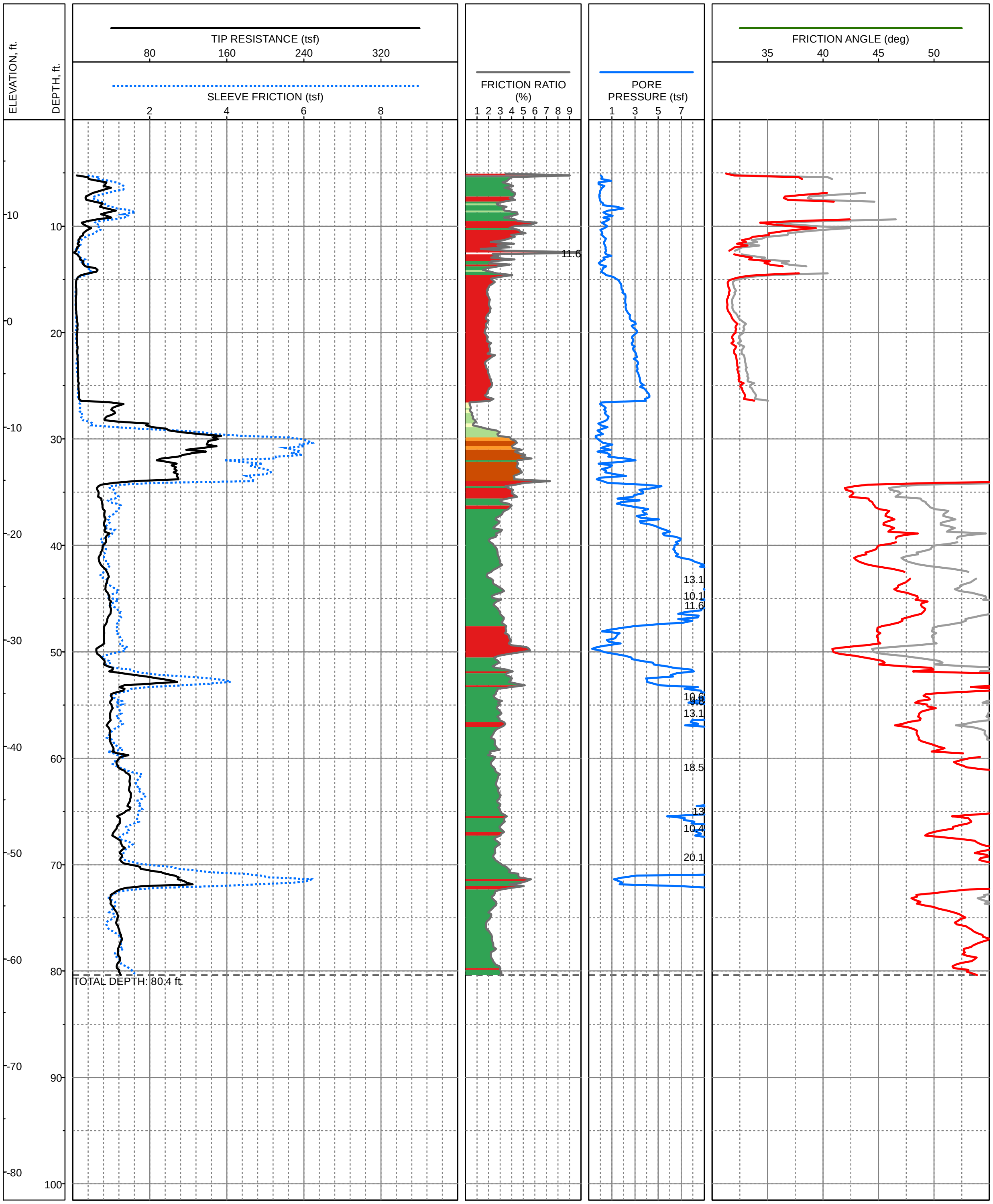
PLATE A-19: LOG OF 2022-CPT-19



LOCATION: E6,052,442, N2,116,671, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 17.2ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

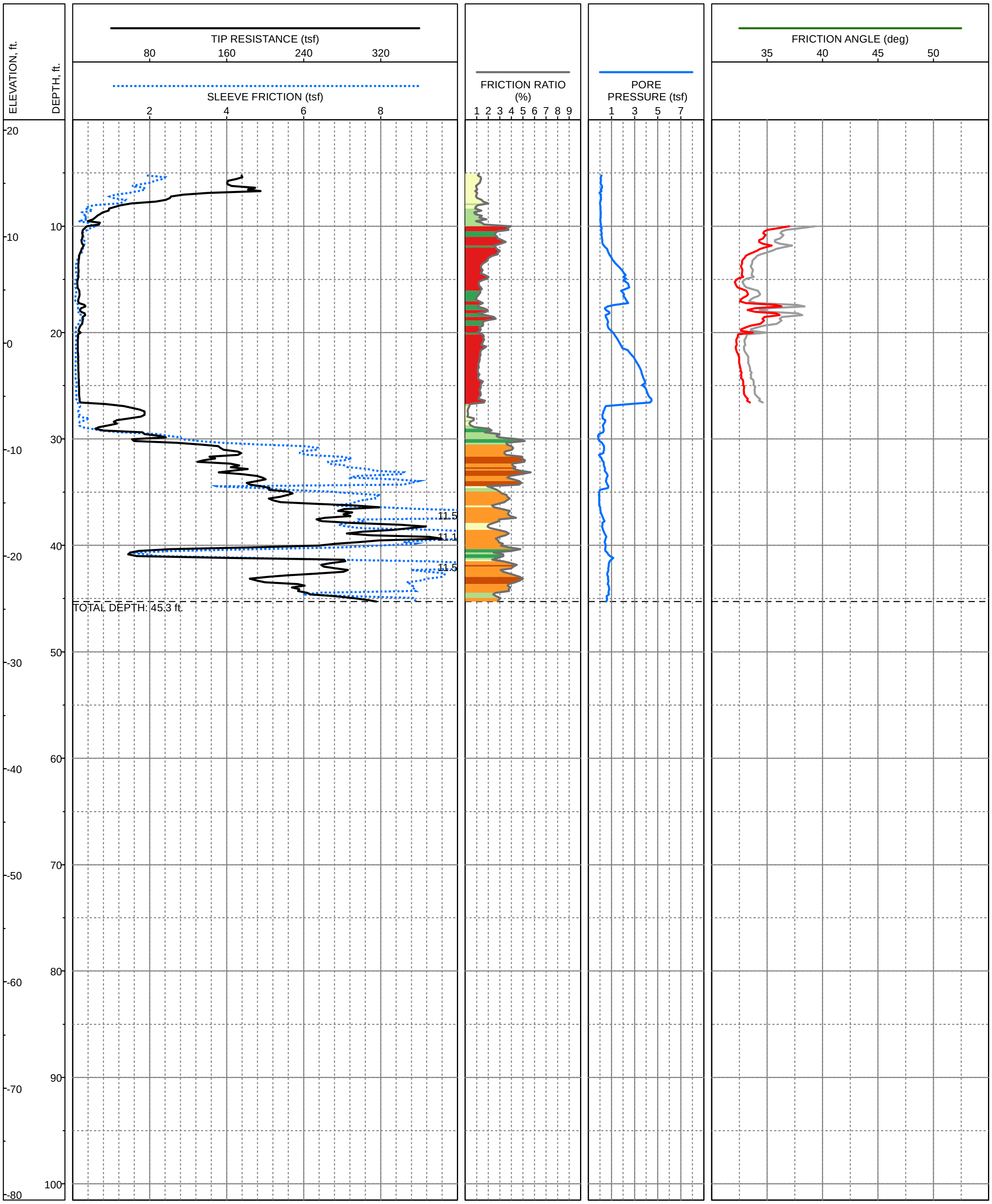
PLATE A-20: LOG OF 2022-CPT-20



LOCATION: E6,052,457, N2,116,815, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.9ft
COMPLETION DEPTH: 80.4ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

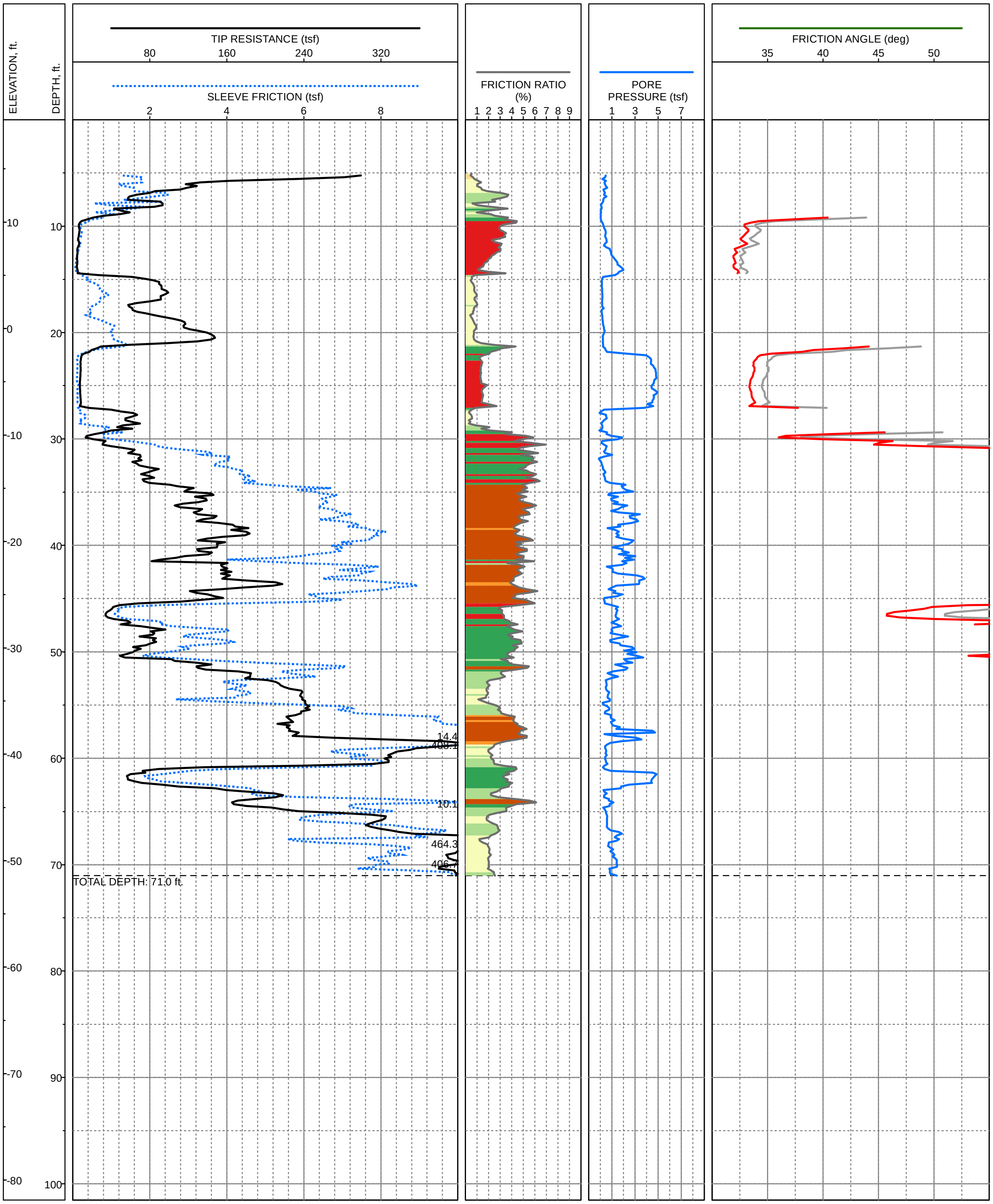
PLATE A-21: LOG OF 2022-CPT-21



LOCATION: E6,052,429, N2,116,770, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 21.0ft
COMPLETION DEPTH: 45.3ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

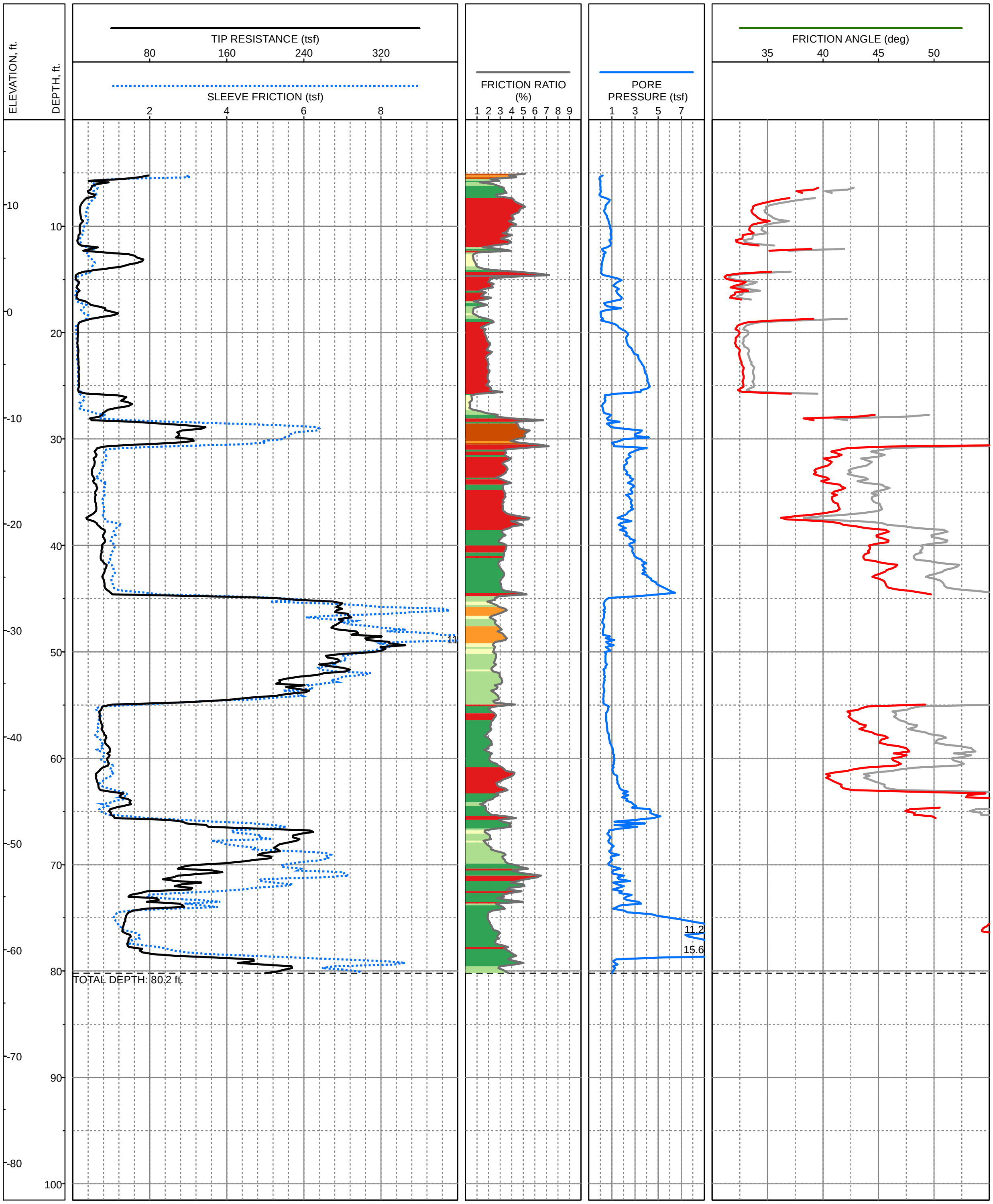
PLATE A-22: LOG OF 2022-CPT-22



LOCATION: E6,052,417, N2,116,734, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 19.6ft
COMPLETION DEPTH: 71.0ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

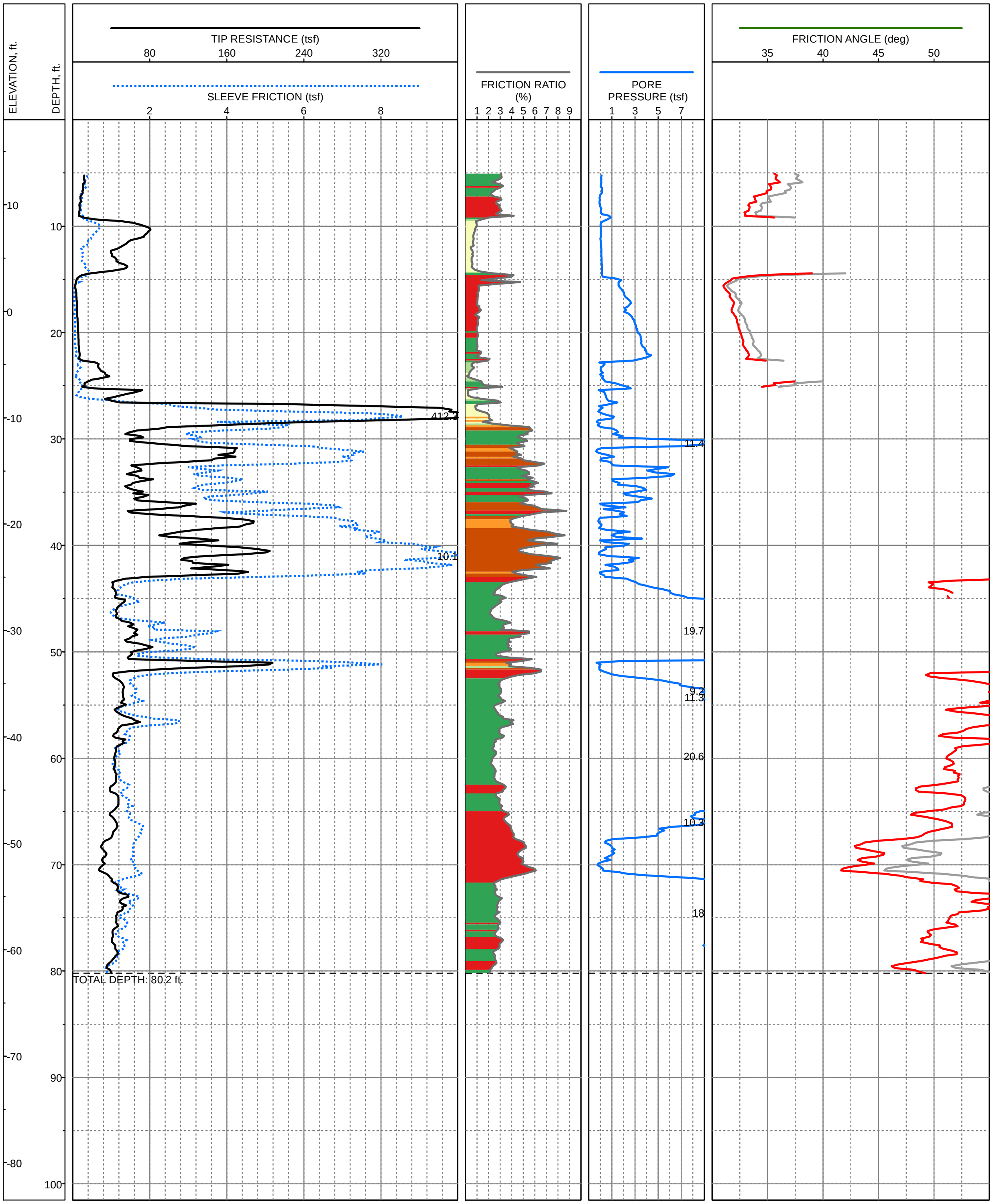
PLATE A-23: LOG OF 2022-CPT-23



LOCATION: E6,052,359, N2,116,755, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.0ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

PLATE A-24: LOG OF 2022-CPT-24

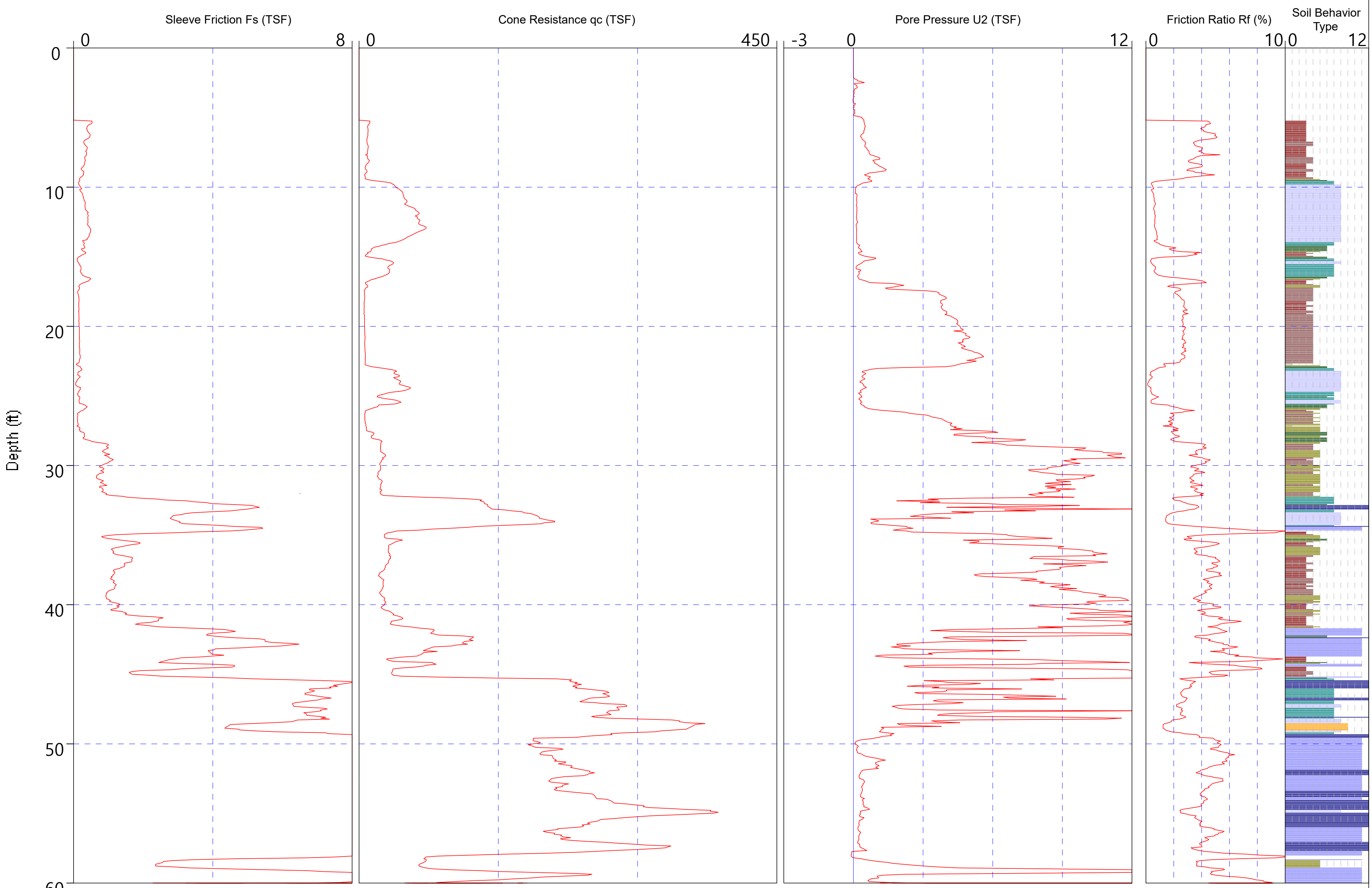


LOCATION: E6,052,339, N2,116,824, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.0ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

PLATE A-25: LOG OF 2022-CPT-25

Job Number:	04.72190021	**CPT Number:**	CPT-01	**Coordinates:**	37.795163 -122.262754
Operator:	Daniel Garza	**Date:**	29-Mar-2019	**Cone Number:**	CP15-CF75PB7SN2-P1E1 2598
Location:	Oakland, CA	**Elevation:**	0.00		



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

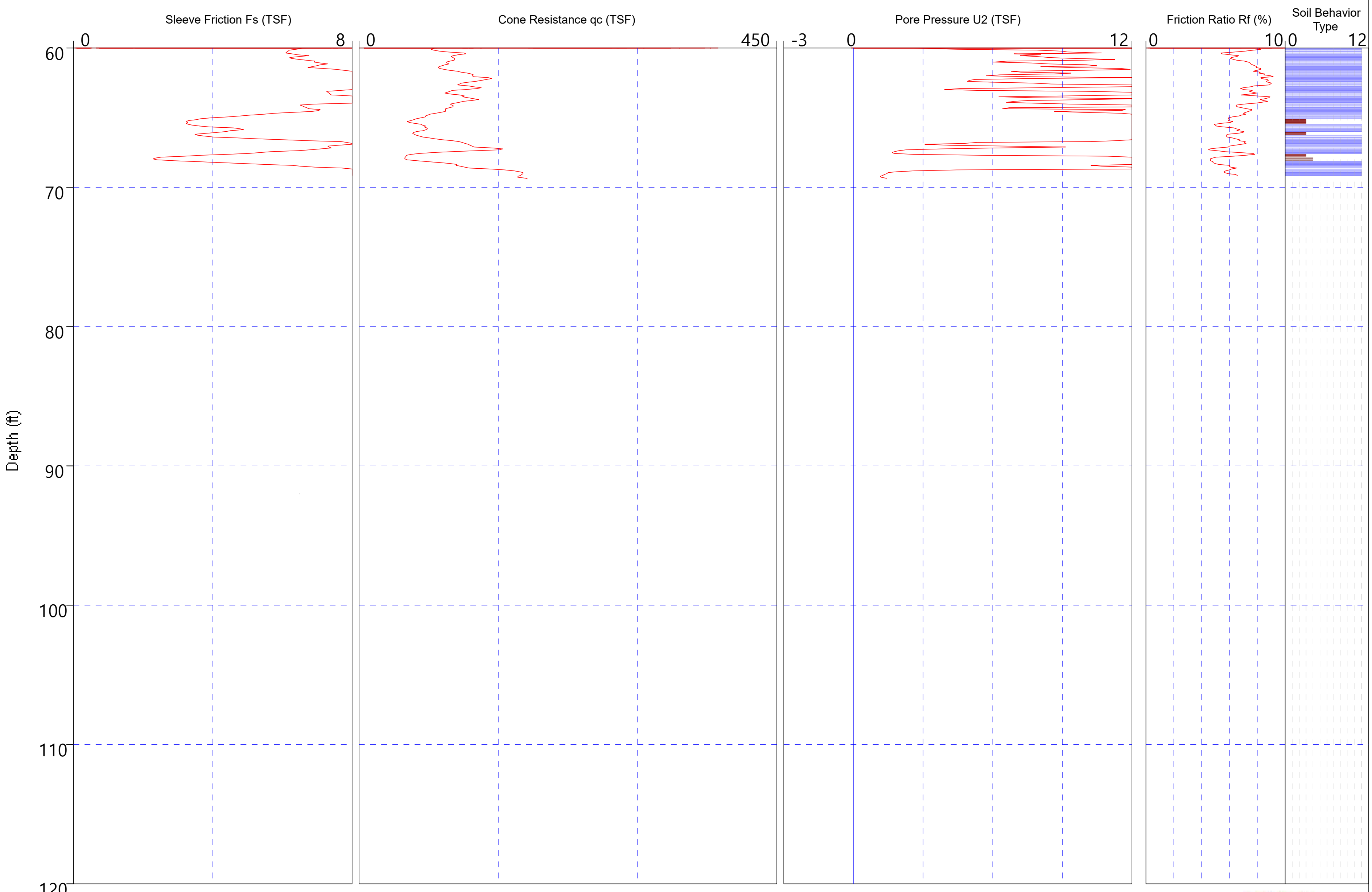
PLATE A-26

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-01
Date: 29-Mar-2019
Elevation: 0.00

Coordinates: 37.795163 -122.262754
Cone Number: CP15-CF75PB7SN2-P1E1 2598



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

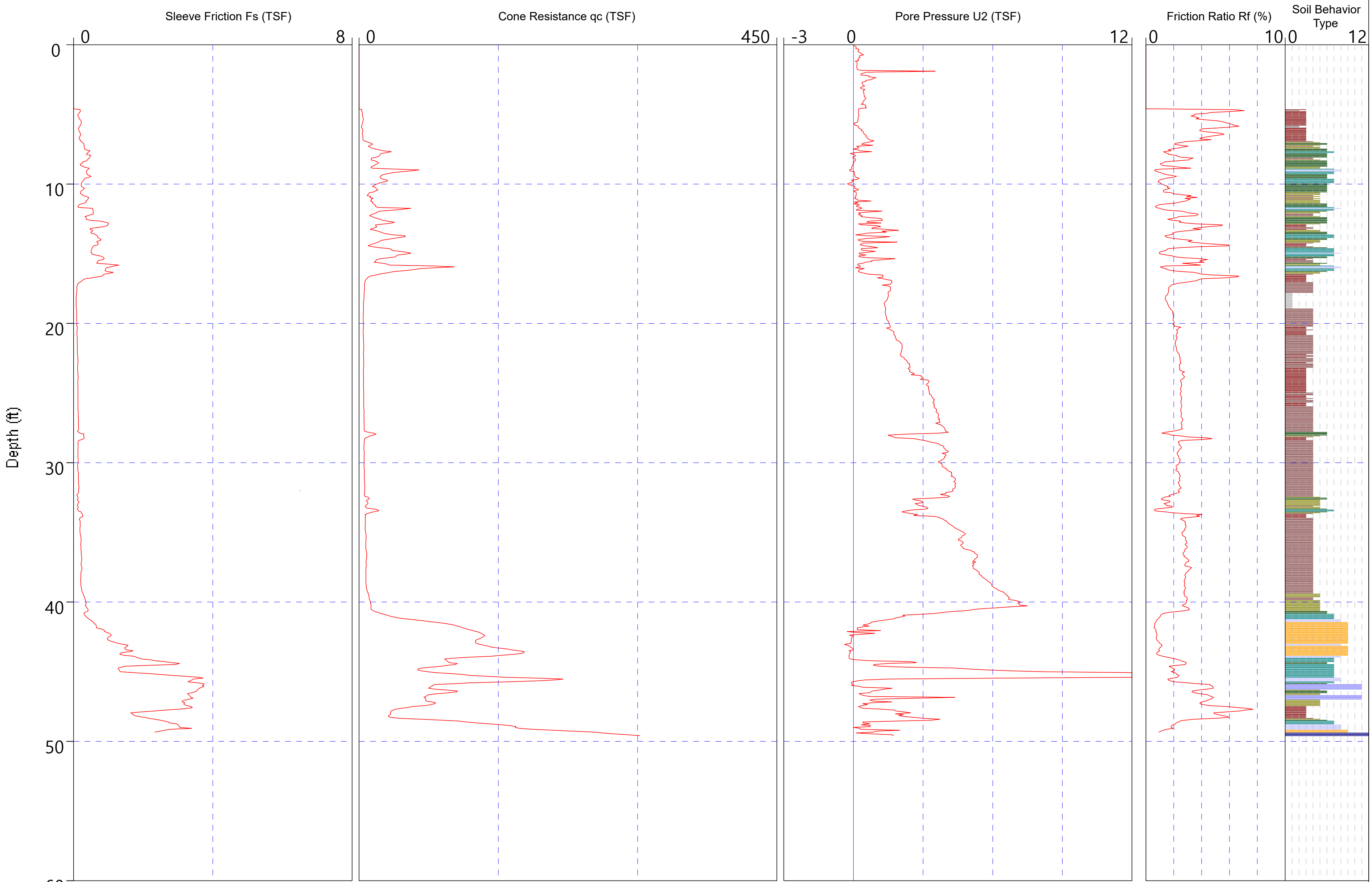
PLATE A-27

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-02
Date: 29-Mar-2019
Elevation: 0.00

Coordinates: 37.794900 -122.261959
Cone Number: CP15-CF75PB7SN2-P1E1 2598

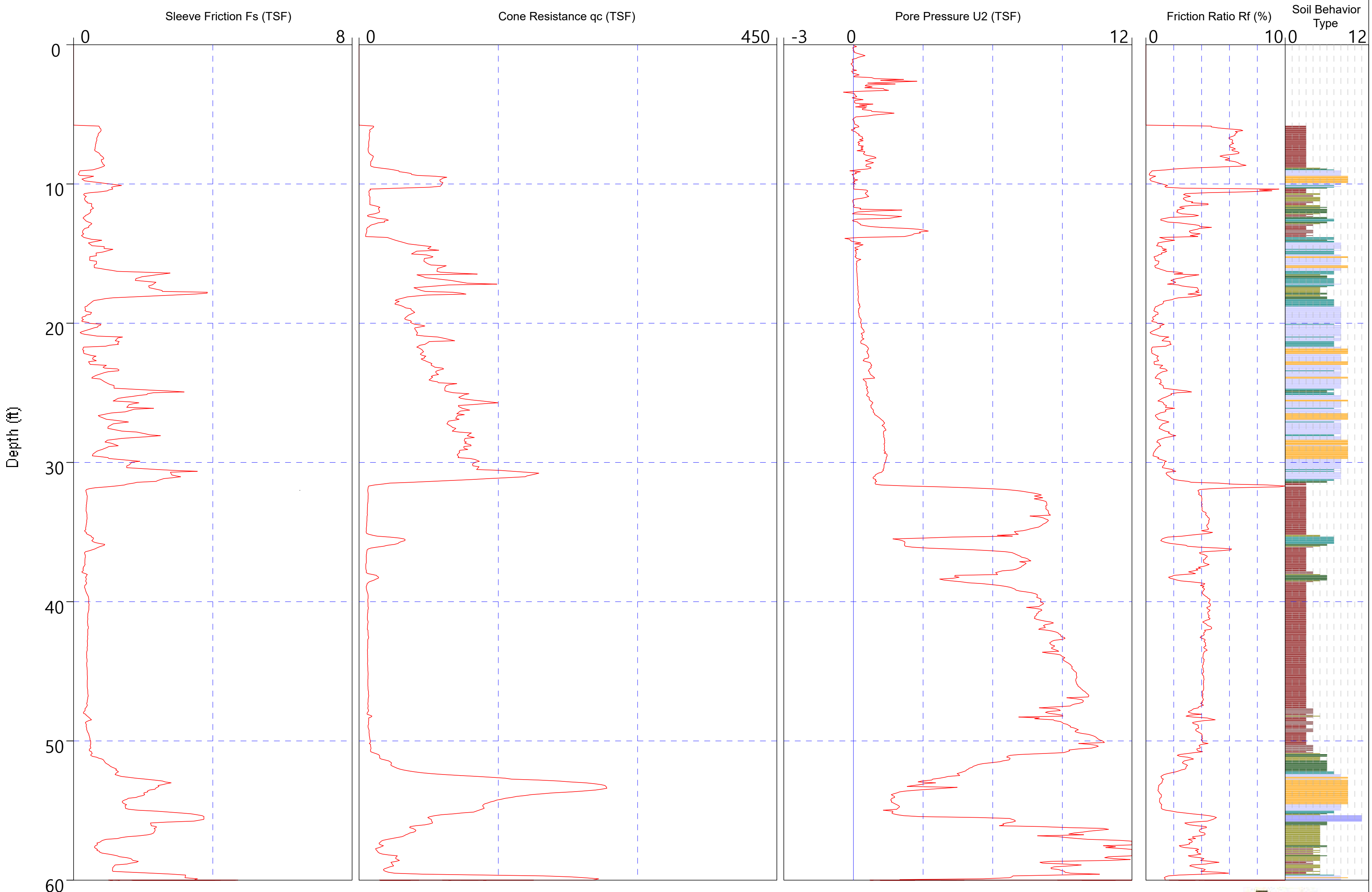


(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

PLATE A-28

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number:	04.72190021	**CPT Number:**	CPT-03	**Coordinates:**	37.794463 -122.262030
Operator:	Daniel Garza	**Date:**	29-Mar-2019	**Cone Number:**	CP15-CF75PB7SN2-P1E1 2598
Location:	Oakland, CA	**Elevation:**	0.00		



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

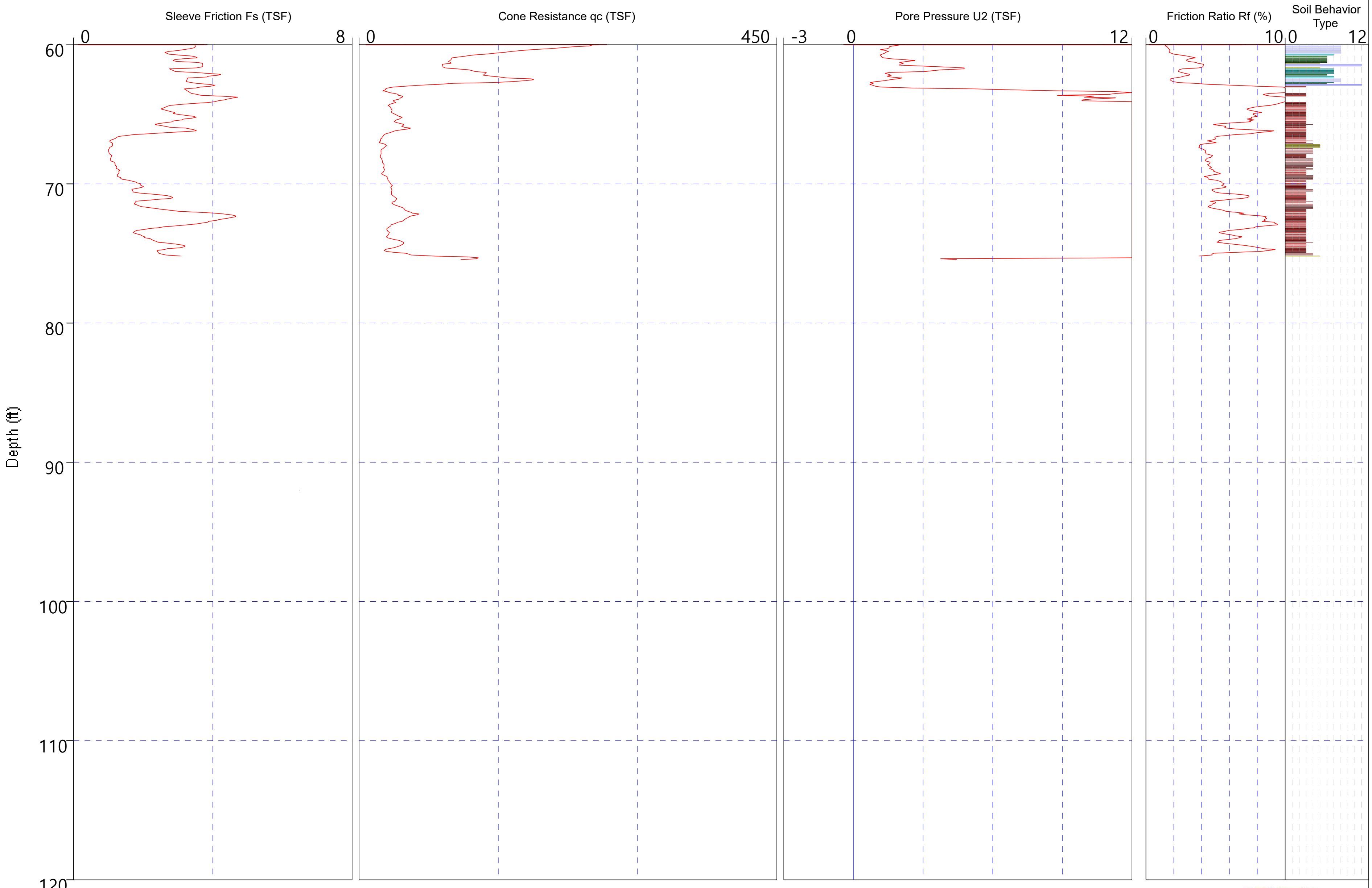
PLATE A-29

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-03
Date: 29-Mar-2019
Elevation: 0.00

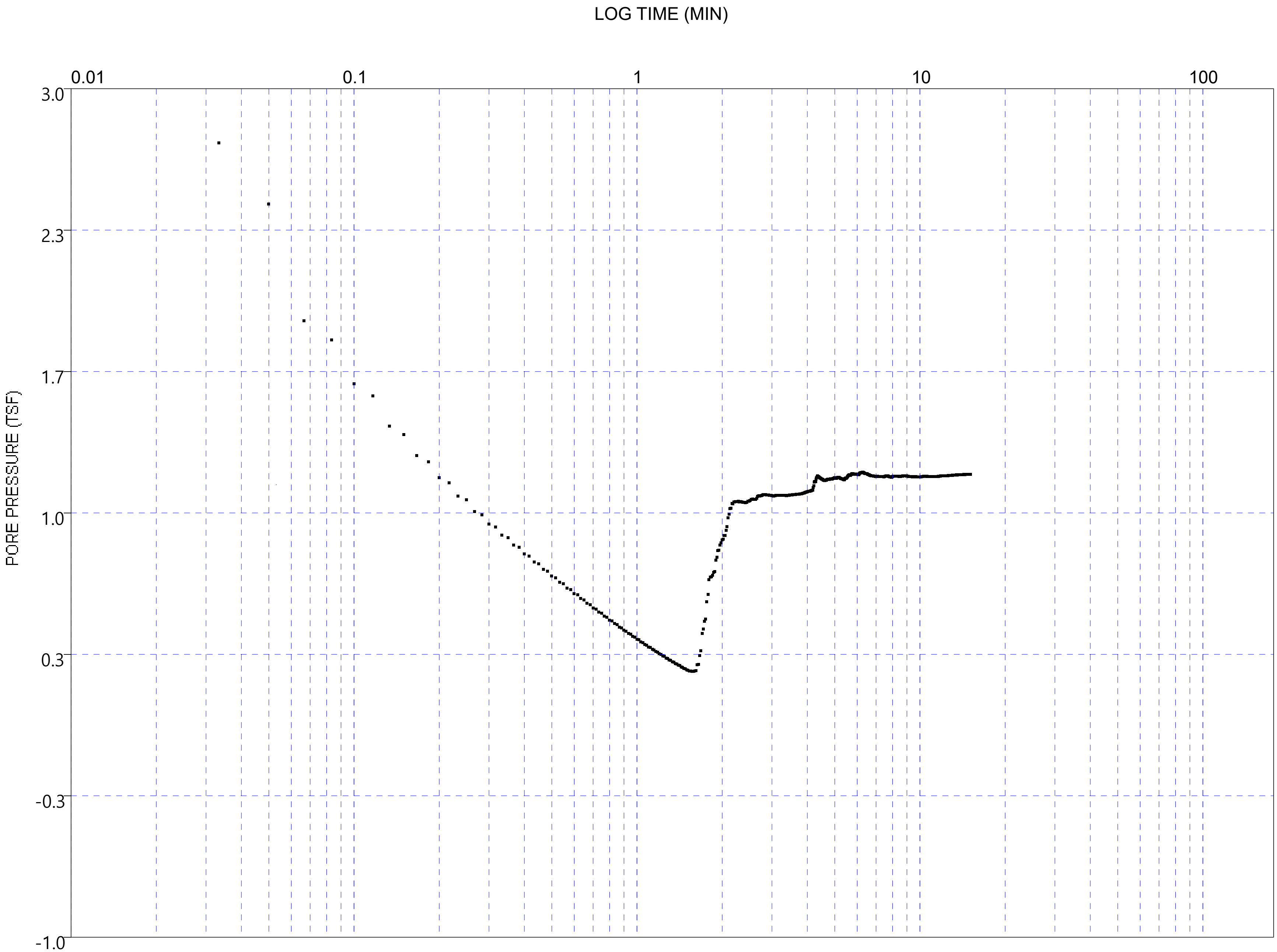
Coordinates: 37.794463 -122.262030
Cone Number: CP15-CF75PB7SN2-P1E1 2598



(1) sensitive fine grained (OL-CH)	(7) silty sand to sandy silt (SM-ML)
(2) organic material (OL-OH)	(8) sand to silty sand (SM-SP)
(3) clay (CH)	(9) sand (SW-SP)
(4) silty clay to clay (CL-CH)	(10) gravel to gravelly sand (SW-GW)
(5) clayey silt to silty clay (MH-CL)	(11) very stiff fine grained* (CH-CL)
(6) sandy silt to clayey silt (ML-MH)	(12) sand to clayey sand* (SC-SM)

PLATE A-30

Robertson et al. 1986 *Overconsolidated or Cemented



DISSIPATION TEST

CPT Number: CPT-01

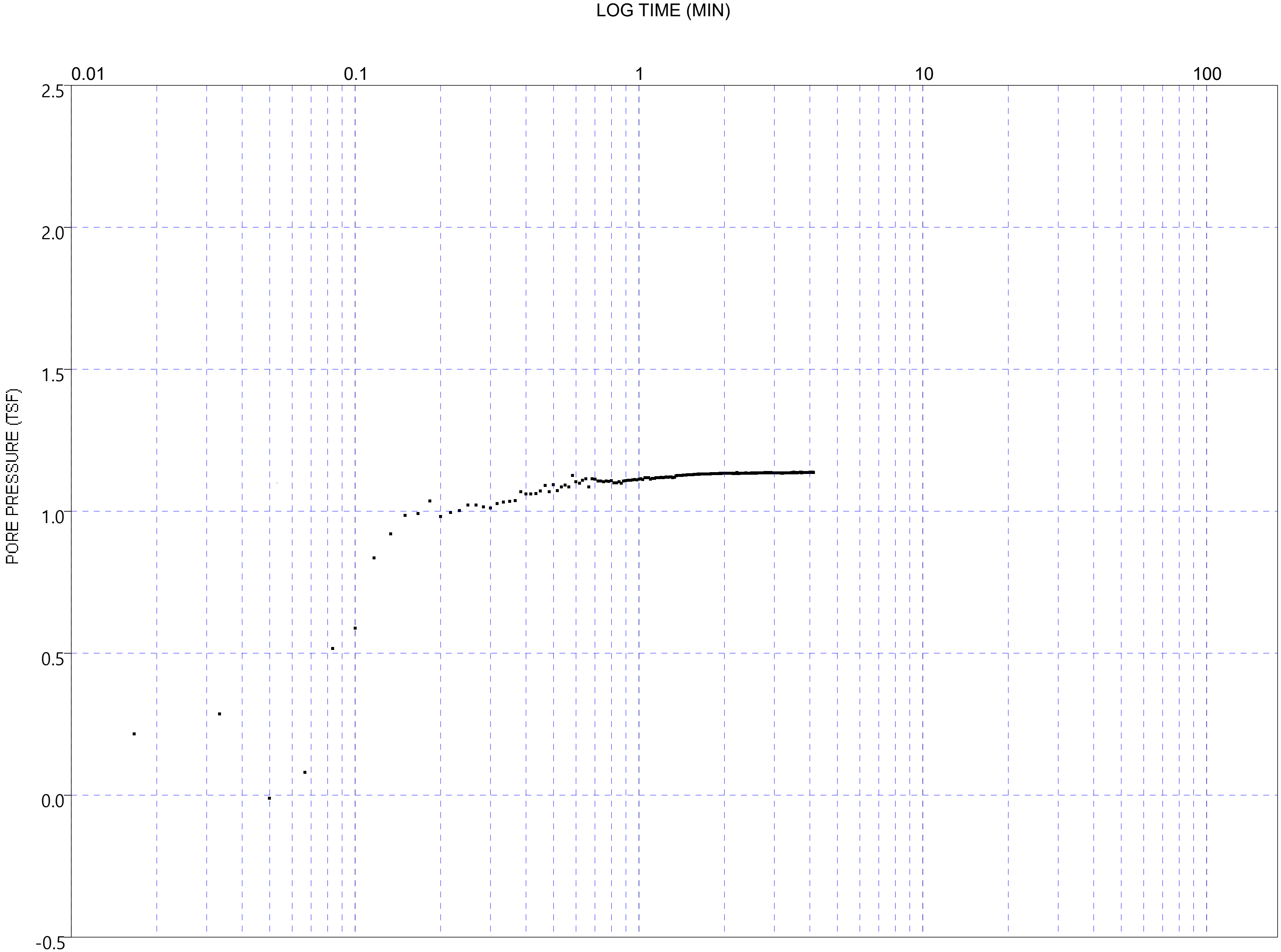
Job Number: 04.72190021

Depth: 48.77

Date: 29-Mar-2019

FUGRO Consultants, Inc.

PLATE A-31



DISSIPATION TEST

CPT Number: CPT-02

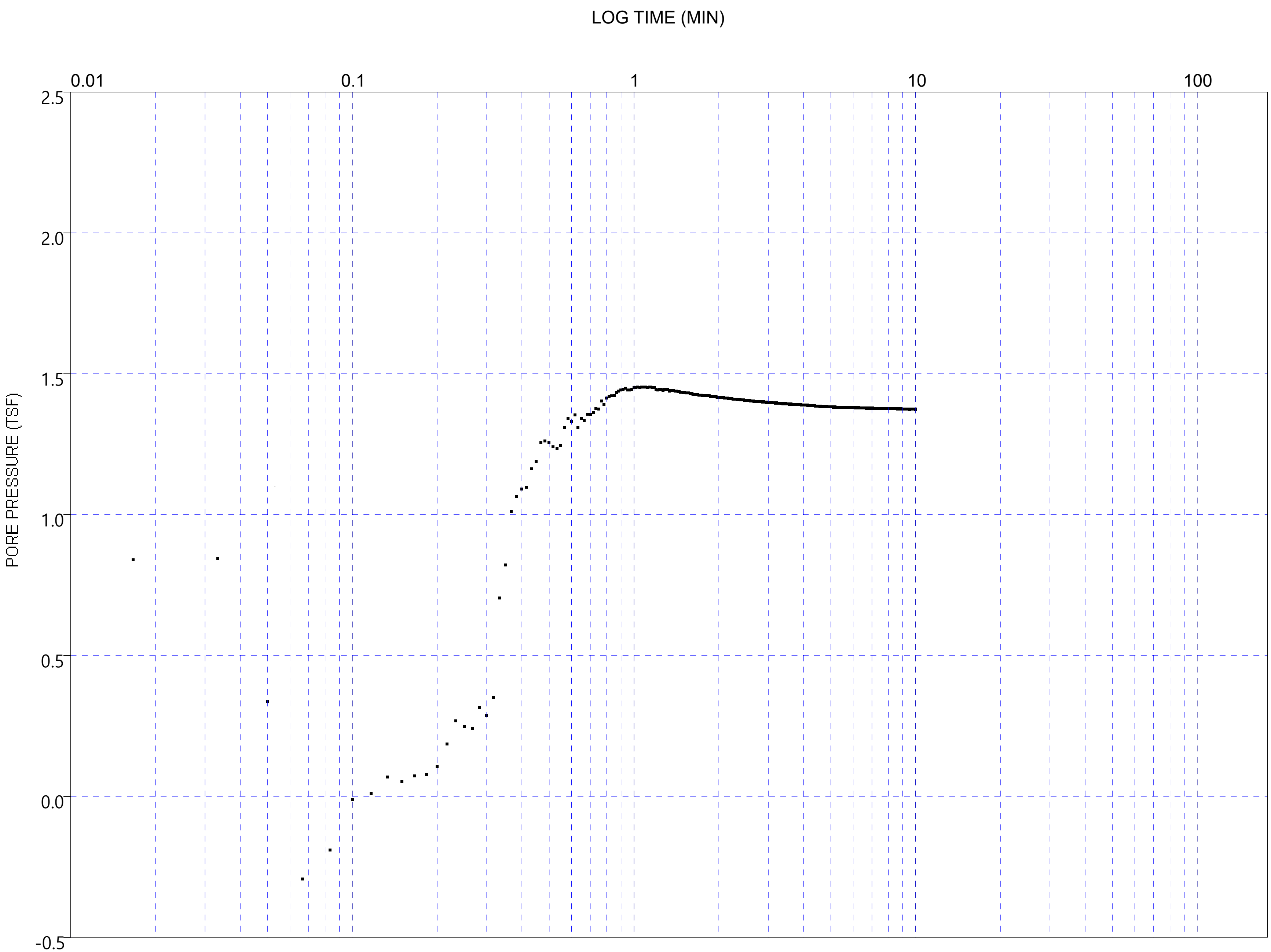
Job Number: 04.72190021

Depth: 42.02

Date: 29-Mar-2019

FUGRO Consultants, Inc.

PLATE A-32



DISSIPATION TEST

CPT Number: CPT-02

Job Number: 04.72190021

Depth: 49.33

Date: 29-Mar-2019

FUGRO Consultants, Inc.

PLATE A-33

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
CPT-04	01/03/2020	75.13	-	-	31.3
CPT-05	01/03/2020	75.13	-	-	41.2
CPT-06	01/03/2020	75.30	-	-	-
SCPT-07	01/03/2020	75.13	-	-	57.6
CPT-08	01/02/2020	51.67	-	-	51.7

2726 Walnut Ave. • Signal Hill, California 90755 • (562) 427-6899 • FAX (562) 427-3314
950 Howe Road. • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

Gregg Drilling, LLC
www.greggdrilling.com

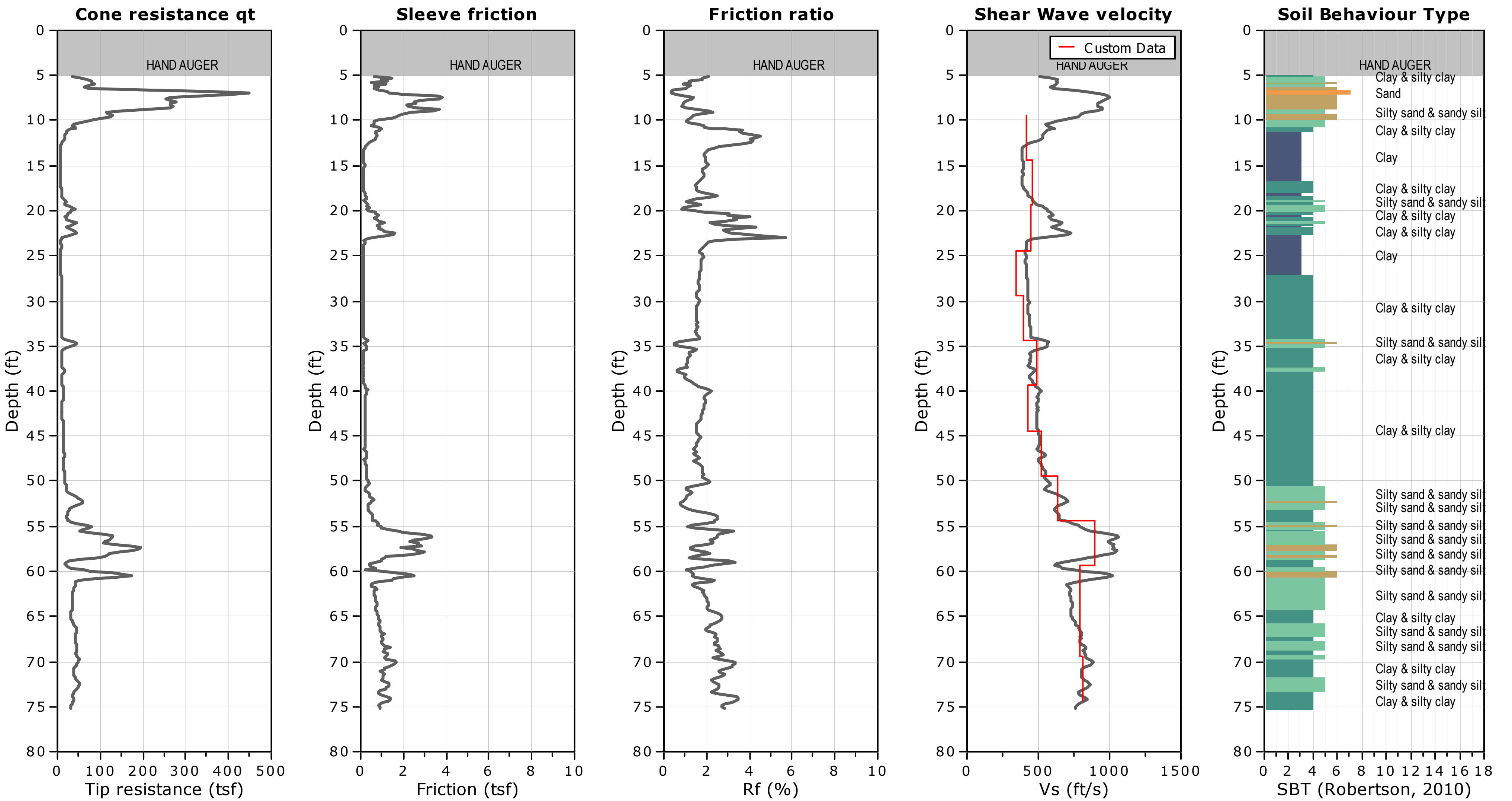
CPT: SCPT-07

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

PLATE A-35

Gregg Drilling, LLC
www.greggdrilling.com

CPT: CPT-04

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020

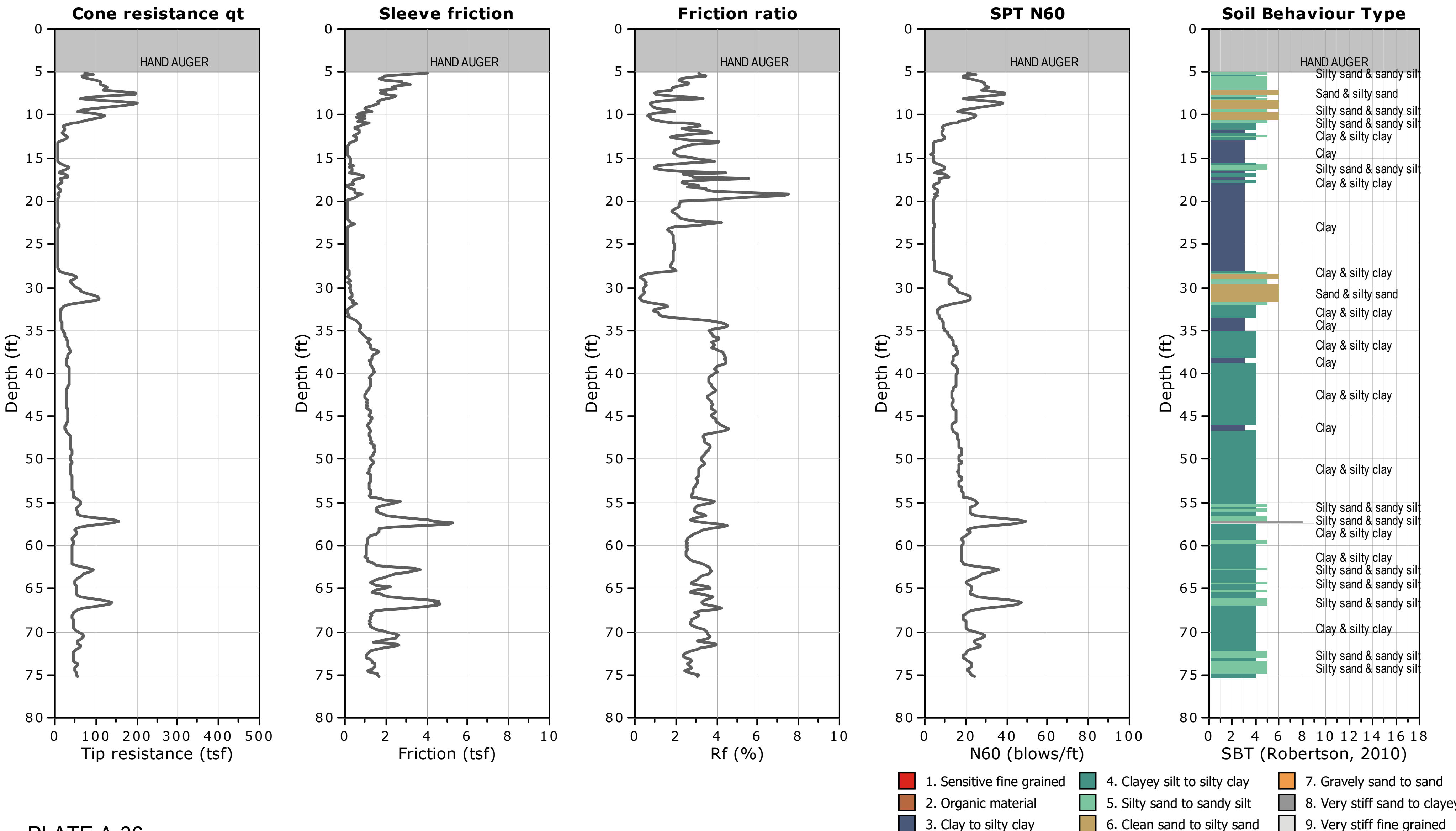


PLATE A-36

Gregg Drilling, LLC
www.greggdrilling.com

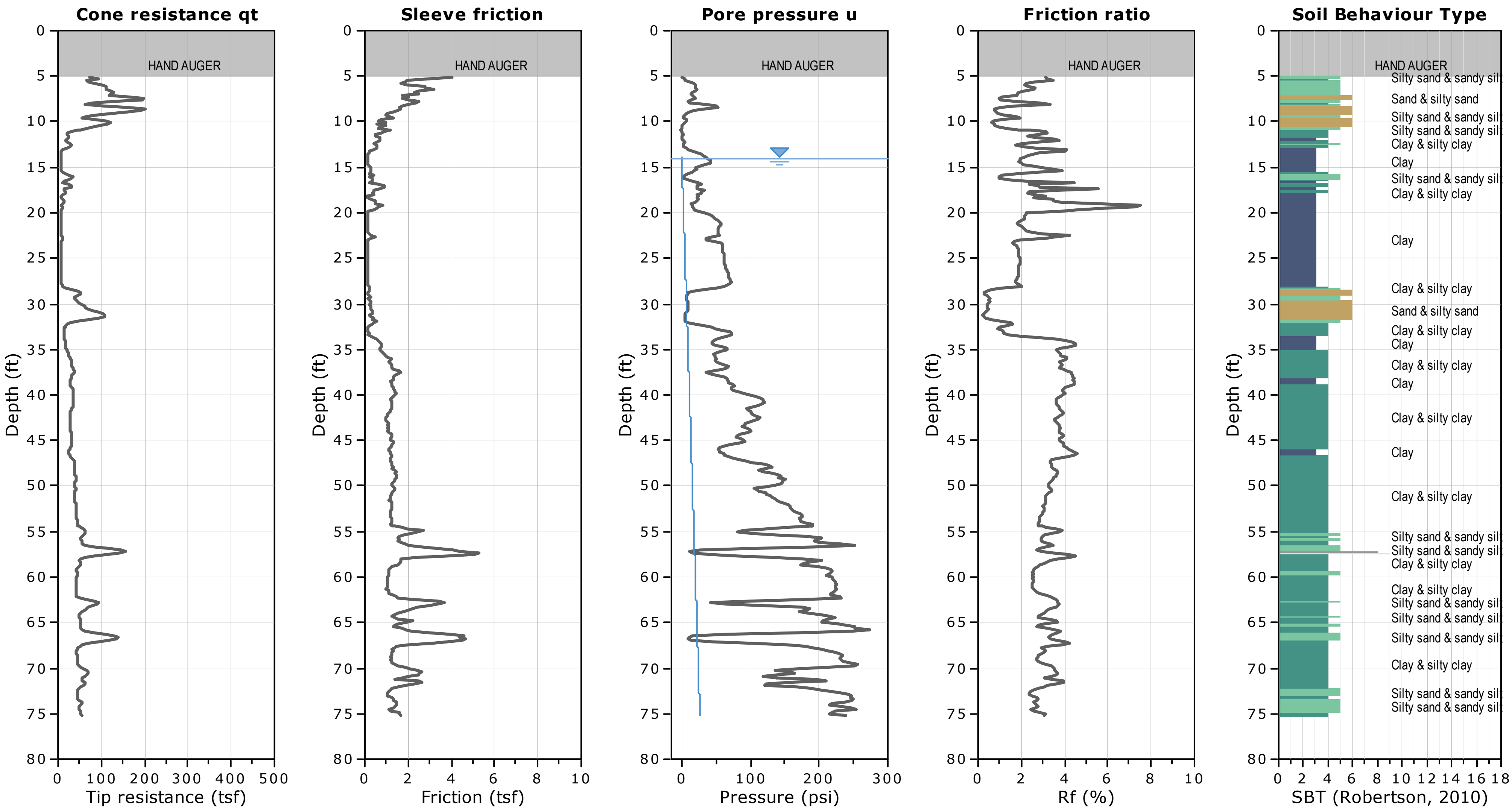
CPT: CPT-04

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-37

Gregg Drilling, LLC
www.greggdrilling.com

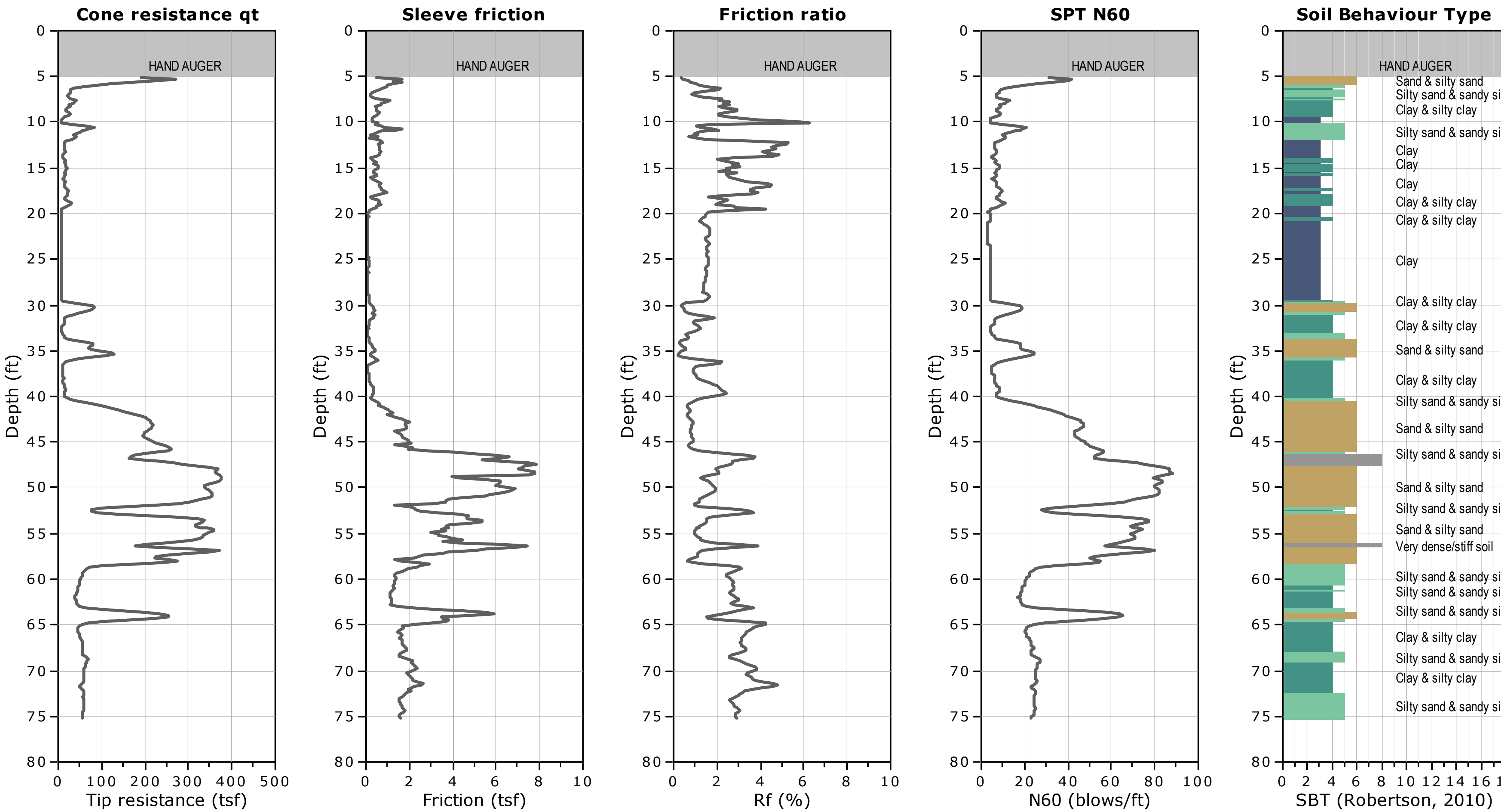
CPT: CPT-05

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

PLATE A-38

Gregg Drilling, LLC
www.greggdrilling.com

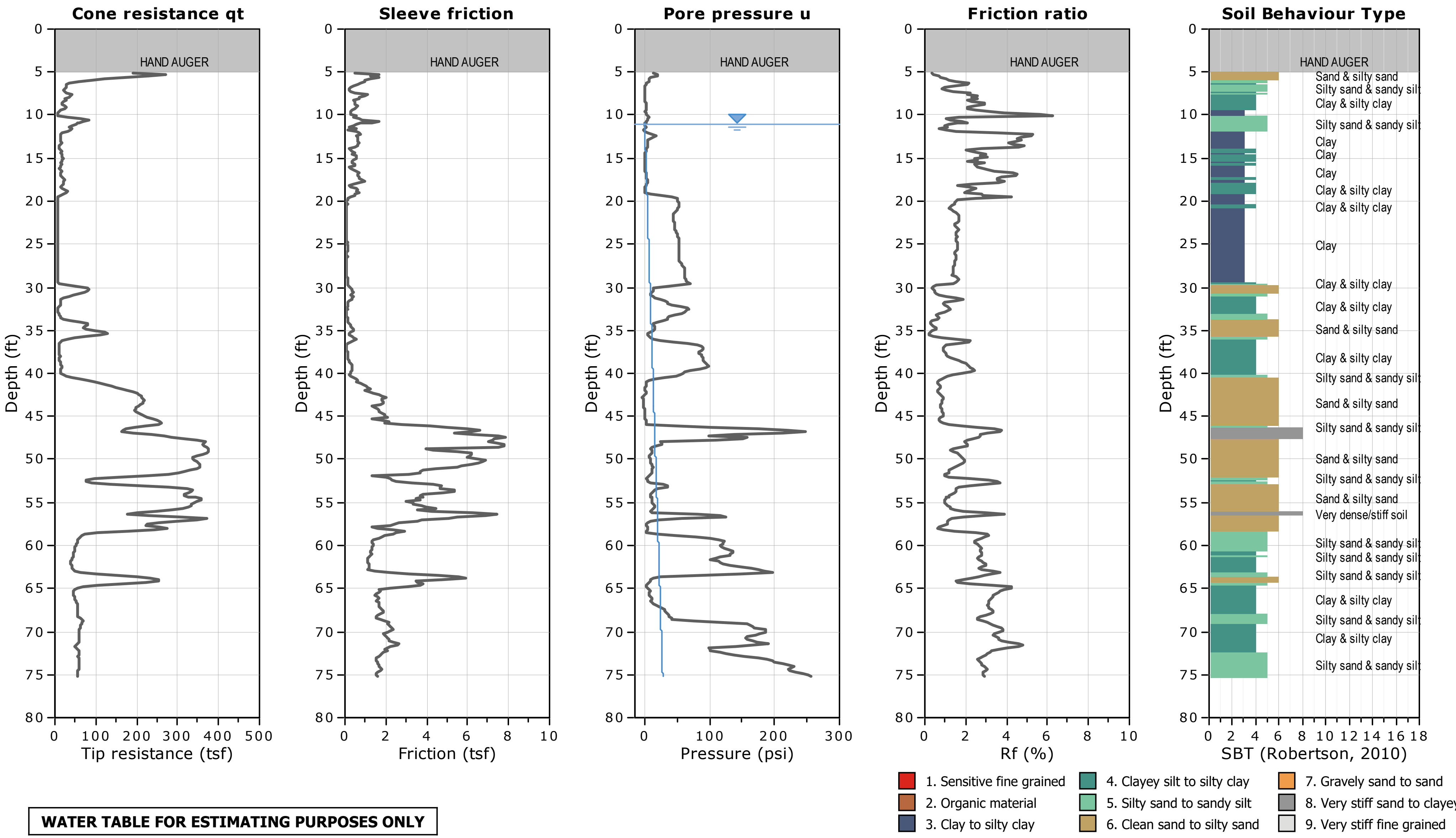
CPT: CPT-05

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-39

Gregg Drilling, LLC
www.greggdrilling.com

CPT: CPT-06

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.30 ft, Date: 1/3/2020

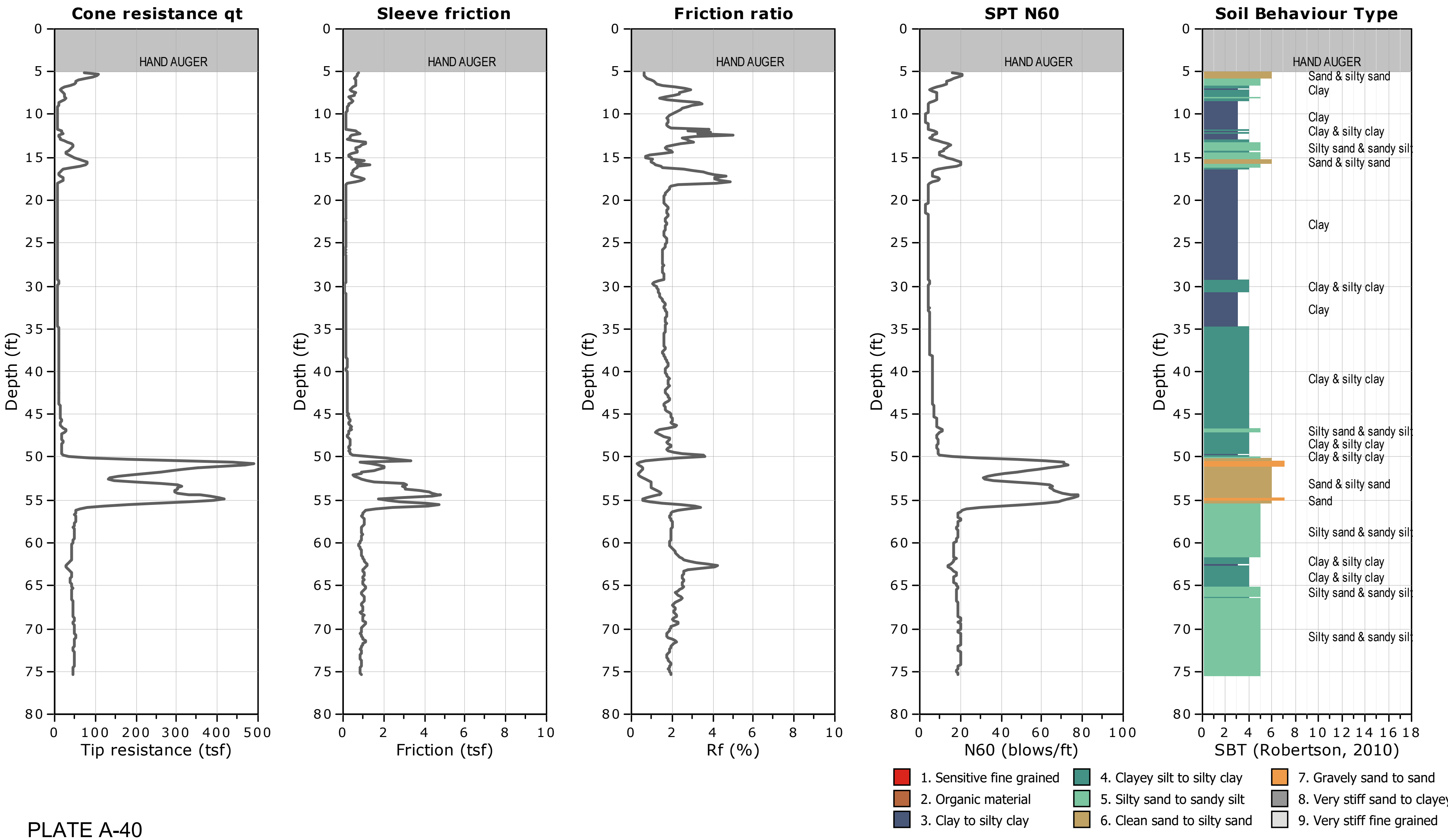


PLATE A-40

Gregg Drilling, LLC
www.greggdrilling.com

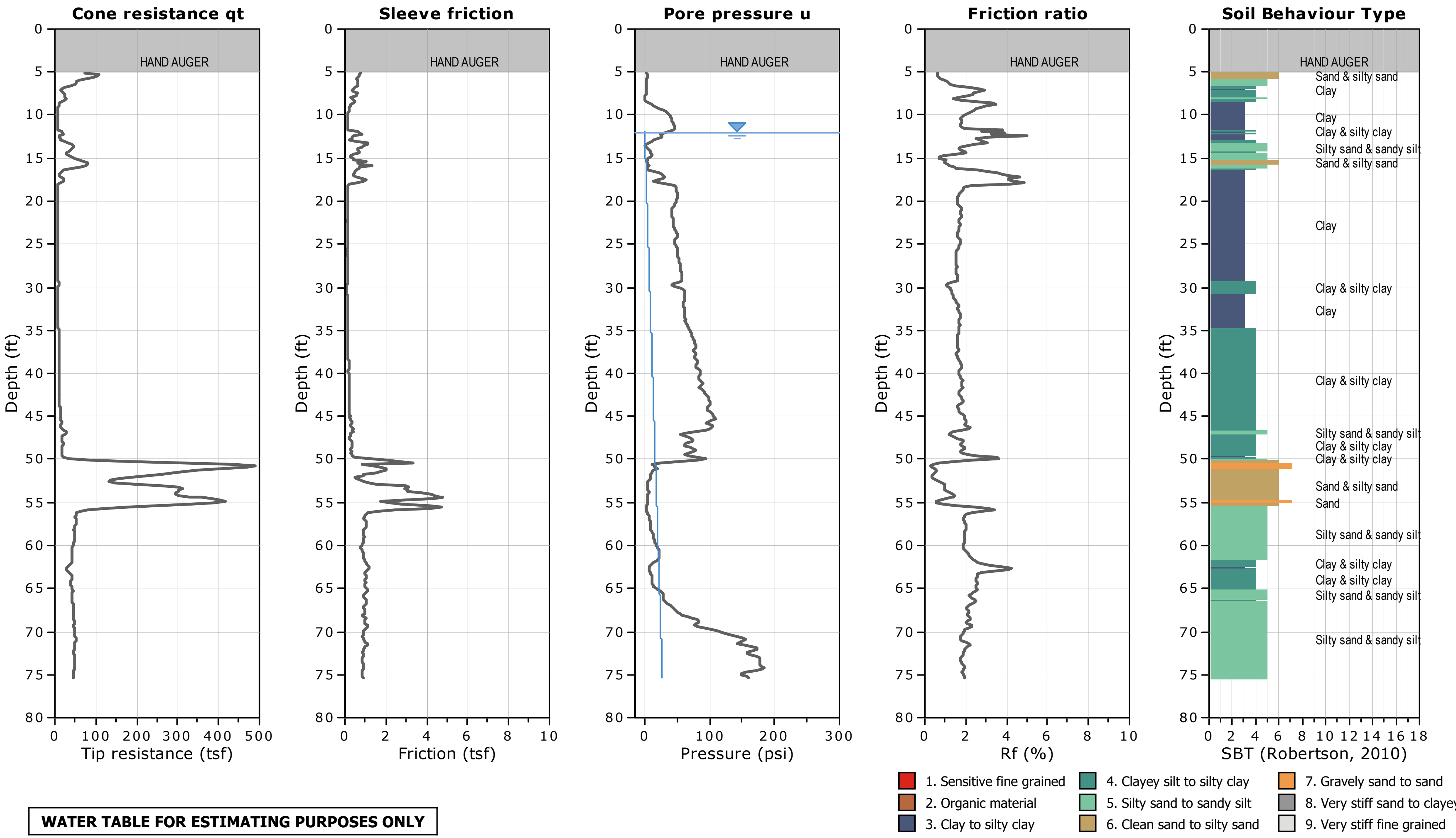
CPT: CPT-06

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.30 ft, Date: 1/3/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-41

Gregg Drilling, LLC
www.greggdrilling.com

CPT: SCPT-07

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020

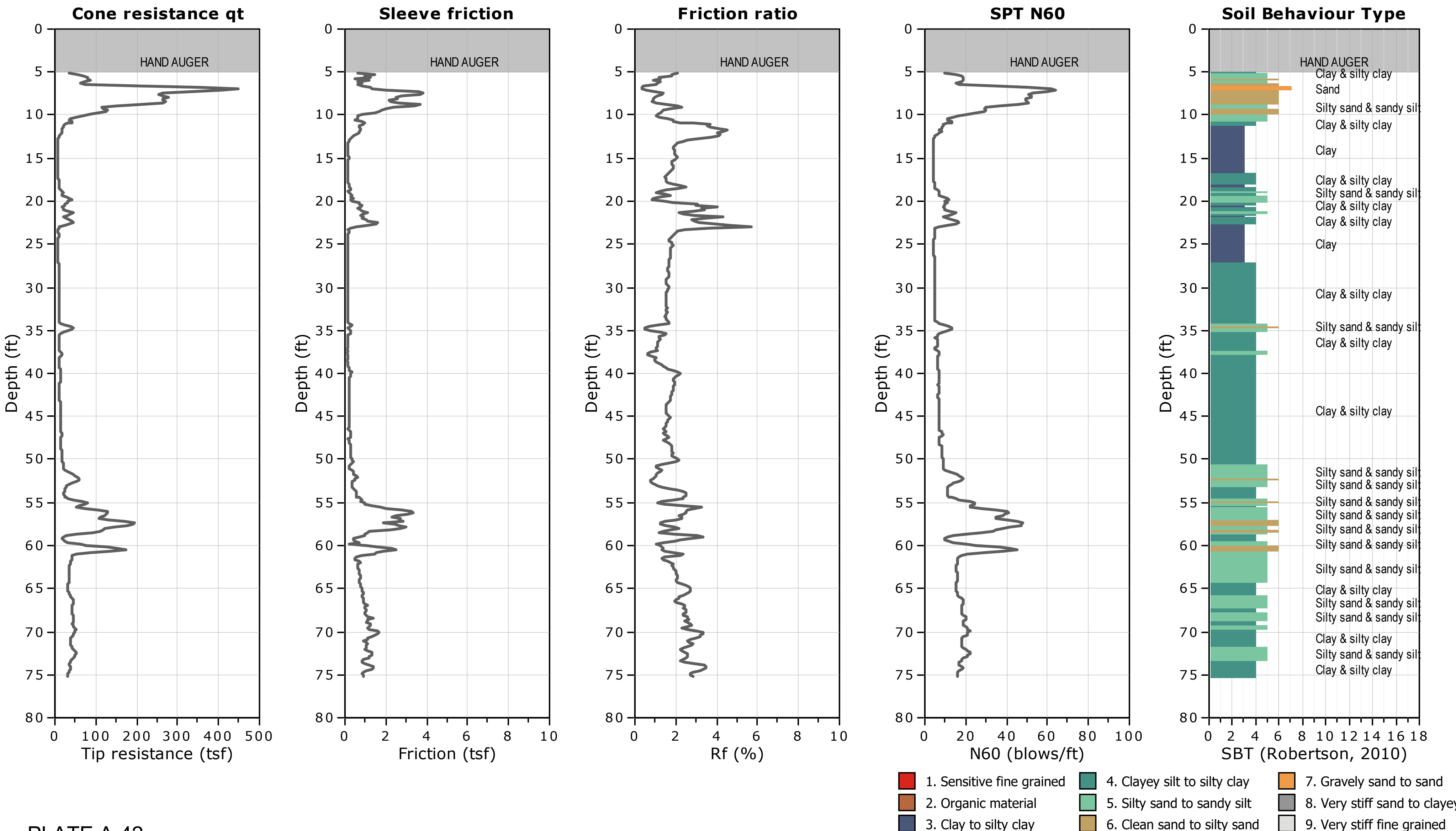


PLATE A-42

Gregg Drilling, LLC
www.greggdrilling.com

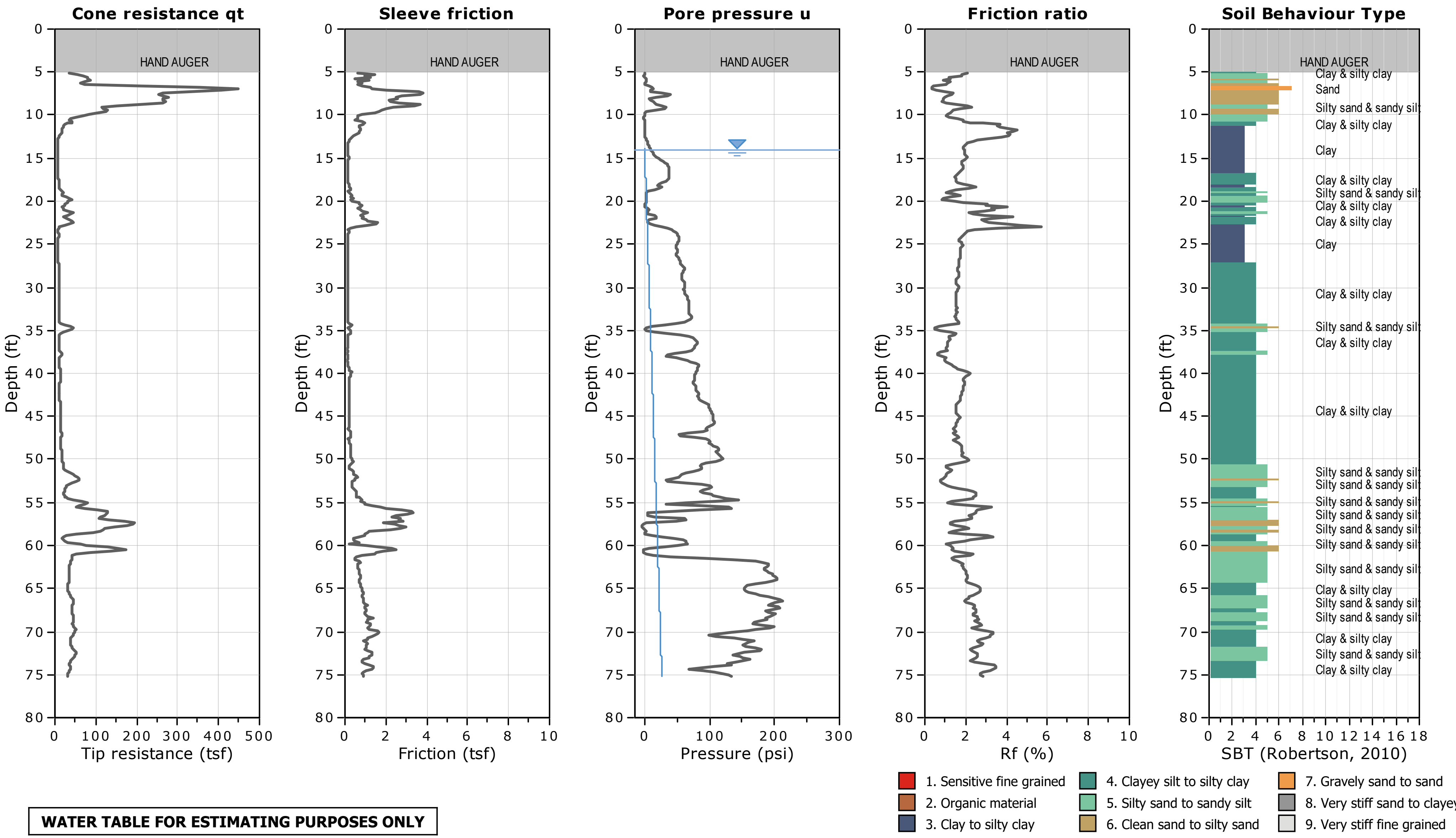
CPT: SCPT-07

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-43

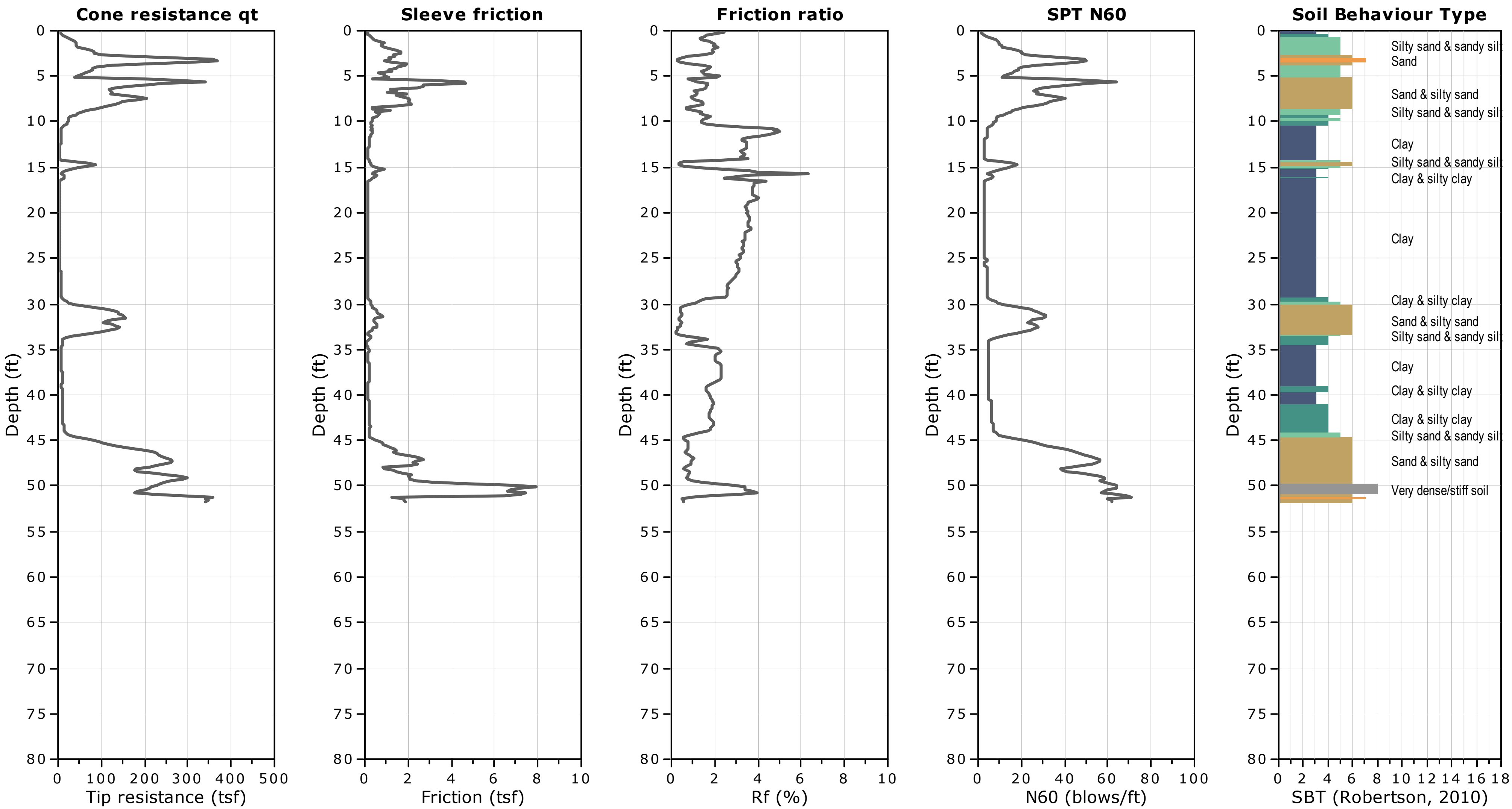
GREGG

Gregg Drilling, LLC
www.greggdrilling.com

CPT: CPT-08

CLIENT: FUGRO
SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD
Total depth: 51.67 ft, Date: 1/2/2020



1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

PLATE A-44

Gregg Drilling, LLC
www.greggdrilling.com

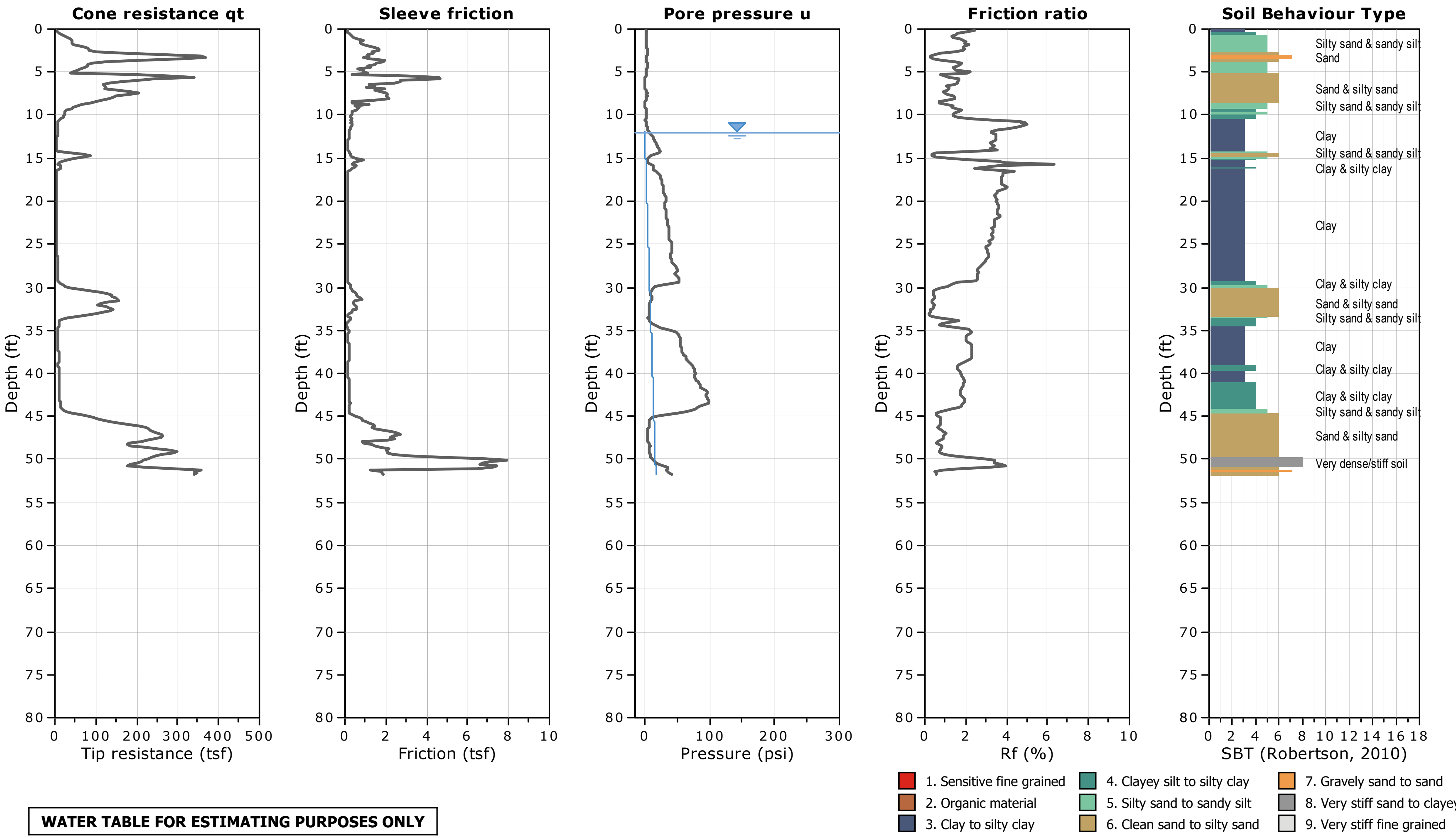
CPT: CPT-08

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 51.67 ft, Date: 1/2/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-45

Waveforms for Sounding SCPT-07

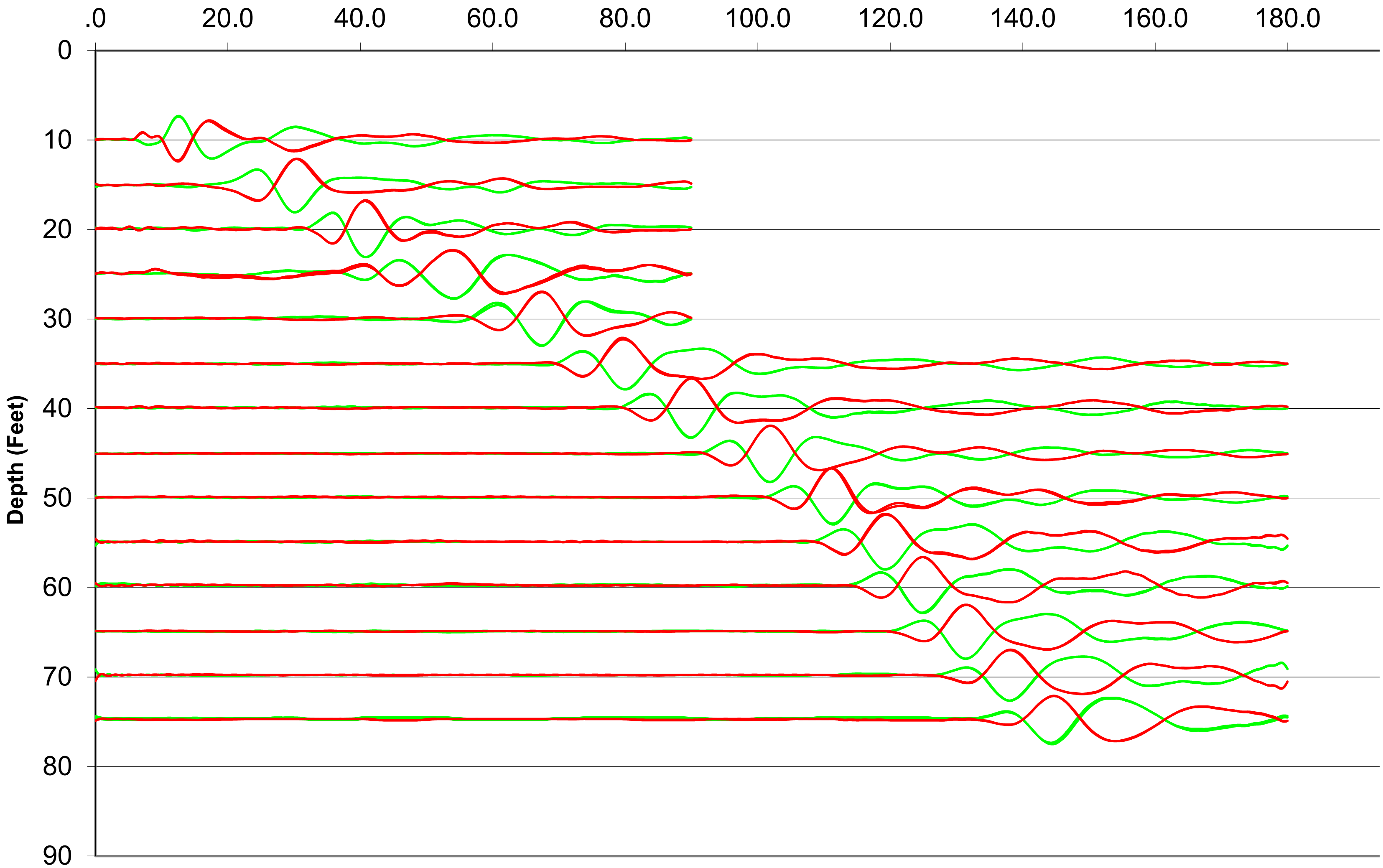


PLATE A-46

Shear Wave Velocity Calculations

Laney College

SCPT-07

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

01/03/20

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	9.49	9.49	14.8000			
15.09	14.43	14.53	5.03	27.0000	12.2000	412.6	11.89
20.01	19.35	19.42	4.90	37.7000	10.7000	457.7	16.89
25.10	24.44	24.50	5.07	49.0500	11.3500	446.7	21.90
30.02	29.36	29.41	4.91	63.5000	14.4500	339.9	26.90
35.10	34.44	34.49	5.08	76.2500	12.7500	398.3	31.90
40.03	39.37	39.40	4.92	86.2000	9.9500	494.1	36.91
45.11	44.45	44.48	5.08	98.1500	11.9500	425.2	41.91
50.03	49.37	49.40	4.92	107.6500	9.5000	517.7	46.91
55.12	54.46	54.48	5.08	115.6000	7.9500	639.3	51.92
60.04	59.38	59.40	4.92	121.1000	5.5000	894.4	56.92
65.12	64.46	64.49	5.08	127.5500	6.4500	788.1	61.92
70.05	69.39	69.41	4.92	133.8000	6.2500	787.2	66.93
75.13	74.47	74.49	5.08	140.0500	6.2500	813.4	71.93

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-04
Depth (ft): 31.33
Site: Laney College
Engineer: Reza Rahimnejad

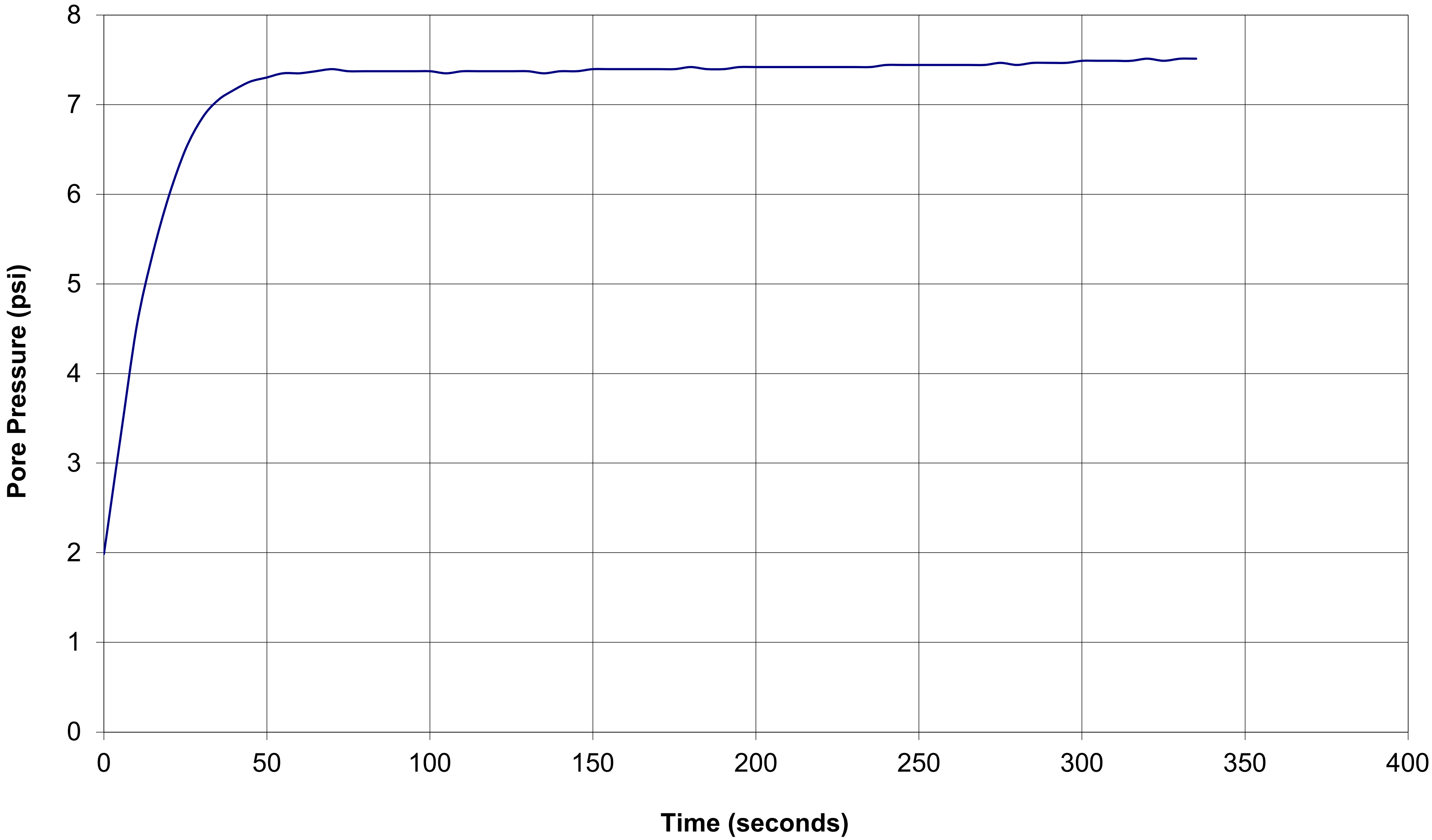


PLATE A-48

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-05
Depth (ft): 41.17
Site: Laney College
Engineer: Reza Rahimnejad

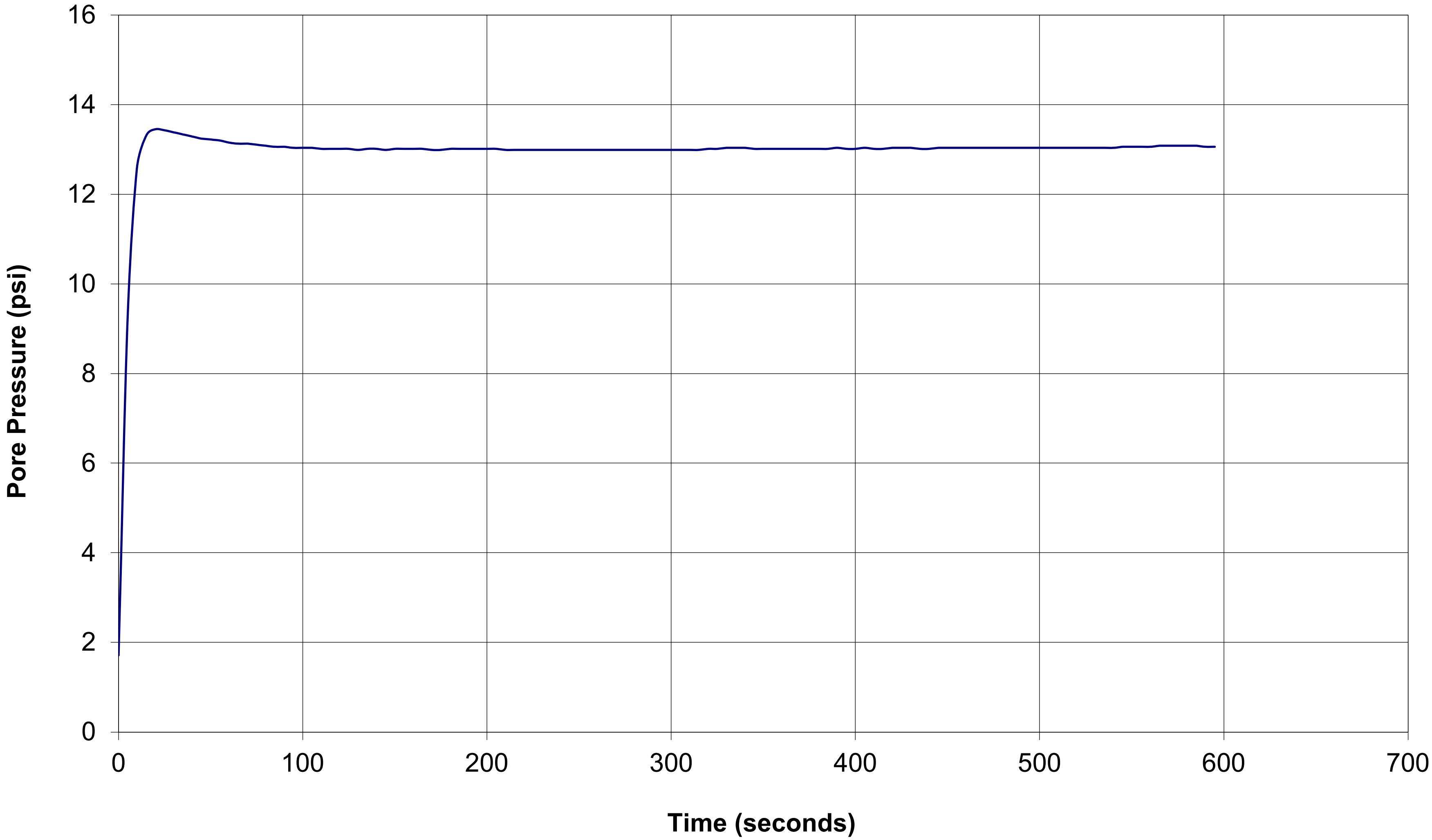


PLATE A-49

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-08
Depth (ft): 51.67
Site: Laney College
Engineer: Reza Rahimnejad

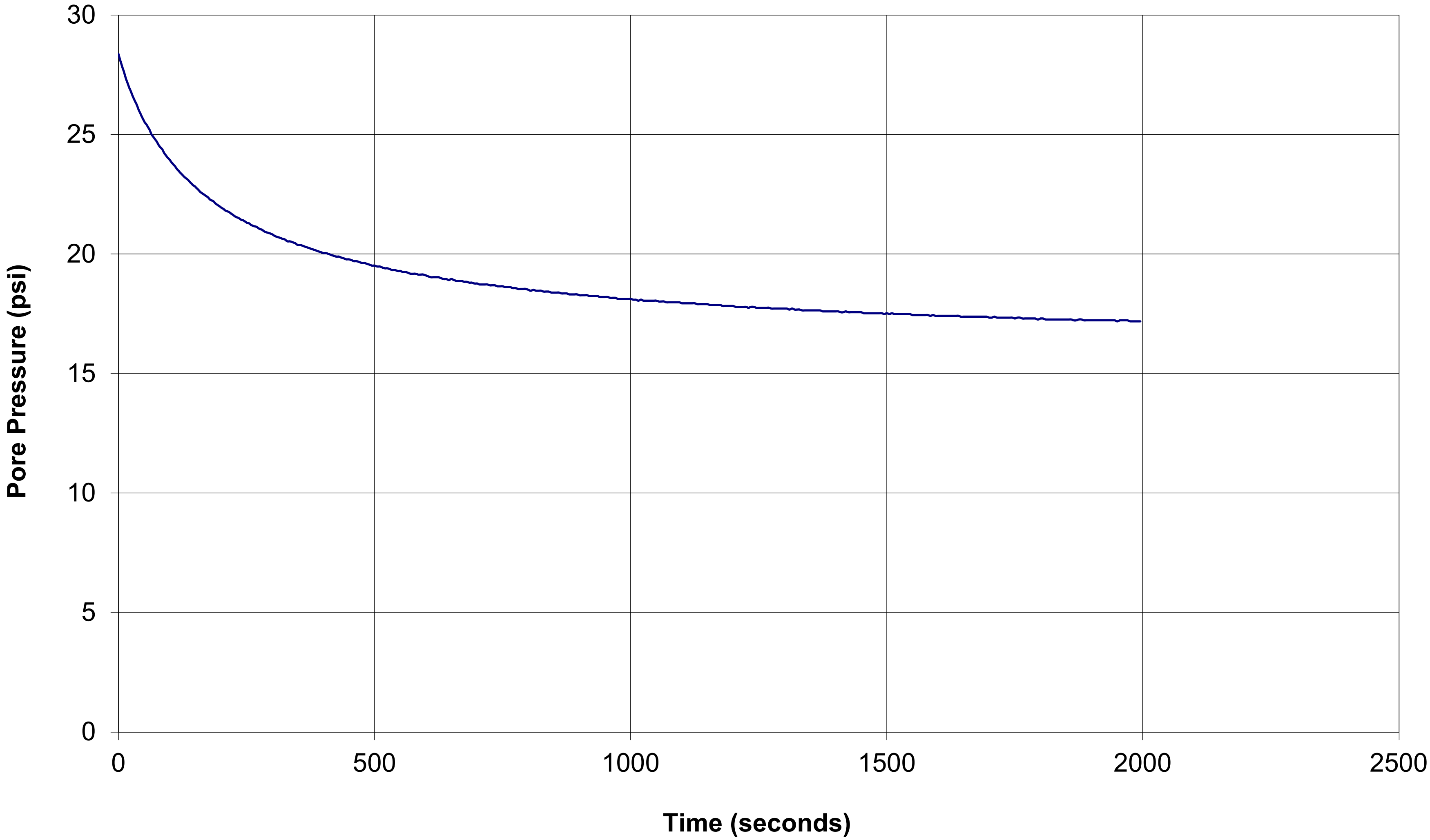


PLATE A-50

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: SCPT-07
Depth (ft): 57.58
Site: Laney College
Engineer: Reza Rahimnejad

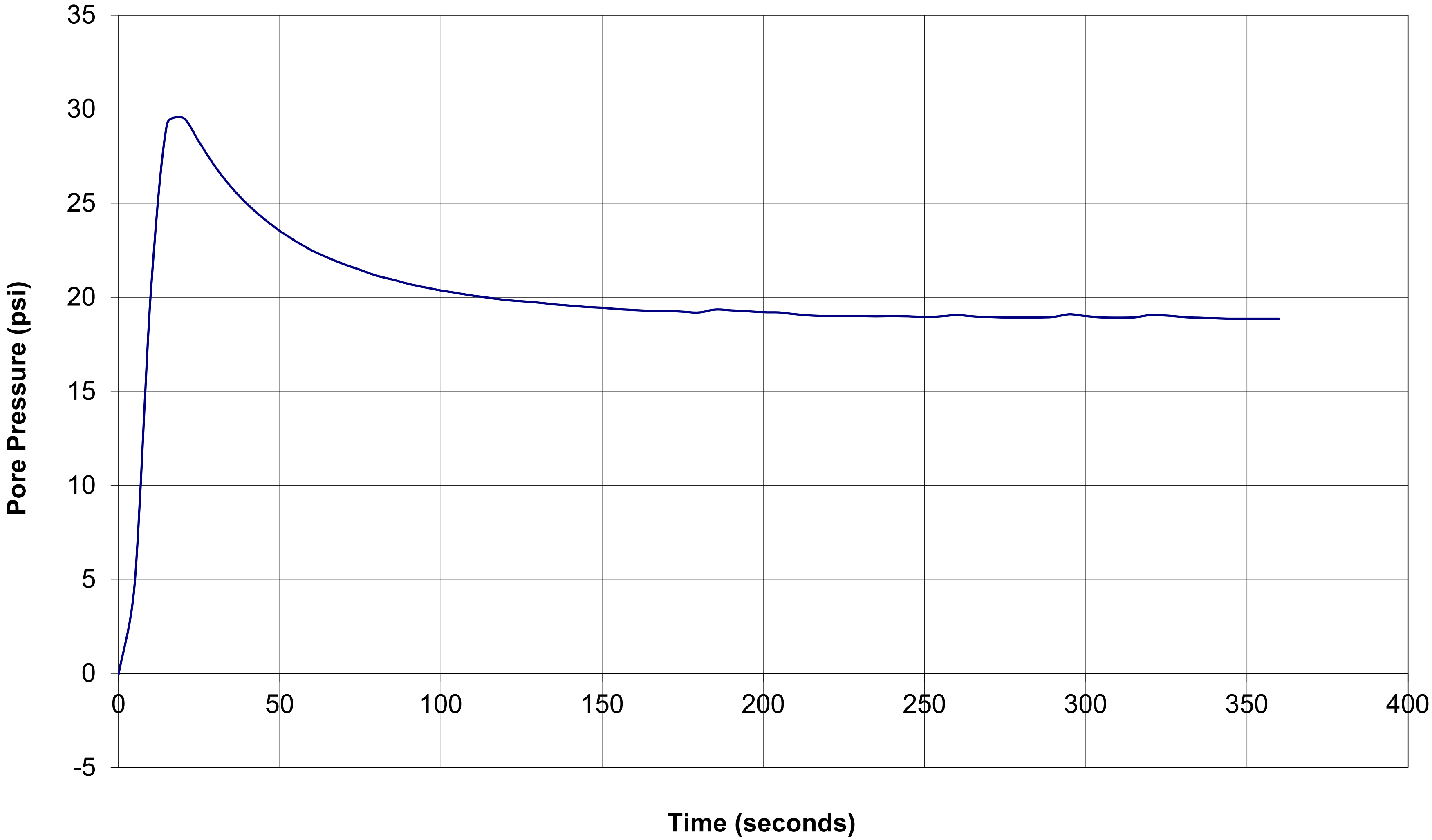


PLATE A-51

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-16	11/22/2022	100.23	-	-	48.06
CPT-17	11/22/2022	100.23	-	-	-
CPT-18	11/18/2022	80.22	-	-	-
CPT-19	11/17/2022	65.29	-	-	30.02
CPT-20	11/17/2022	80.22	-	-	-
CPT-21	11/18/2022	80.38	-	-	-
CPT-22	11/17/2022	45.28	-	-	-
CPT-23	11/17/2022	71.03	-	-	-
CPT-24	11/17/2022	80.22	-	-	-
CPT-25	11/18/2022	80.22	-	-	27.23

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-16

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

Total depth: 100.23 ft, Date: 11/22/2022

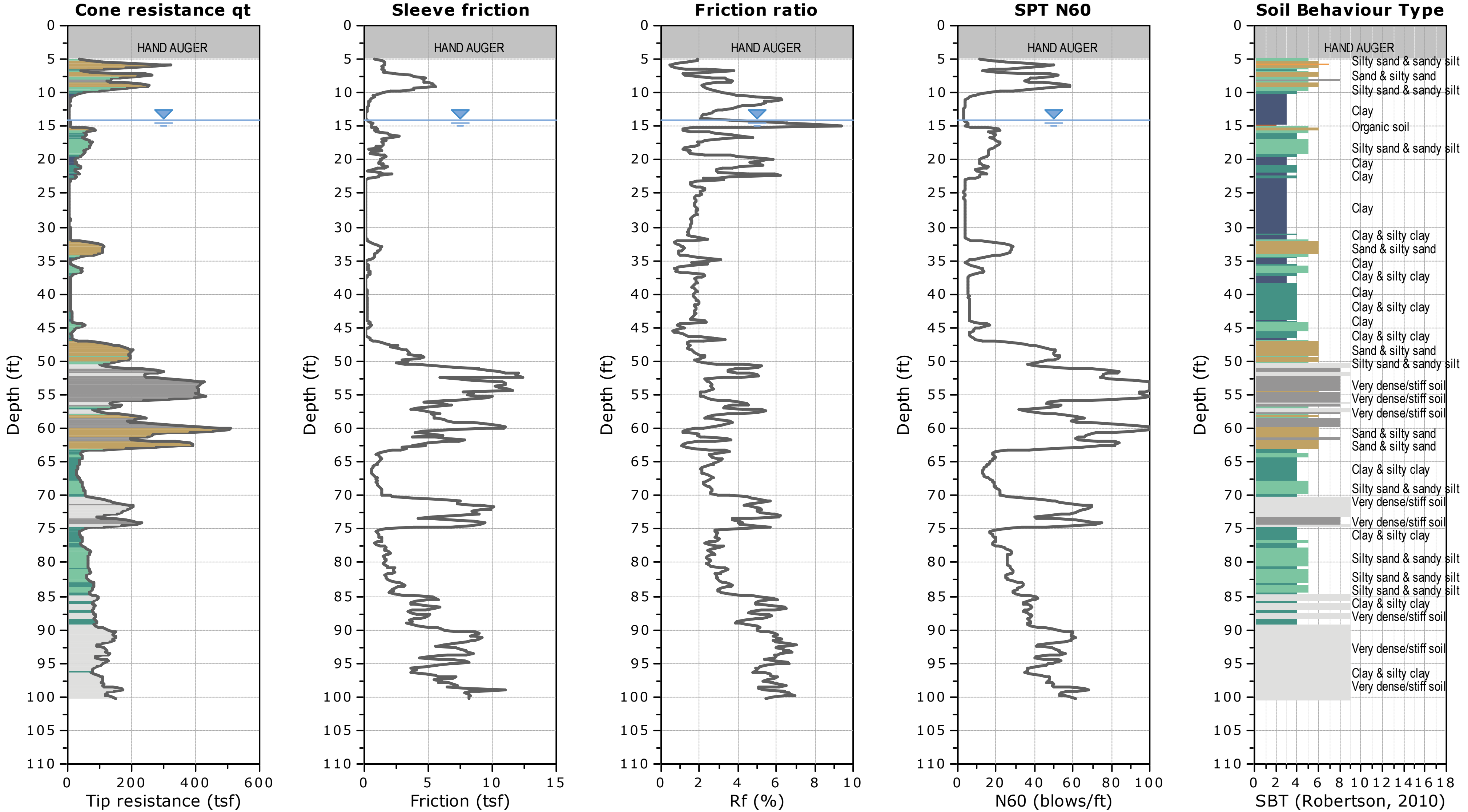


PLATE A-53

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-16

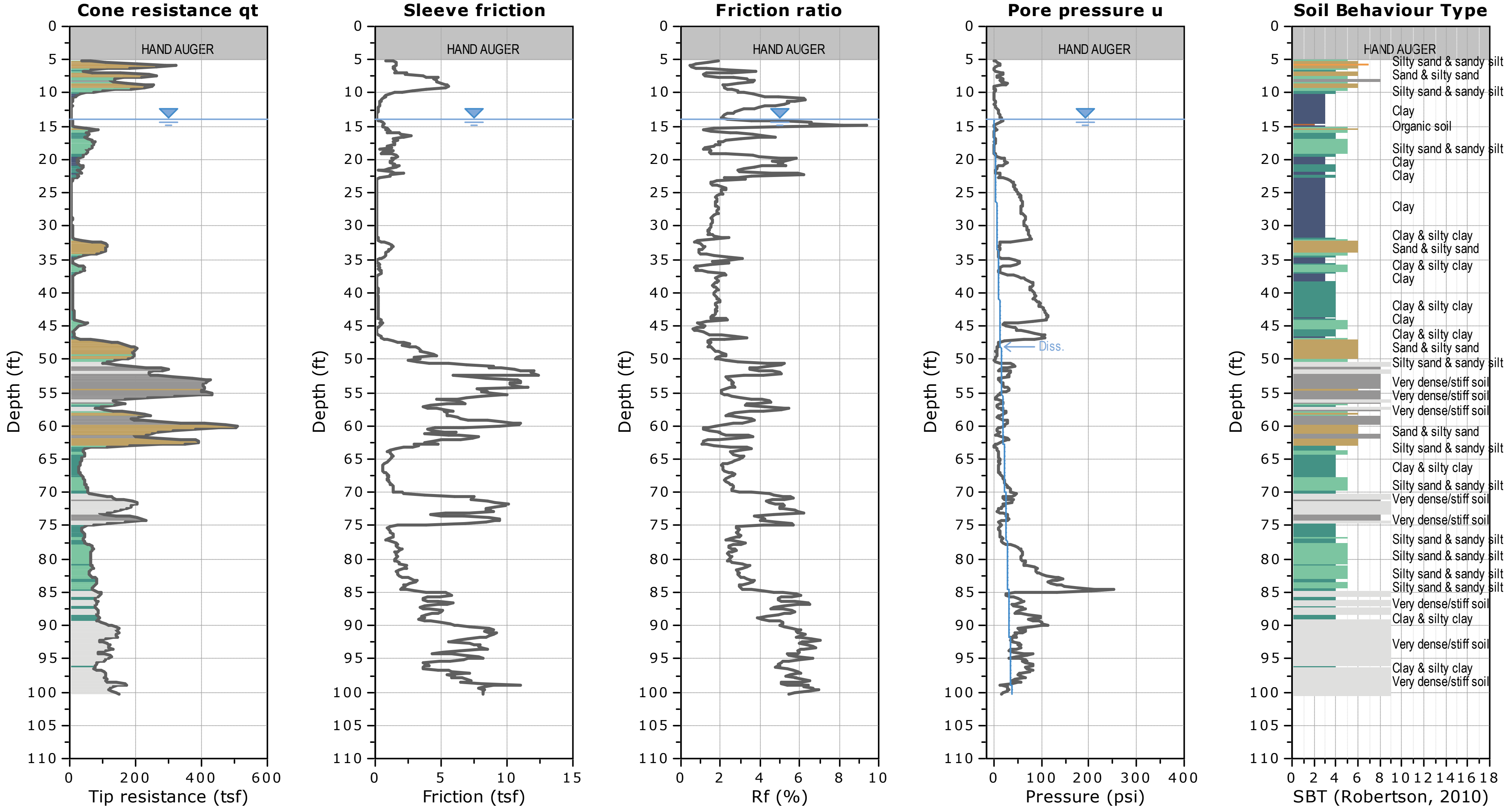
CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT

Cone ID: GDC-24

Total depth: 100.23 ft, Date: 11/22/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-54

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-17

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

Total depth: 100.23 ft, Date: 11/22/2022

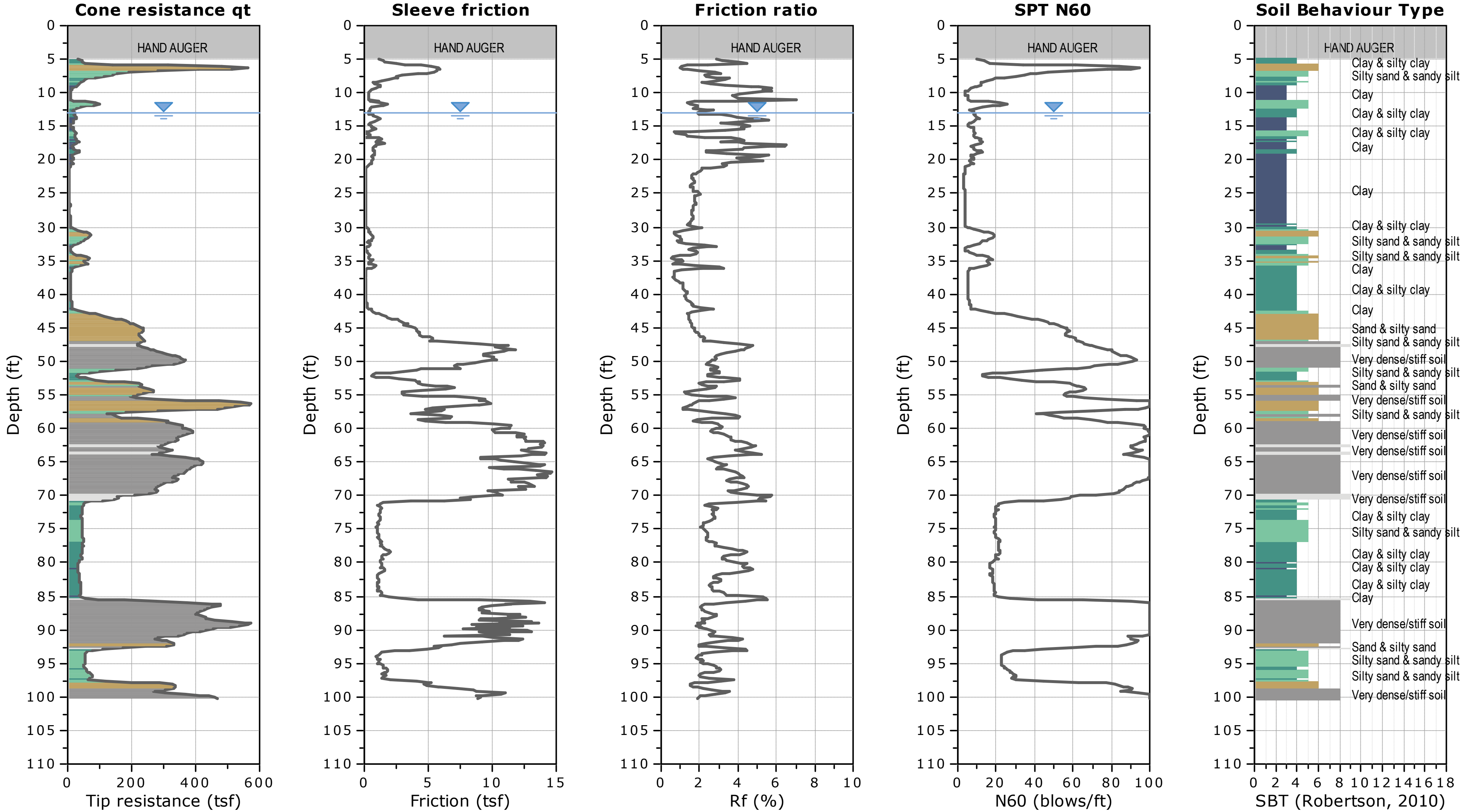


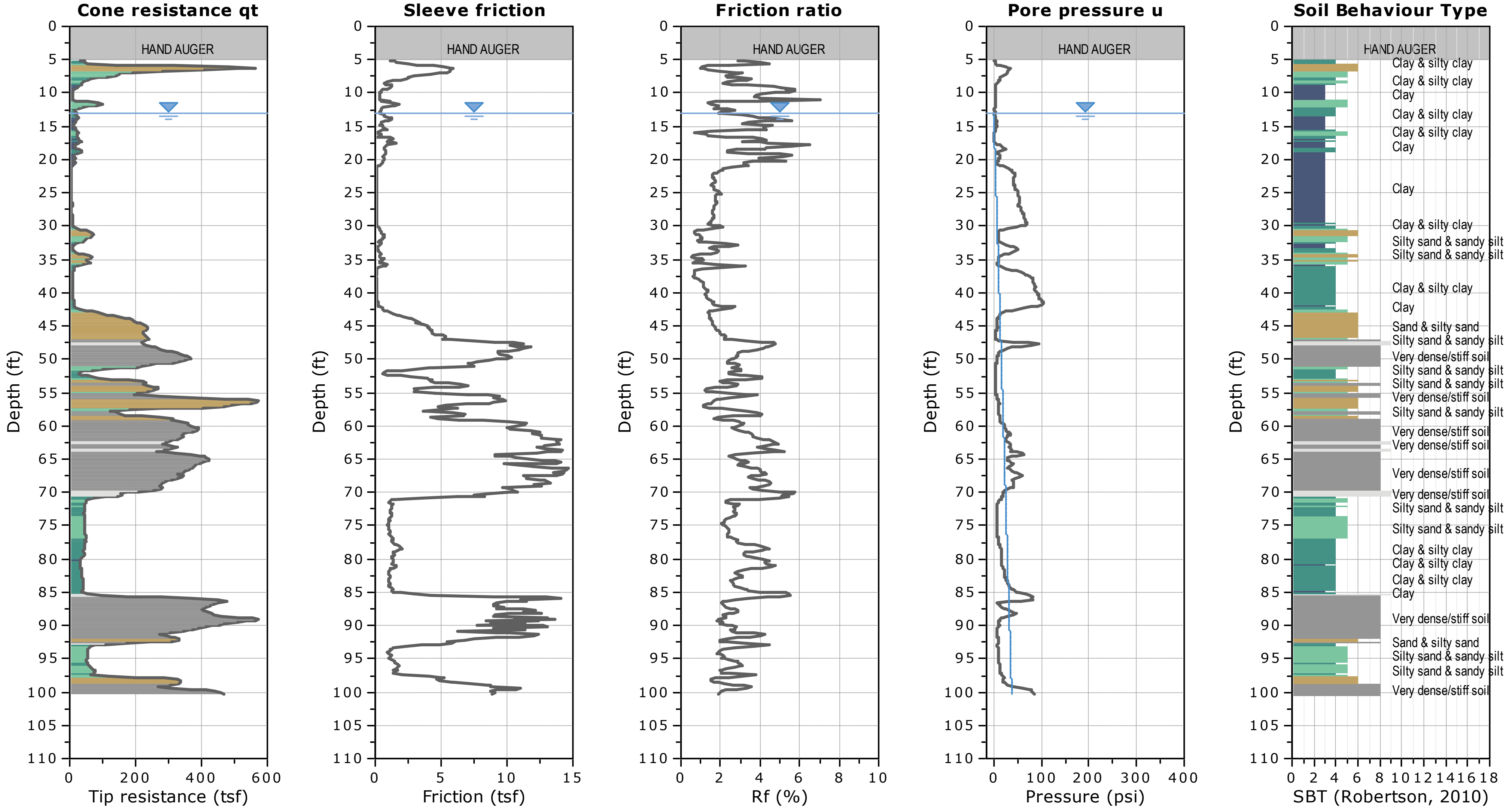
PLATE A-55

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-17

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 100.23 ft, Date: 11/22/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-56

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-18

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/18/2022

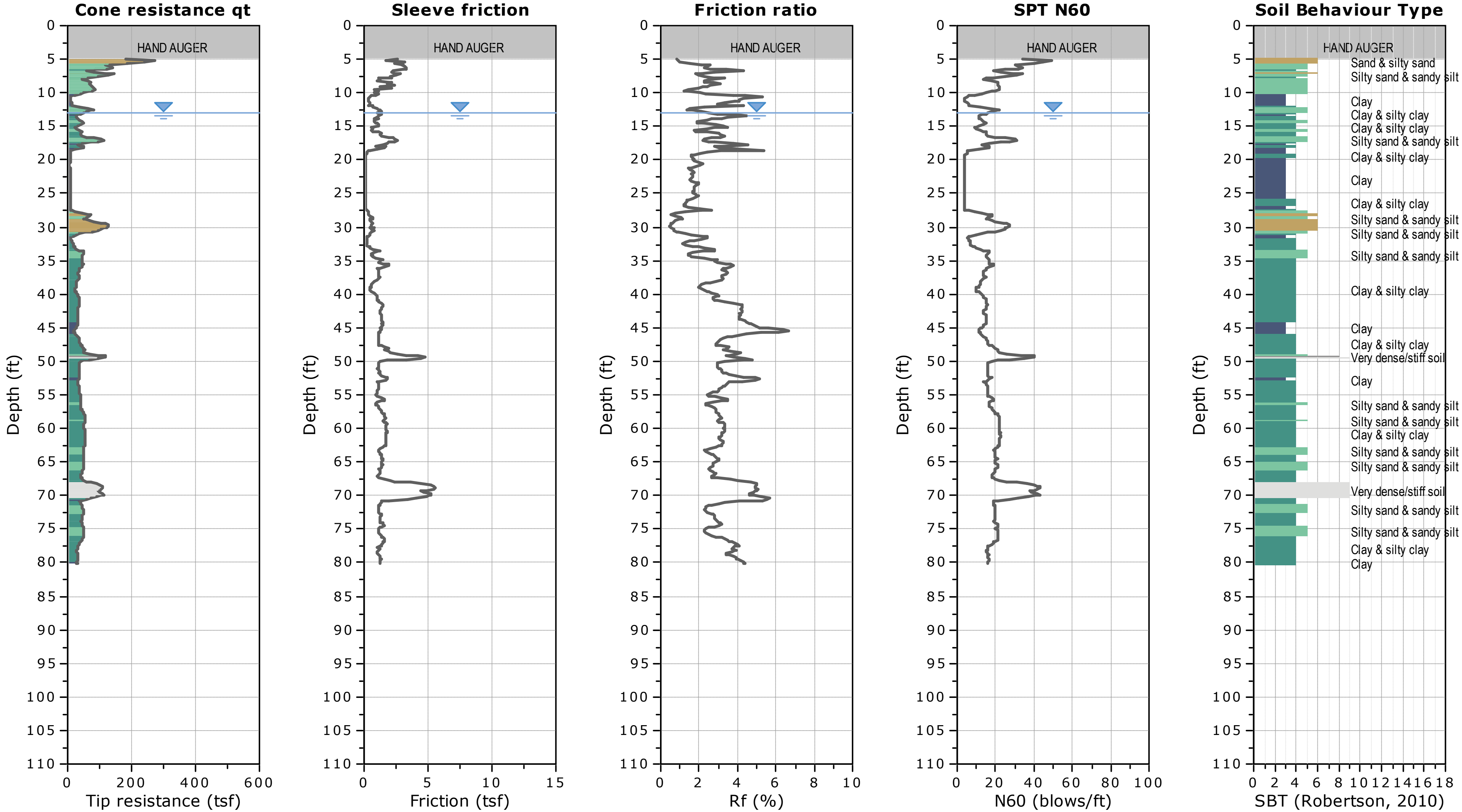


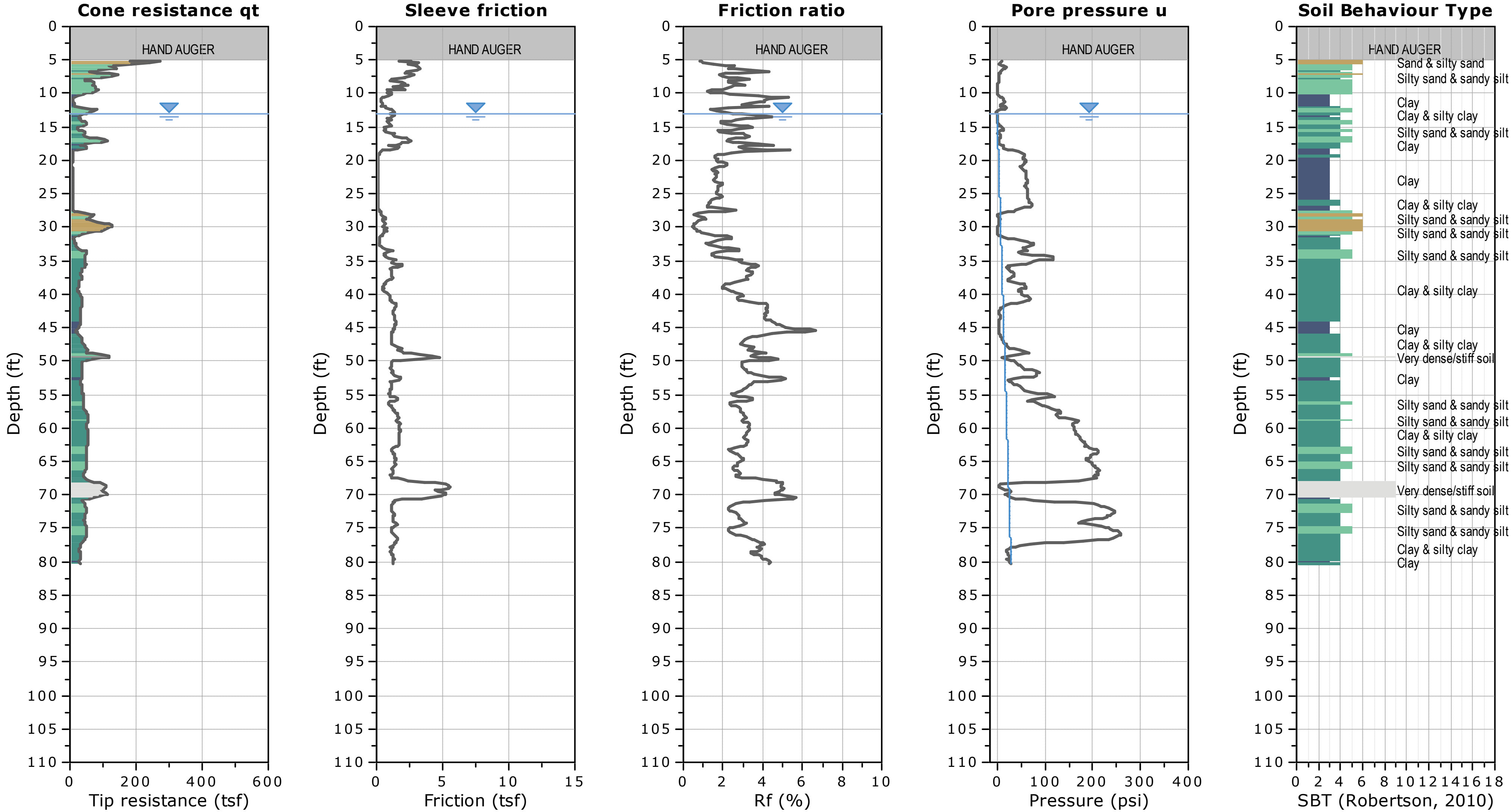
PLATE A-57

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-18

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/18/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-58

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-19

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 65.29 ft, Date: 11/17/2022

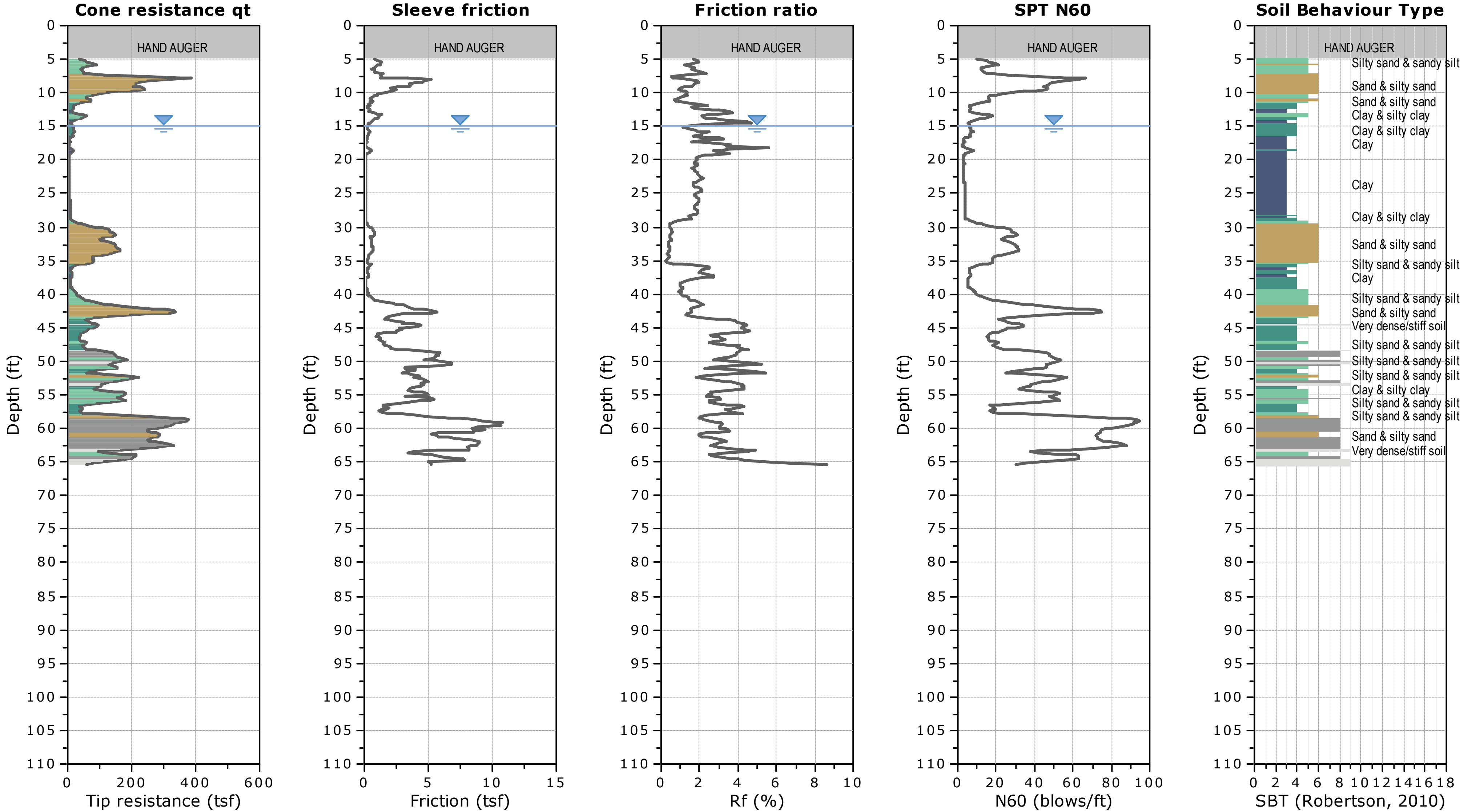


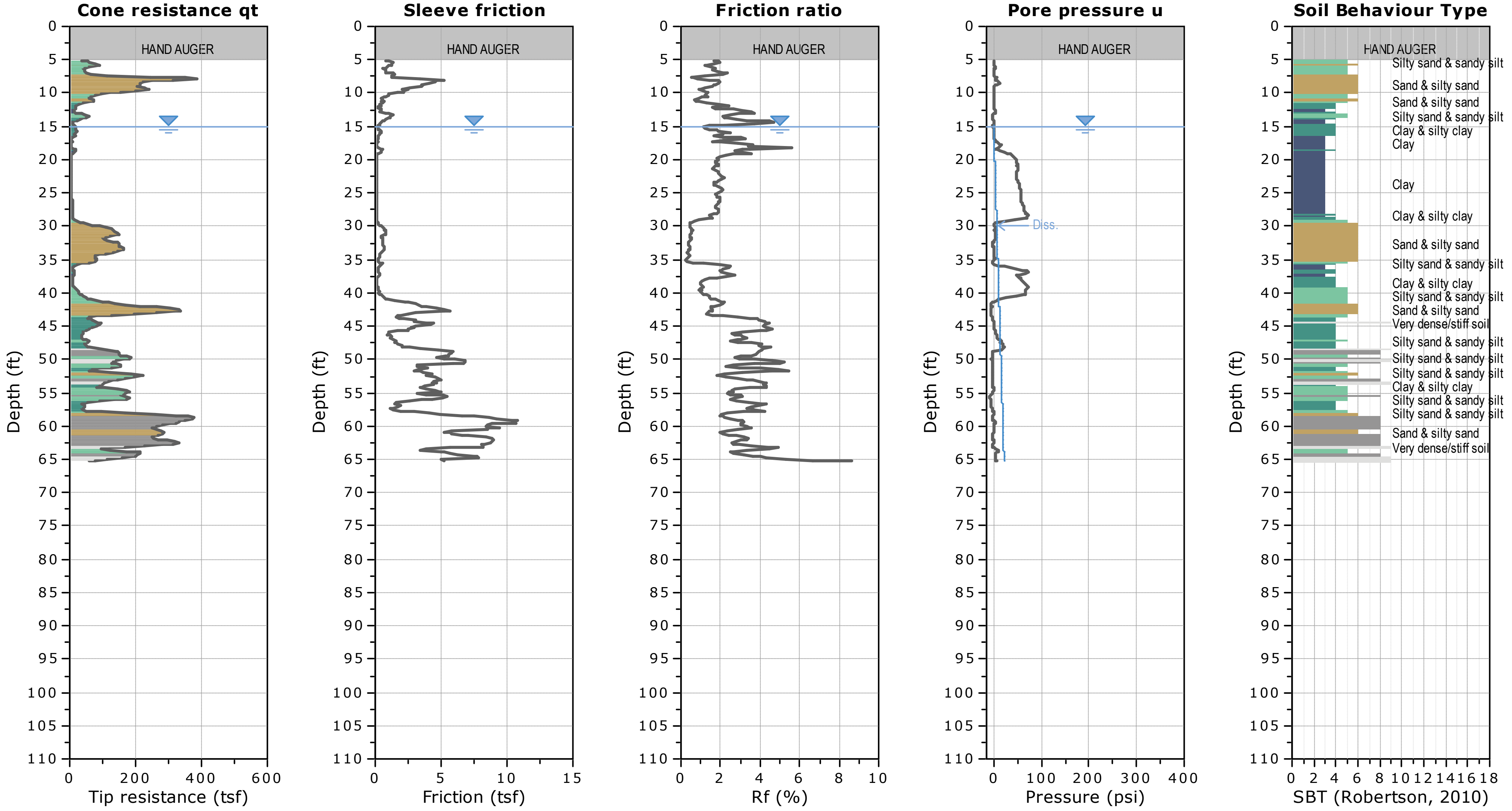
PLATE A-59

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-19

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 65.29 ft, Date: 11/17/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-60

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-20

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/17/2022

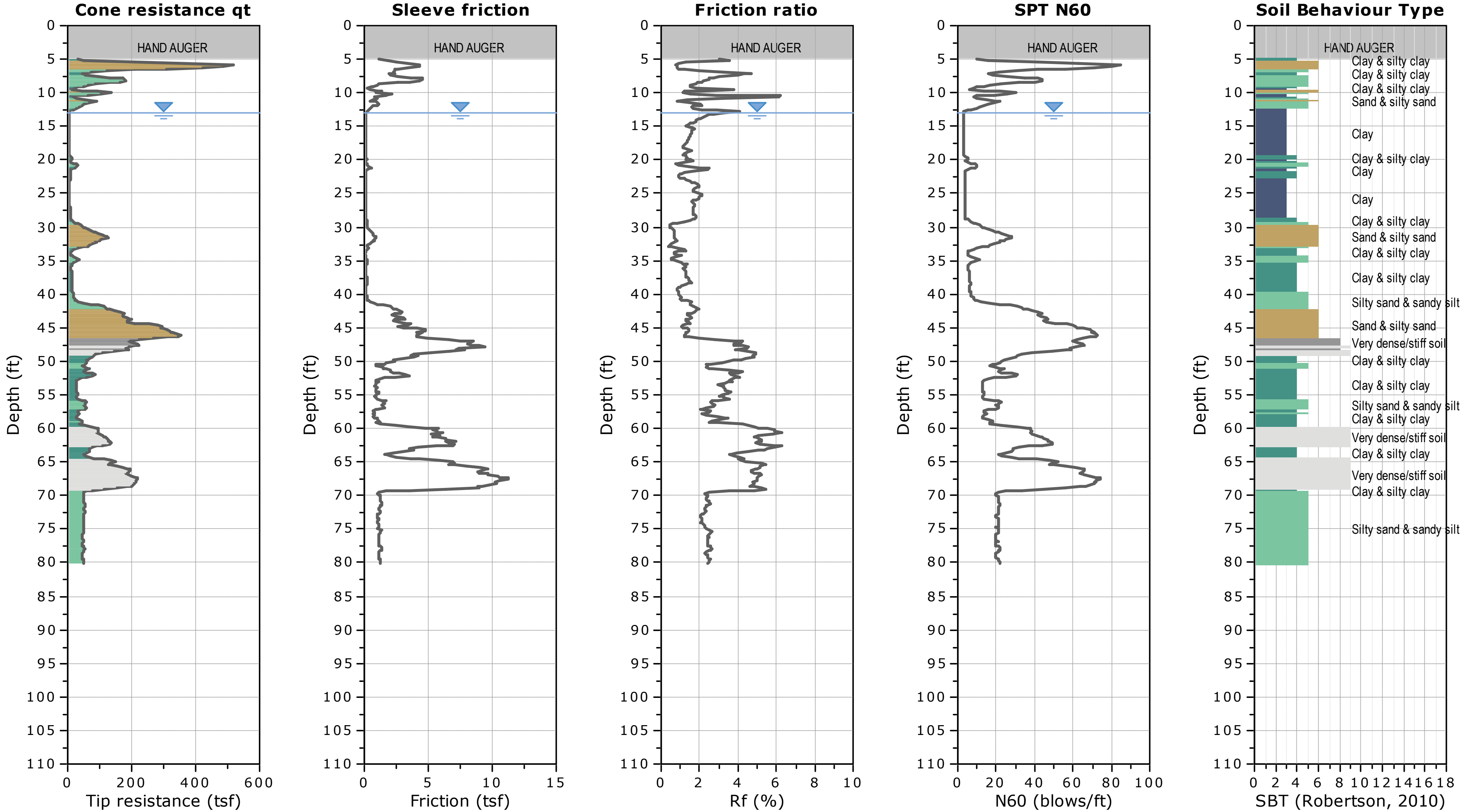


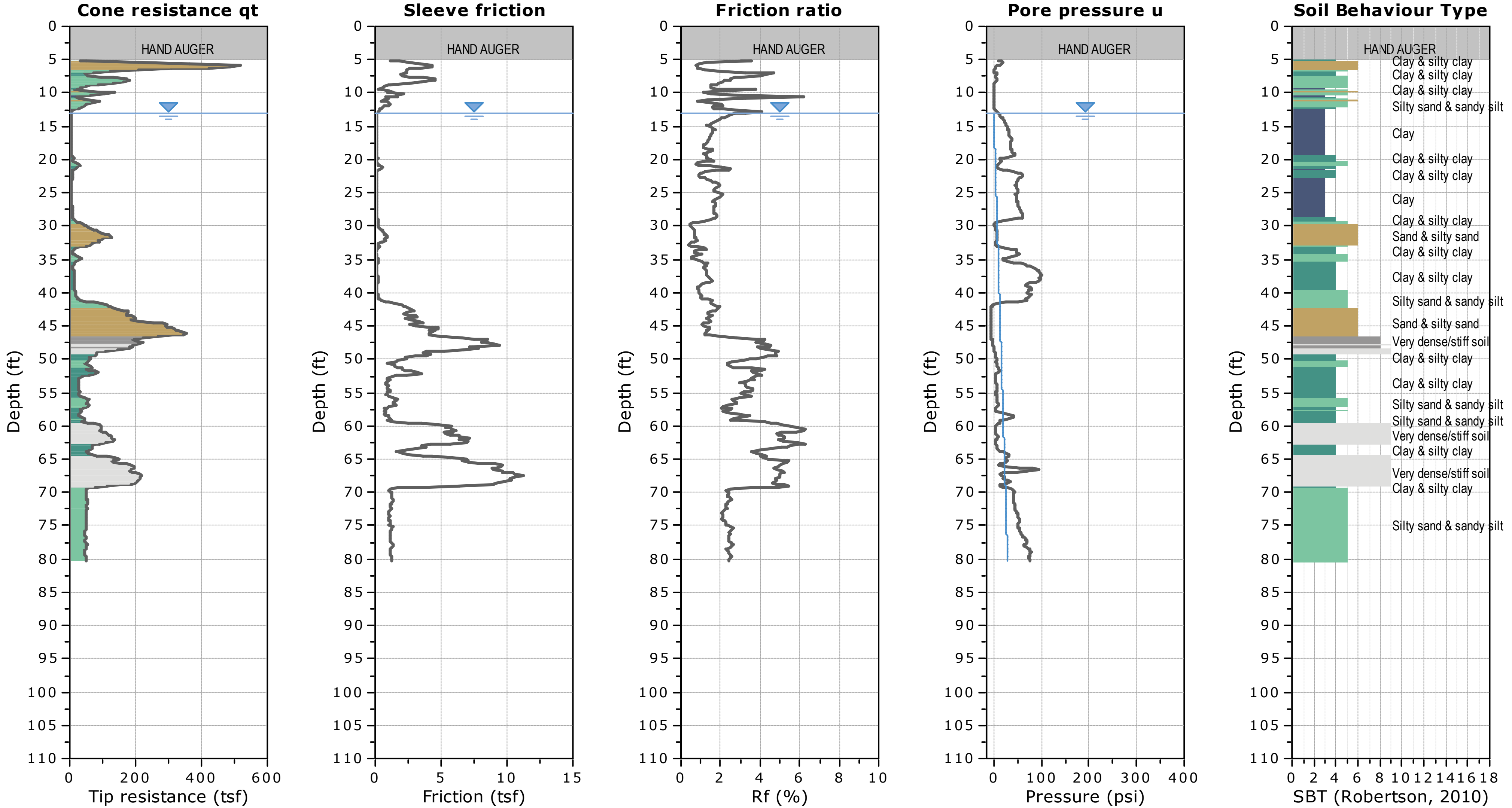
PLATE A-61

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-20

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/17/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-62

CPT: CPT-21

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.38 ft, Date: 11/18/2022

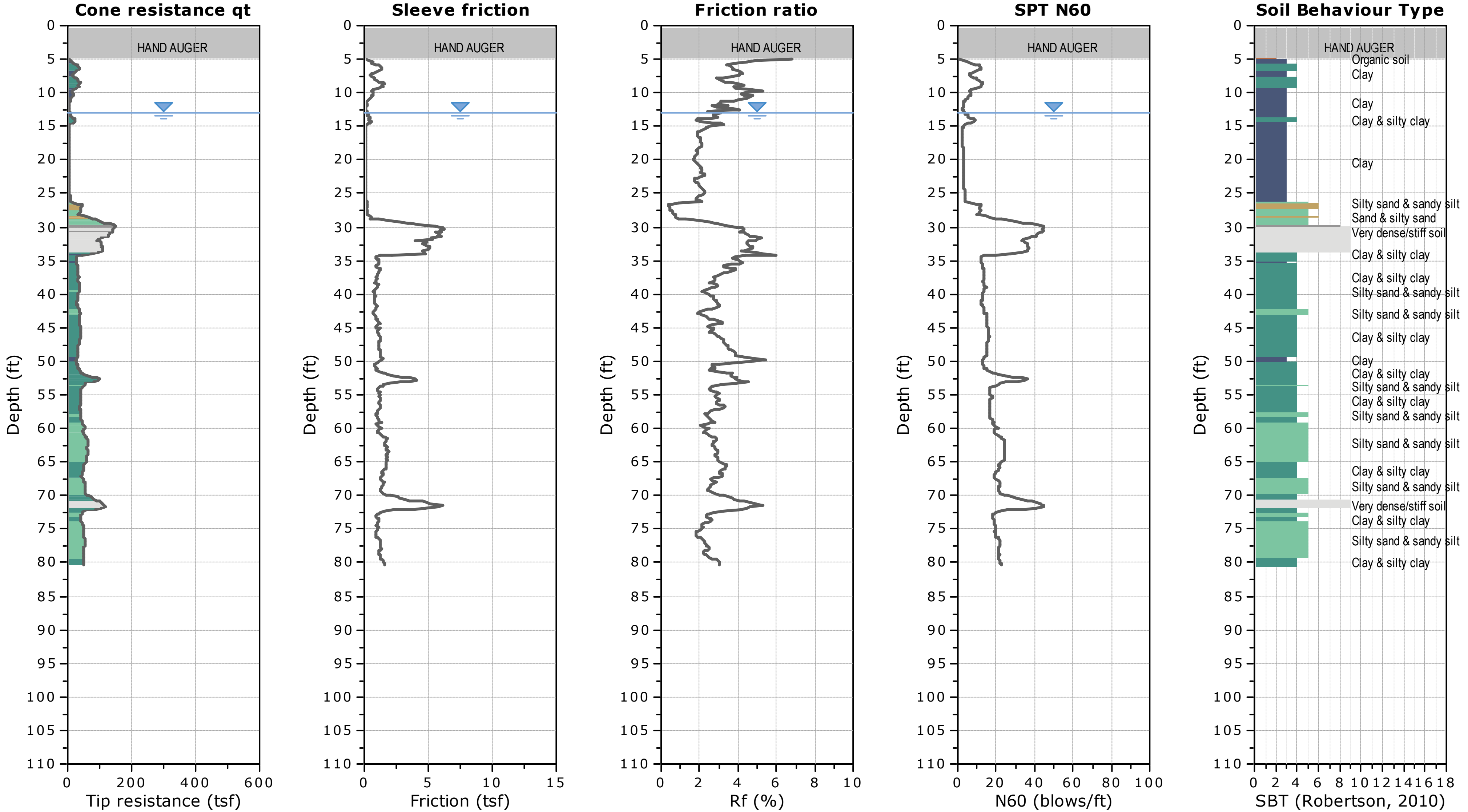


PLATE A-63

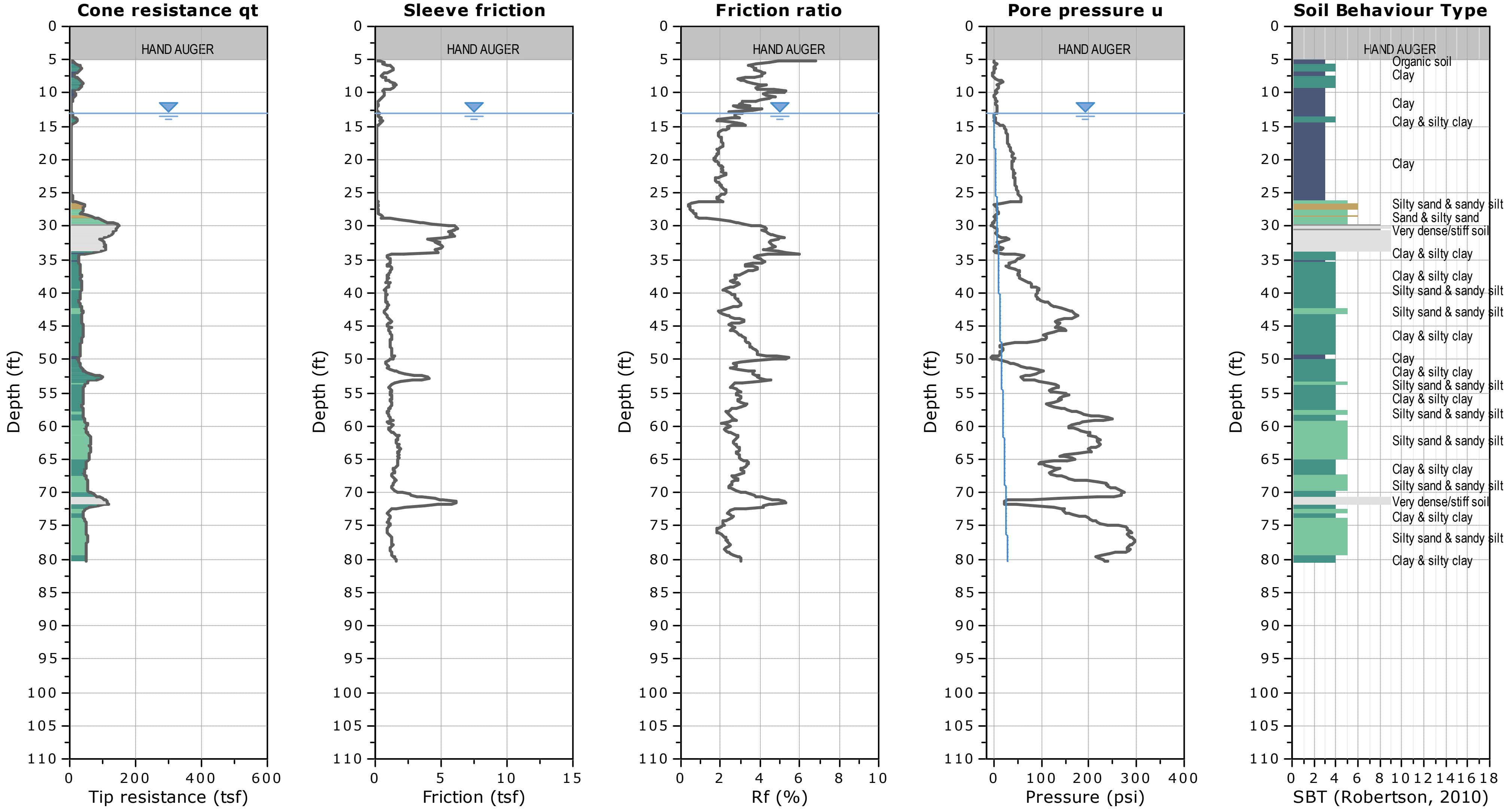
GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-21

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

Total depth: 80.38 ft, Date: 11/18/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-64

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-22

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 45.28 ft, Date: 11/17/2022

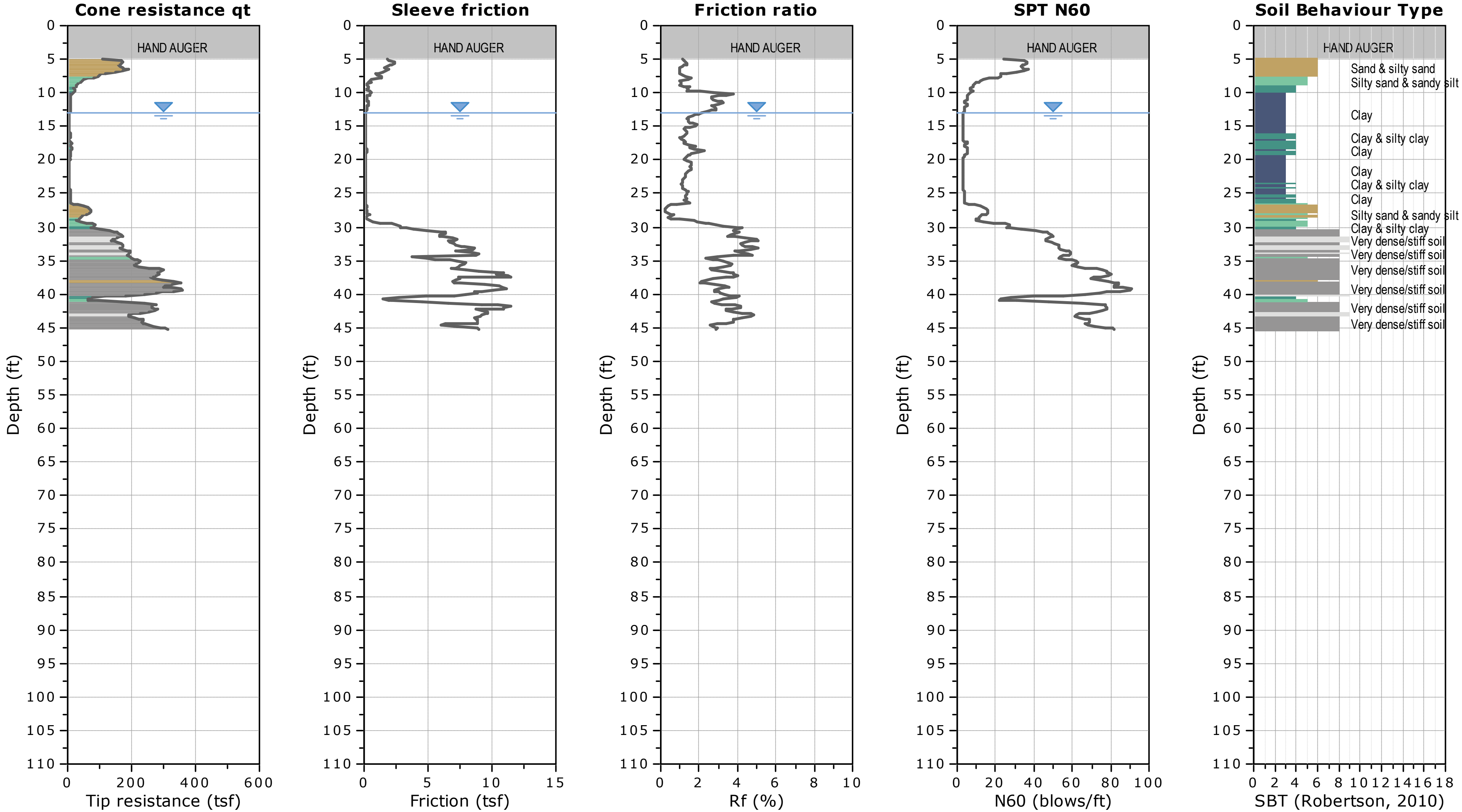


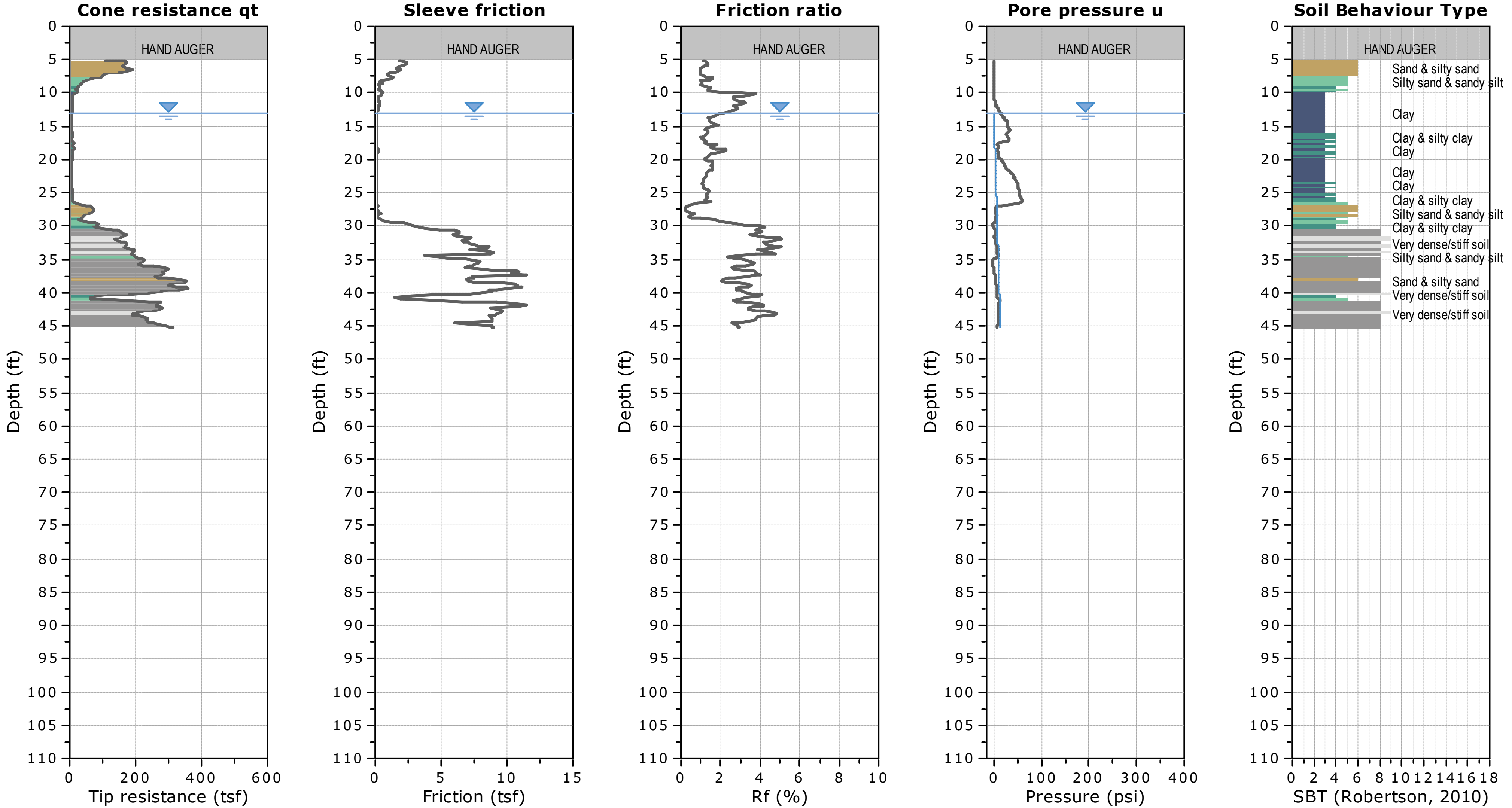
PLATE A-65

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-22

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 45.28 ft, Date: 11/17/2022

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-66

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-23

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 71.03 ft, Date: 11/17/2022

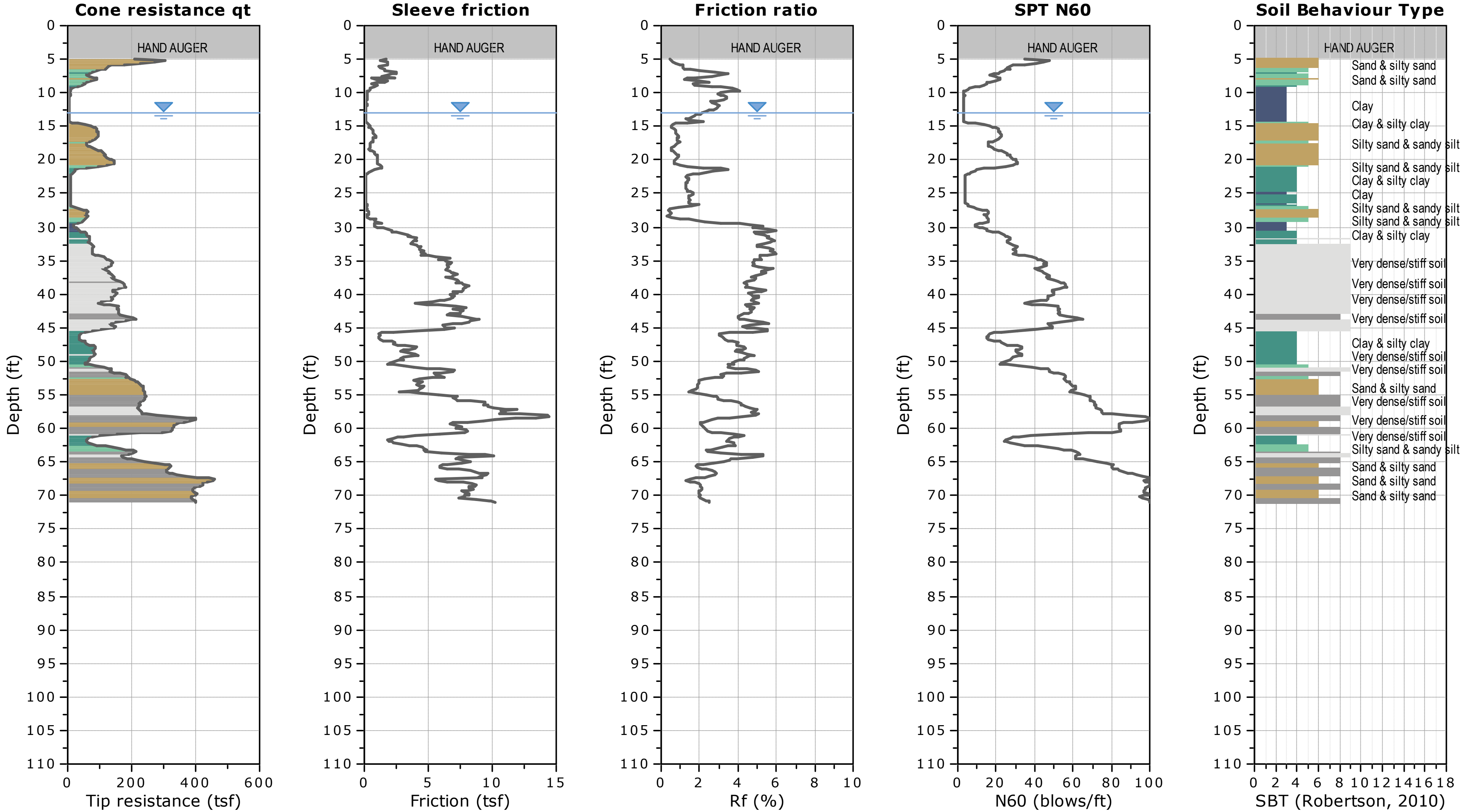


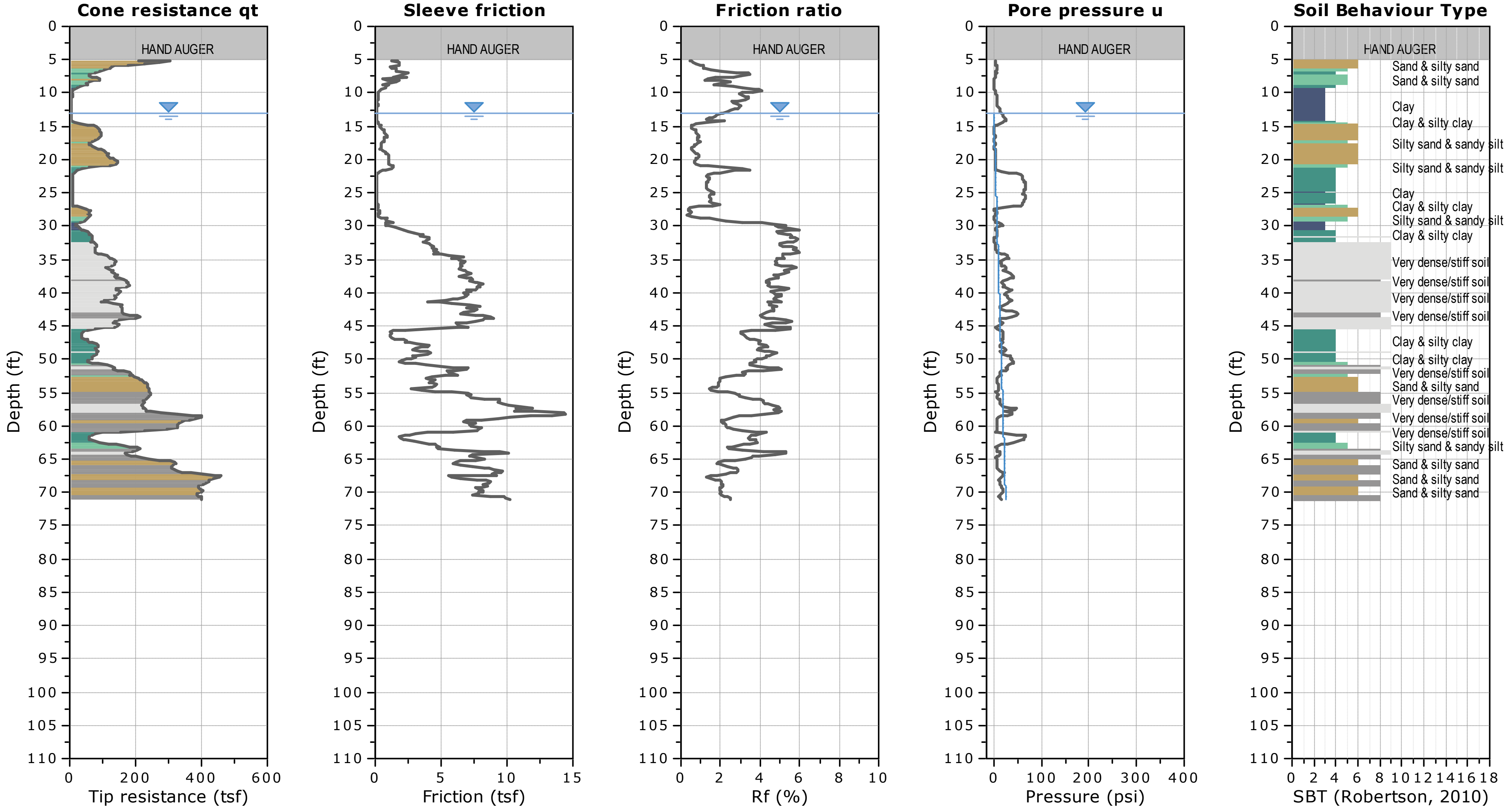
PLATE A-67

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-23

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 71.03 ft, Date: 11/17/2022

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-68

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-24

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/17/2022

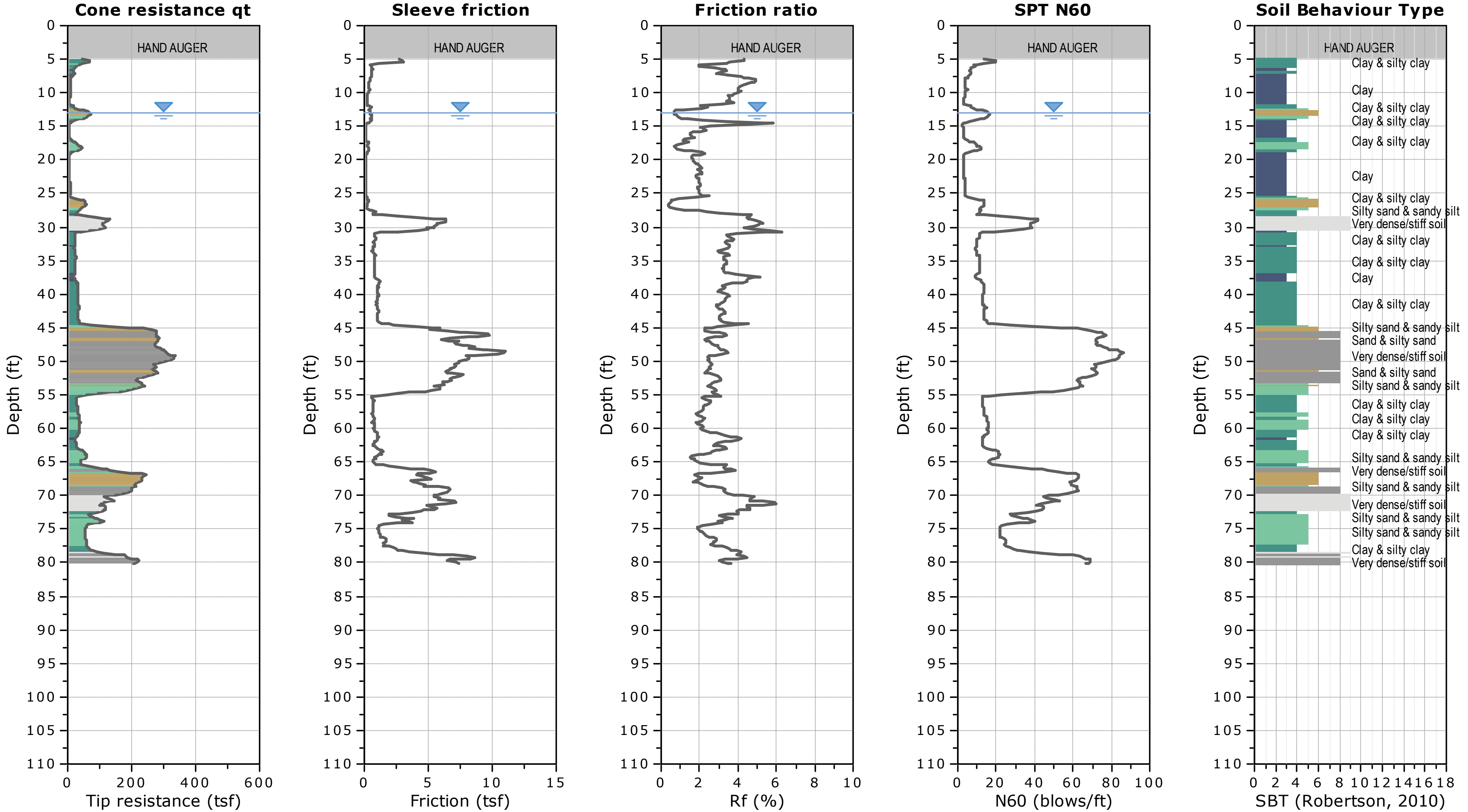


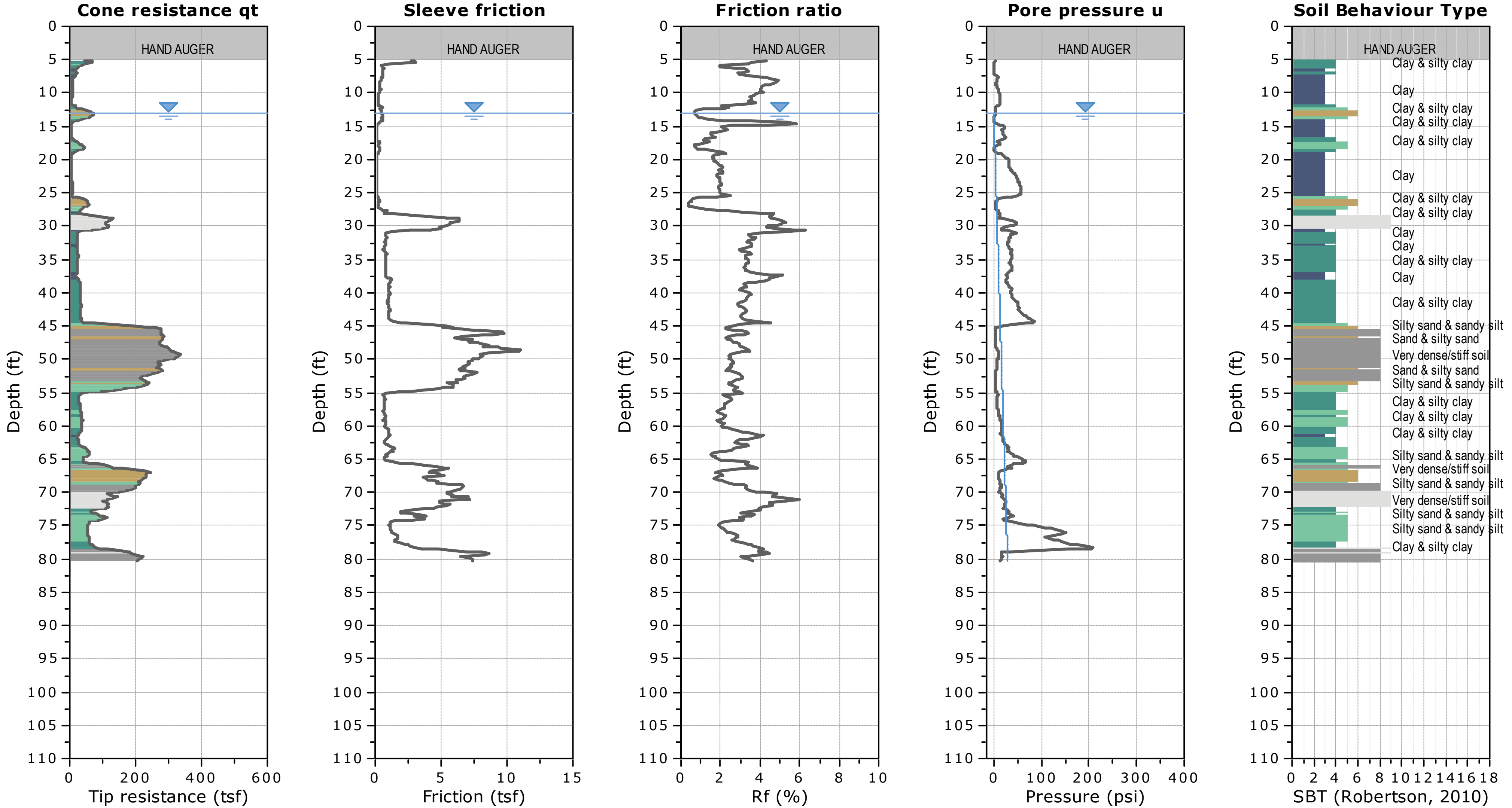
PLATE A-69

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-24

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/17/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-70

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-25

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

Total depth: 80.22 ft, Date: 11/18/2022

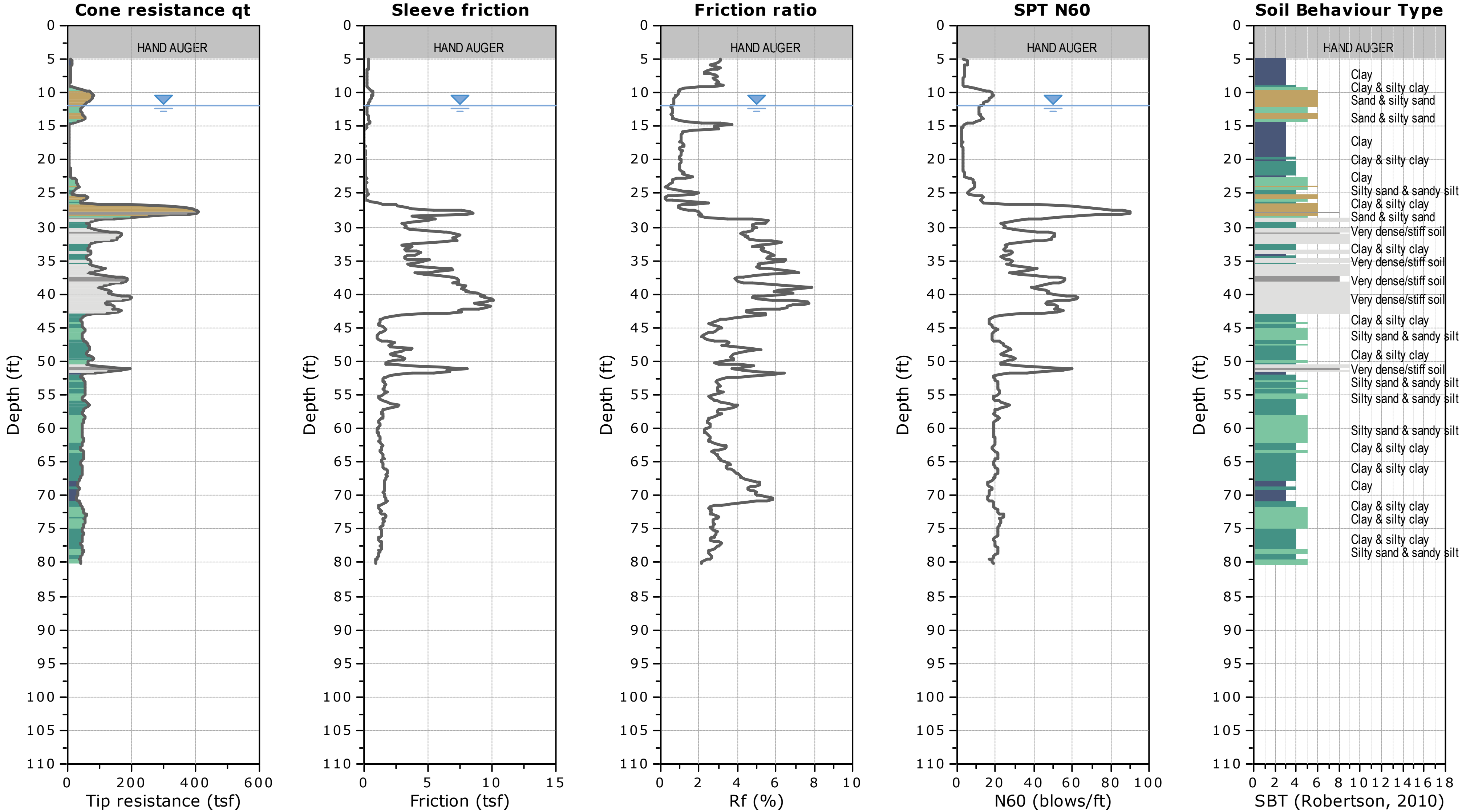


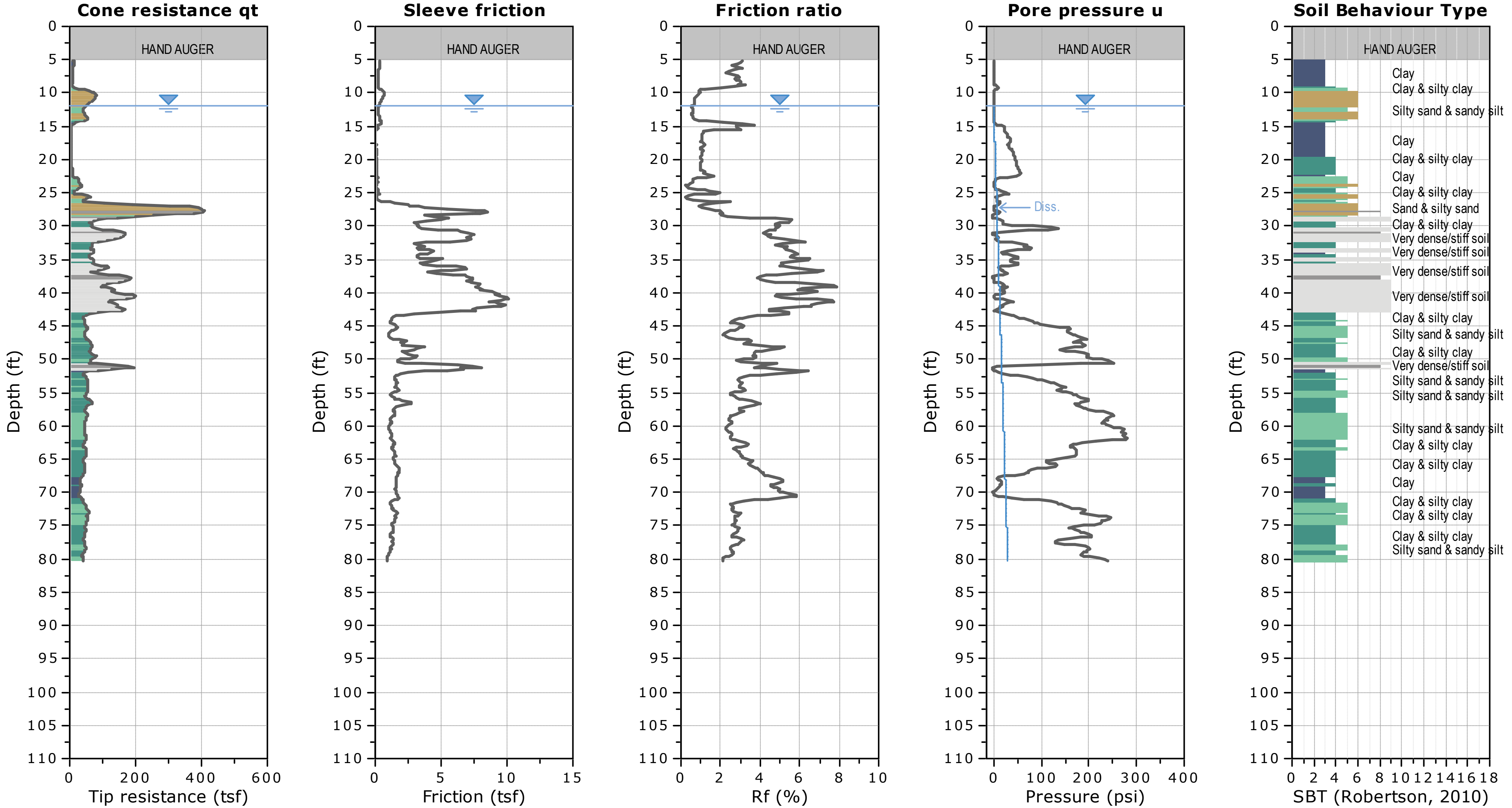
PLATE A-71

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-25

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/18/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-72

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-16
Depth (ft): 48.06
Site: Laney College
Engineer: Abdul Sadat

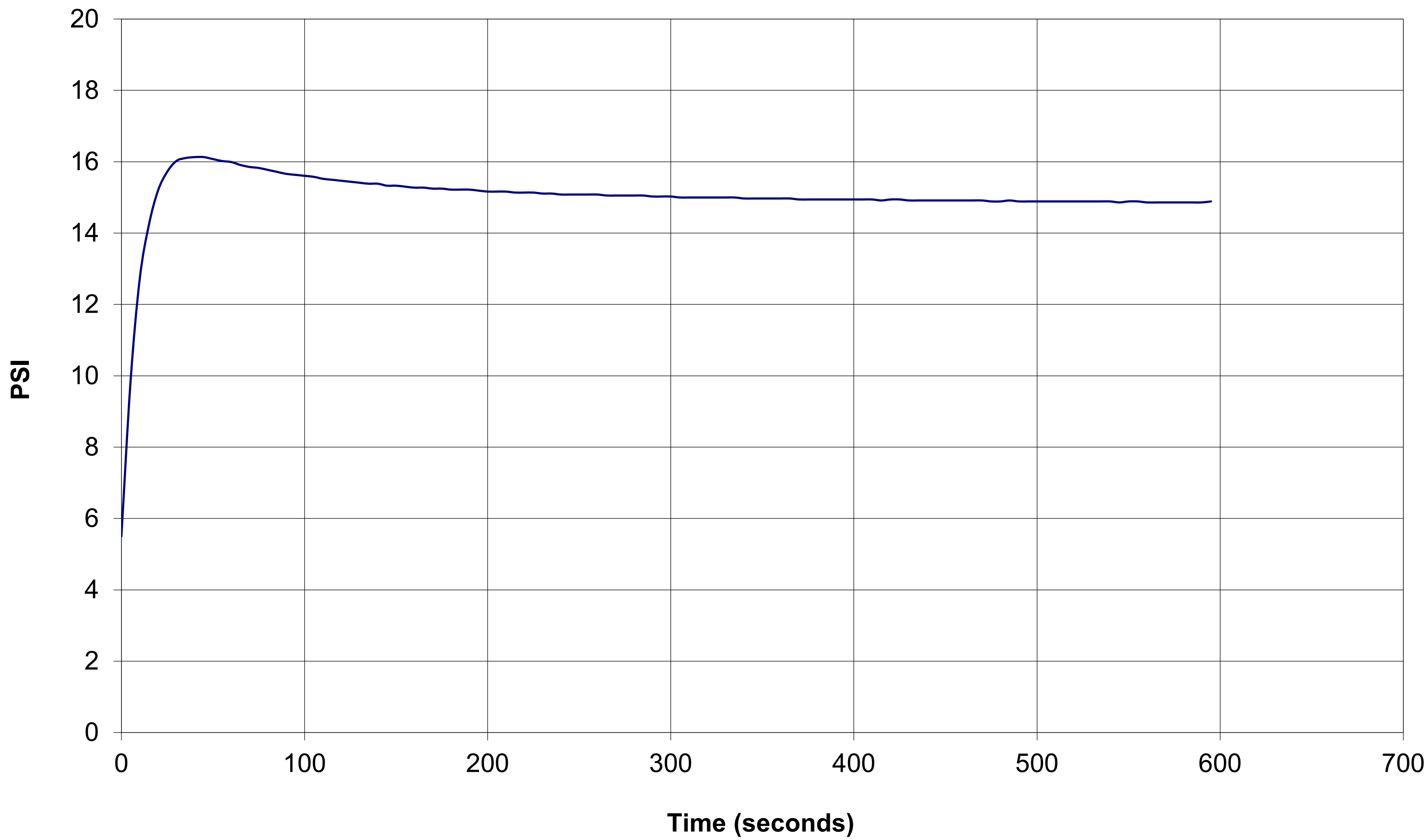


PLATE A-73

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-19
Depth (ft): 30.02
Site: Laney College
Engineer: Abdul Sadat

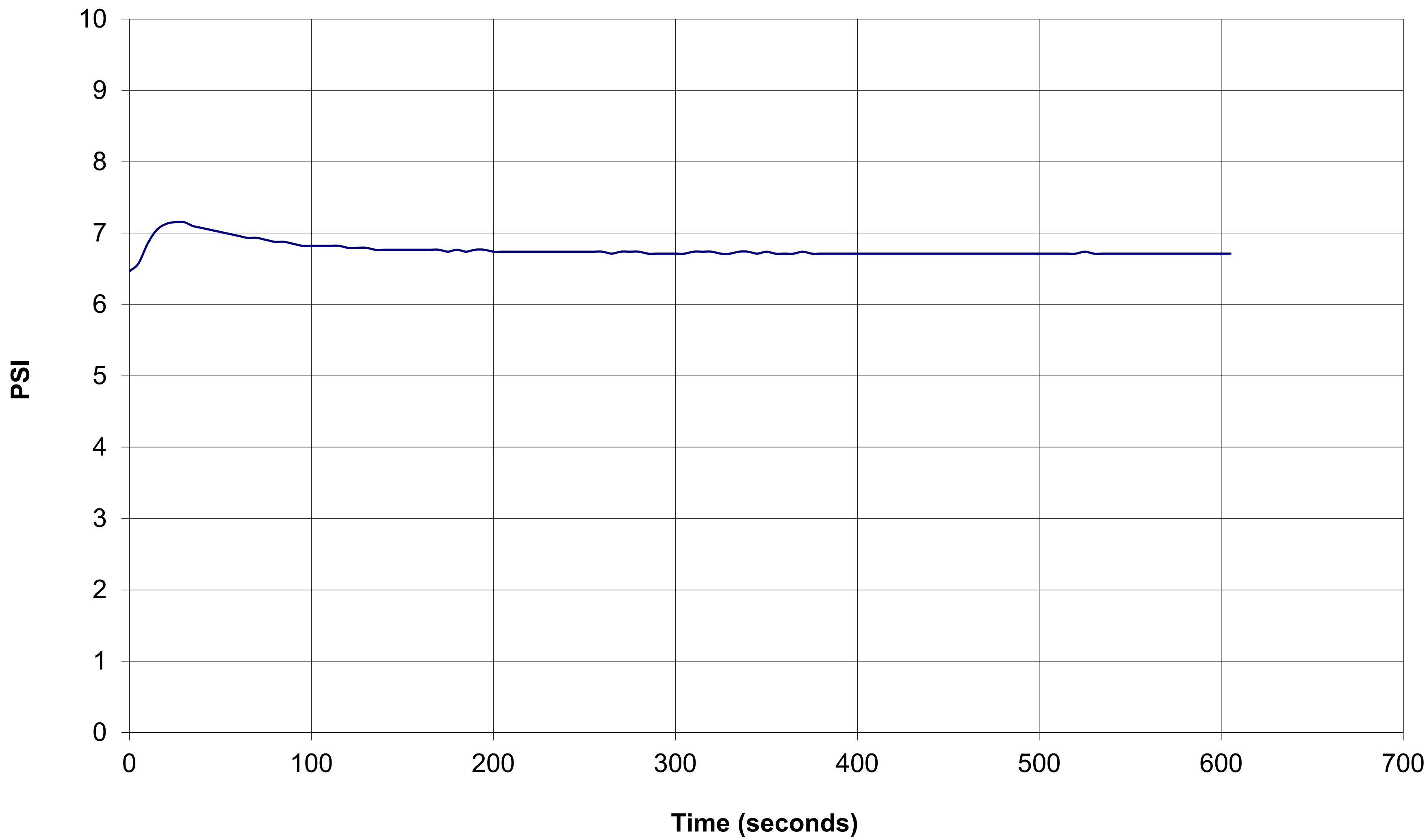


PLATE A-74

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-25
Depth (ft): 27.23
Site: Laney College
Engineer: Abdul Sadat

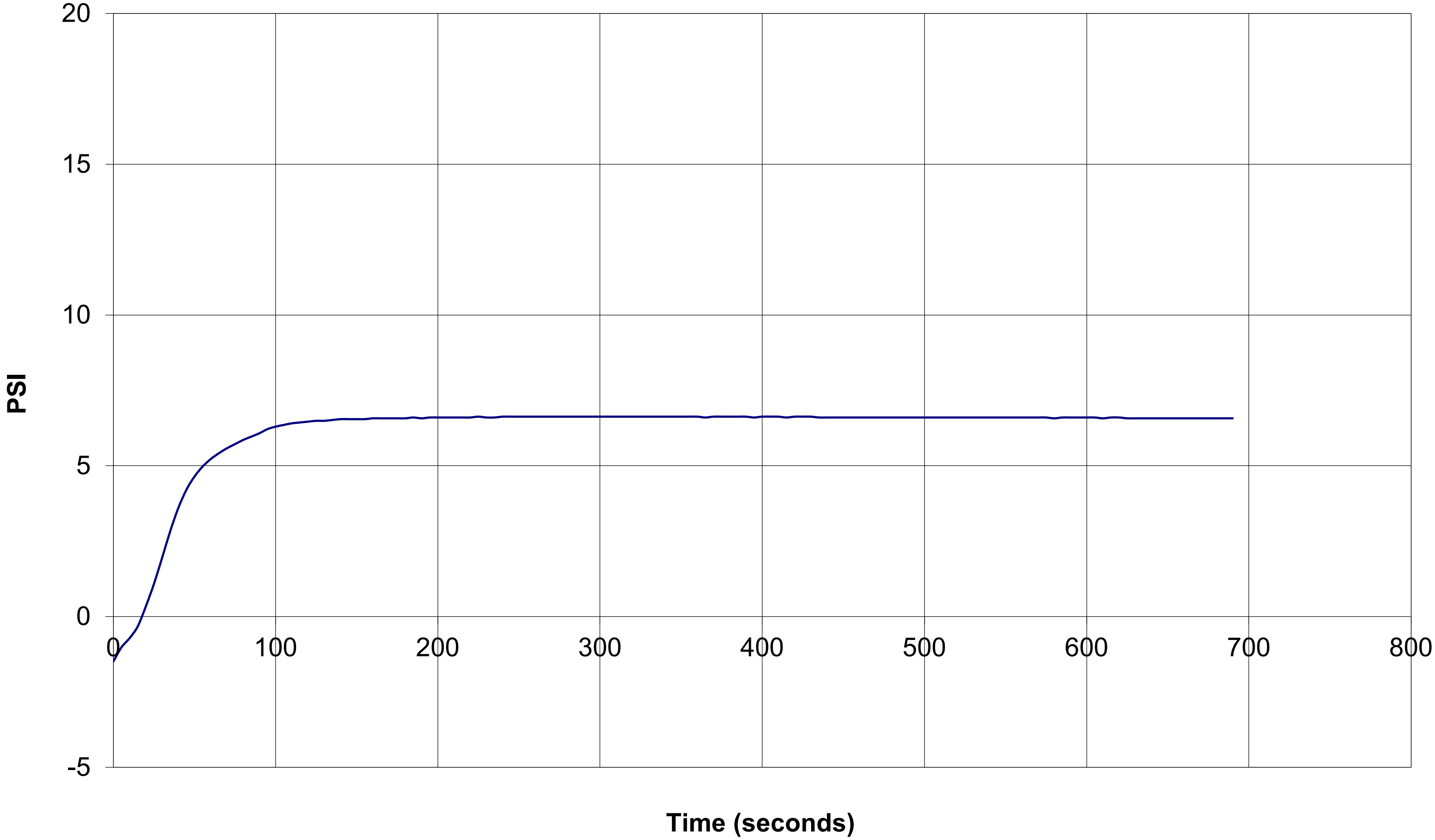


PLATE A-75

Appendix B

Laboratory Testing Program

Peralta Community College District
Project No. 04.72190021

FUGRO

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC%	FINES %	ATTERBURG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	ORGANIC CONTENT (%)	TEST LISTING
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	SO_4				
2019-CPT-01	2.5	S1	SILTY SAND (SM)													6400	7.59	N.D.	22				Co
2019-CPT-02	4.5	S2	PEAT (PT)			55																	M
2019-CPT-02	5.5	S3	Fat CLAY (CH)			58																	M
2019-CPT-03	4.0	S1	CLAYEY SAND (SC)			13	20									2600	7.97	N.D.	16				M, FC, Co
2020-B-01	11.0	S5	SILTY SAND with GRAVEL (SM)	112	91	24	21																T, M, S
2020-B-01	16.0	S7	Poorly-graded SAND with SILT (SP-SM)	119	95	26	6															5	T, M, O, S
2020-B-01	17.0	S8	ORGANIC CLAY with SAND (OH)			82																21.2	M, O
2020-B-01	21.0	S9	Fat CLAY (CH)			53																6.6	M, O
2020-B-01	27.0	S10	Fat CLAY (CH)	95	52	83																	T, M, C
2020-B-01	30.0	S11	Fat CLAY (CH)	109	69	58	93	73	43						0.48(2.2)								T, M, A, S, Q
2020-B-01	31.0	S11	Poorly-graded SAND with SILT (SP-SM)	120	94	27	8																T, M, S
2020-B-01	40.5	S13	Fat CLAY (CH)	101	59	71									0.73(2.6)								T, M, Q
2020-B-01	51.0	S15	SILTY SAND (SM)	132	112	18	16																T, M, S
2020-B-01	55.0	S16	SILTY SAND (SM)	135	116	17	17																T, M, S
2020-B-01	66.0	S17	SILTY SAND (SM)			19	19																M, FC
2020-B-01	76.0	S18	Lean CLAY (CL)			37																	M

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

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

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

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

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

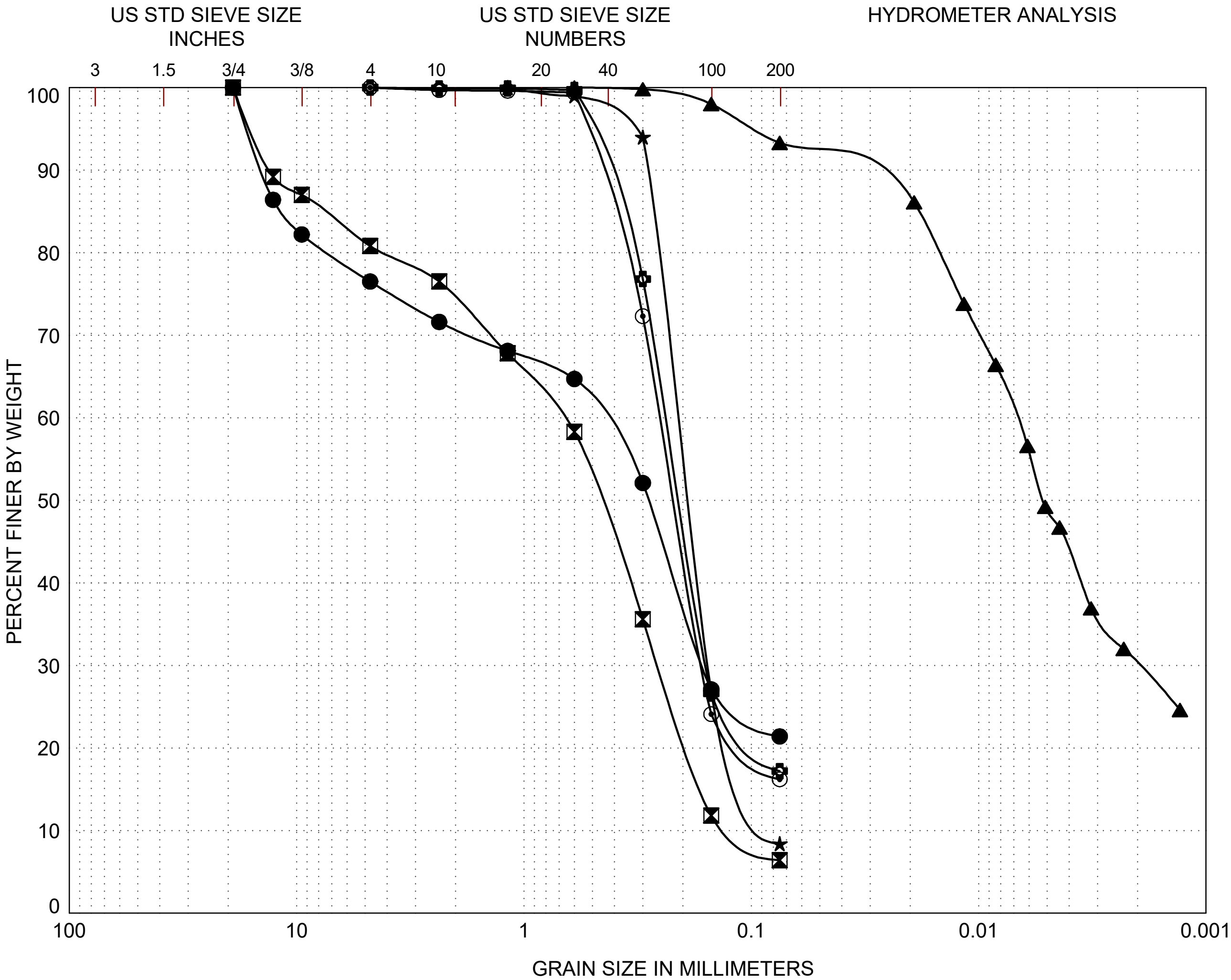
Test Listing Abbreviations
M = Moisture Content
T = Total & Dry Unit Weight
S = Sieve Analysis
FC = % Passing #200 Sieve
H = Hydrometer Analysis
A = Atterberg Limits
P = Compaction Test
D = Direct Shear Test
C = Consolidation Test
Co = Corrosivity Tests
CU = CU Triaxial
U = UU Triaxial
R = R-Value
SE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

SUMMARY OF LABORATORY TEST RESULTS

Laney College Library Learning Resource Center
Oakland, California

PLATE B-1

SUM-1 LAB_SUMMARY (W:\PROJECTS\LOCATION-72\2019\04.72190021 LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER\06_FIELD_AND_LAB\06_BORINGS\01 GINT\LANEY COLLEGE - 2020 BORING.GPJ) -VTA- 2/5/2

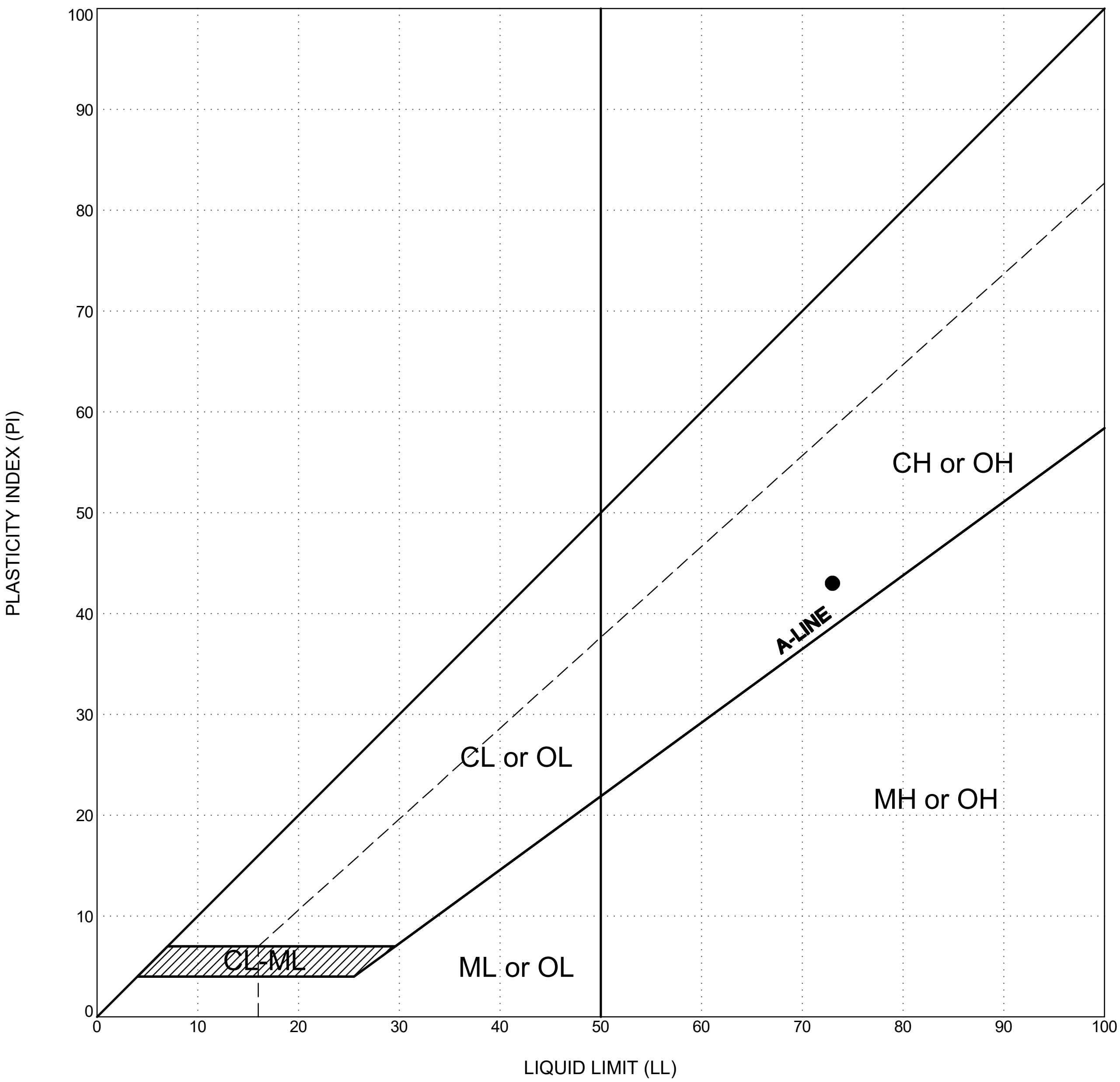


GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LEGEND (location)	(depth,ft)	CLASSIFICATION	Cc	Cu	D10	D30	D60
● 2020-B-01	11.0	SILTY SAND with GRAVEL (SM)				0.16	0.46
⊠ 2020-B-01	16.0	Poorly-graded SAND with SILT (SP-SM)	0.8	5.7	0.12	0.25	0.68
▲ 2020-B-01	30.0	Fat CLAY (CH)				0.00	0.01
★ 2020-B-01	31.0	Poorly-graded SAND with SILT (SP-SM)	1.4	2.7	0.08	0.16	0.21
⊙ 2020-B-01	51.0	SILTY SAND (SM)				0.16	0.25
✪ 2020-B-01	55.0	SILTY SAND (SM)				0.16	0.24

GRAIN SIZE CURVES
Laney College Library Learning Resource Center
Oakland, California

PLATE B-2



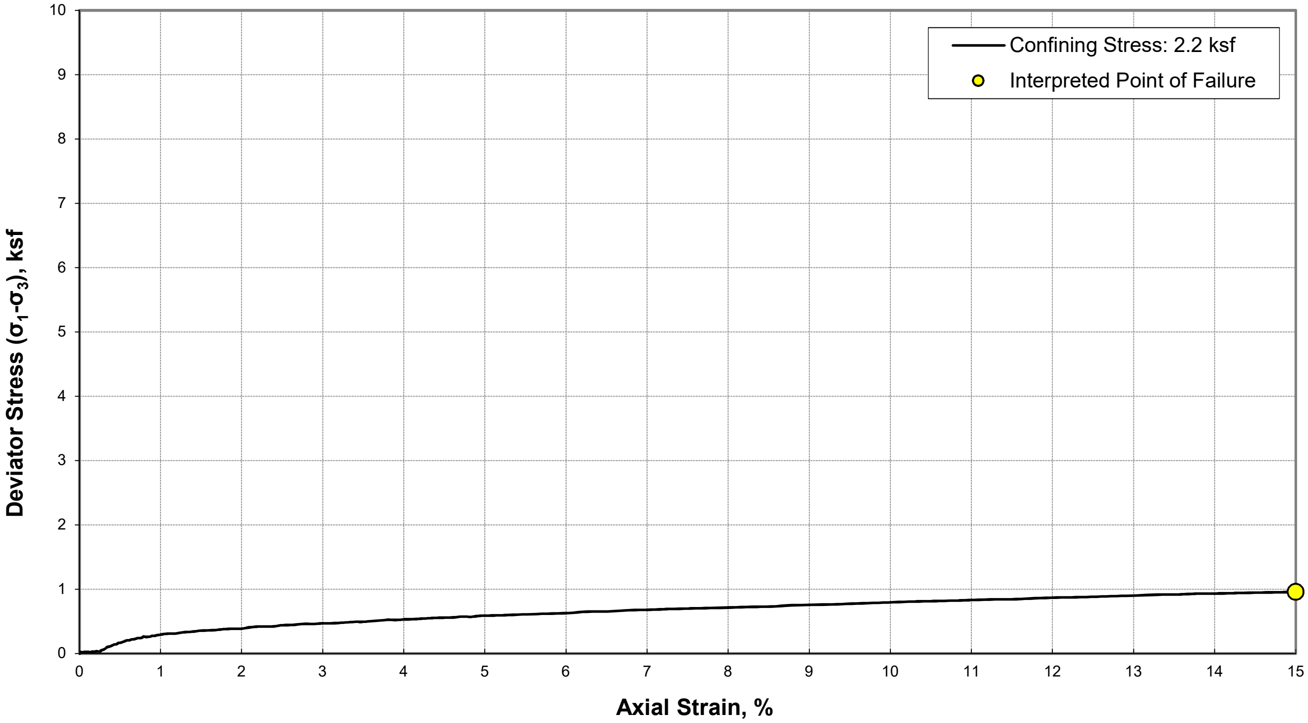
ATTERBERG LIMITS TEST RESULTS

LEGEND			CLASSIFICATION	LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
	location	depth, ft				
●	2020-B-01	30.0	Fat CLAY (CH)	73	30	43

PLASTICITY CHART

Laney College Library Learning Resource Center
Oakland, California

PLATE B-3



SAMPLE ID

Boring Number:	B-01
Sample Number:	S11
Sample Depth:	30.0 ft
USCS Classification:	Fat CLAY (CH): olive gray

SAMPLE PROPERTIES

Water Content, %	58.3%
Dry Unit Weight, pcf	68.7
Diameter, in	2.39
Height, in	5.60

SAMPLE IMAGES


CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
#4 (4.75mm)	---	Plastic Limit	---
#16 (1.18mm)	---	Plasticity Index	---
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

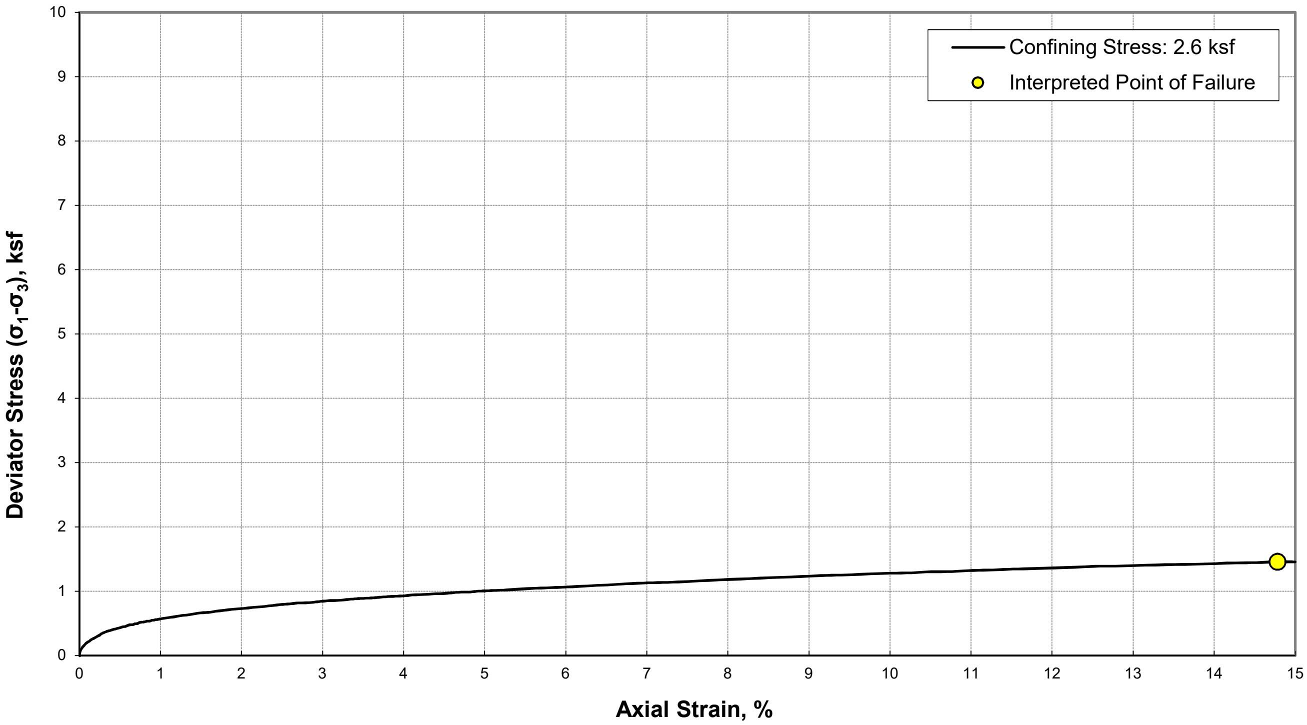
Maximum Deviator Stress, ksf	0.96
Undrained Shear Strength, ksf	0.48
Axial Strain at Failure, %	15.0
Strain Rate, %/min	1.0
Cell Pressure, ksf	2.2
Tested By:	JB
Date Tested:	1/20/20

REMARKS

Test Method: ASTM 2850

Note presence of approximate 2" sized gravel in upper portion of UU test sample.

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST
Laney College Library Learning Resource Center
Oakland, California



SAMPLE ID

Boring Number:	B-01
Sample Number:	S13
Sample Depth:	40.5 ft
USCS Classification:	Fat CLAY (CH): olive gray

SAMPLE PROPERTIES

Water Content, %	71.3%
Dry Unit Weight, pcf	59.1
Diameter, in	2.39
Height, in	5.79

SAMPLE IMAGES

CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
#4 (4.75mm)	---	Plastic Limit	---
#16 (1.18mm)	---	Plasticity Index	---
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

Maximum Deviator Stress, ksf	1.46
Undrained Shear Strength, ksf	0.73
Axial Strain at Failure, %	14.8
Strain Rate, %/min	1.0
Cell Pressure, ksf	2.6
Tested By:	JB
Date Tested:	1/20/20

REMARKS

Test Method: ASTM 2850

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

Laney College Library Learning Resource Center
Oakland, California

SUMMARY OF LABORATORY TEST RESULTS

Project: Laney College Library Learning Resource Center
Address: Oakland, California
Owner: Peralta Community College District

Job Number: 04.72190021
Date: 1/28/2020
Lab ID: 10044

Source:
Location Sampled: B-01, Laney College Library
Date Sampled: N/A
Sample By: N/A
Test Methods: ASTM D2974

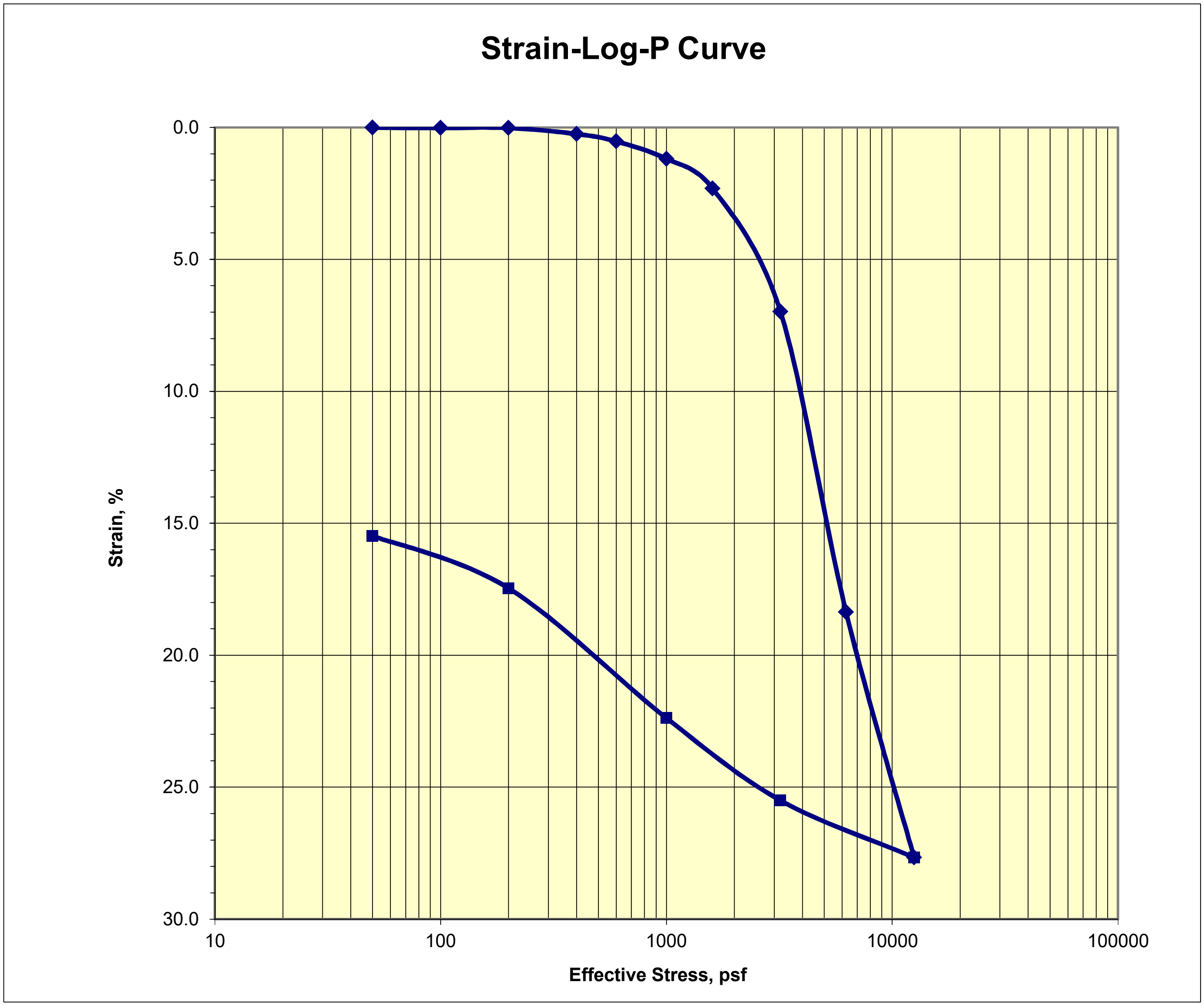
Sample No.	Depth (ft)	Sample Description	Water Content (%)	Ash Content (%)	Organic Content (%)
B-01	16	Poorly Graded SAND with SILT (SP - SM)	25.9	95.0	5.0
B-01	17	Organic CLAY with SAND (OH)	82.5	78.8	21.2
B-01	21	Fat CLAY (CH)	53.4	93.4	6.6

Remarks: None

Distribution:

Consolidation Test
ASTM D2435

Job No.:	446-303	**Boring:**	B-01	**Run By:**	MD
Client:	Fugro USA Land, Inc.	**Sample:**		**Reduced:**	PJ
Project:	04.72190021	**Depth, ft.:**	25-27.5(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY (Bay Mud)			**Date:**	2/4/2020



Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	82.9	65.6
Dry Density, pcf:	51.8	61.2
Void Ratio:	2.316	1.804
% Saturation:	98.4	100.0

Remarks:

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

10 April 2019

Job No. 1904058
Cust. No. 11608

Mr. Franco A. DePaola
Fugro Consultants, Inc.
1777 Botelho Drive, Suite 262
Walnut Creek, CA 94596

Subject: Project No.: 04.72190021
Project Name: Laney College, 900 Fallon St., Oakland, CA
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. DePaola:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 05, 2019. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations are 16 & 22 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 7.59 & 7.97, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 270 & 280-mV. These samples are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Fugro West, Inc.
Client's Project No.: 04.72190021
Client's Project Name: Laney College, 900 Fallon St., Oakland, CA
Date Sampled: 29-Mar-19
Date Received: 5-Apr-19
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 11-Apr-2019

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1904058-001	CPT-03 @ 4' - 5' (S-3)	270	7.97	-	2,600	-	N.D.	16
1904058-002	CPT-01 @ 2.5' - 3' (S-1)	280	7.59	-	6,400	-	N.D.	22

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	9-Apr-2019	9-Apr-2019	-	5-Apr-2019	-	9-Apr-2019	9-Apr-2019

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

PLATE B-9

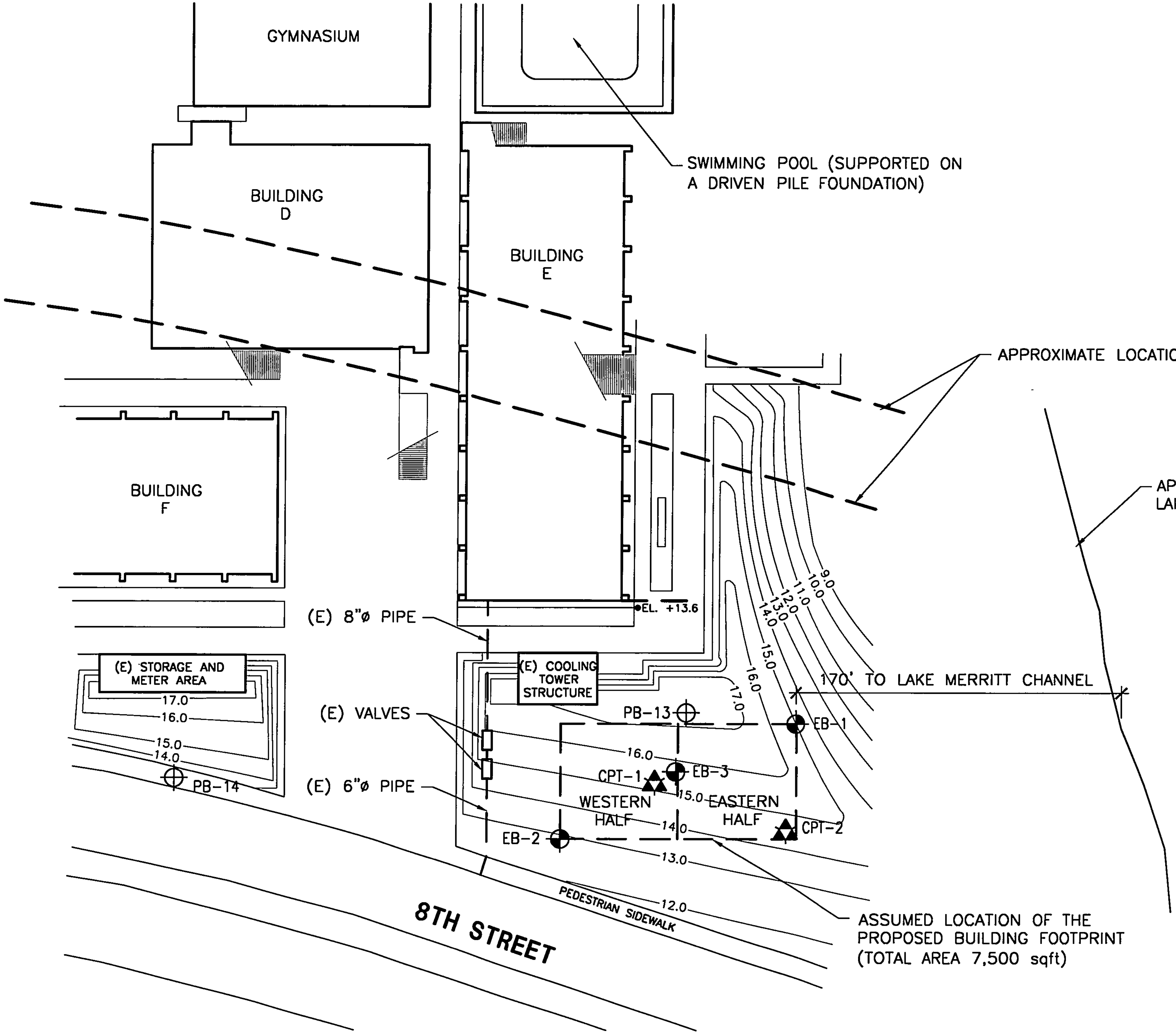
Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Appendix C

Previous Field Exploration Logs

and Laboratory Test Results

C.1 Exploration Boring Logs and CPTs by Fugro, February 2002, Fugro No. 1430.001



LEGEND

- EB-3 APPROXIMATE LOCATION OF EXPLORATORY BORING (2002)
- CPT-2 APPROXIMATE LOCATION OF CONE PENETRATION TEST (2002)
- PB-14 APPROXIMATE LOCATION OF PREVIOUS BORING BY OTHERS (1968)

NOTE:

1) GROUND CONTOUR LINES WERE BASED ON THE 1968 SITE PLAN. IT MAY HAVE CHANGED OVER TIME.

2) WESTERN HALF OF NEW ART BUILDING SUPPORTED BY 50' LONG PILES.

EASTERN HALF OF NEW ART BUILDING SUPPORTED BY 60' LONG PILES.

FUGRO
FUGRO WEST INC.
425 Roland Way.
Oakland, California. 94621
Tel:(510)568-4001 Fax:(510)568-2205

DRAWN BY:	ROC
PREP'D BY:	NS
APP'D BY:	SR
SCALE:	1" = 60'
DATE:	25FEB02
DWG FILE:	1430.001-01

SITE PLAN

NEW ART BUILDING AT LANEY COLLEGE
OAKLAND, CALIFORNIA

FIGURE 1

PROJECT No. 1430.001

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	14.4 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	15 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
FILL: CLAY (CL), dark brown, mottled, sandy (fine- to medium-grained), some silt, damp	Firm								
FILL: SAND (SM/SC), brown, mottled, fine- to coarse-grained, silty, some clay, trace gravel and shell fragment, damp	Medium Dense				19				
					13				
			5		21	16	116		PP = 2.5
grades to gray-brown at 6 feet									
					49	11	126		% of Passing #200 Sieve = 24
grades to blue-gray-brown, some silt at 10 feet			10						
BAY MUD: CLAY (CH), black, some sand (fine- to coarse-grained), some silt, mild hydrocarbon odor, trace wood fragment, moist									
	Firm				9				No Recovery
			15			▼			
grades to wet at 16 feet	Soft				6	▽			No Recovery
					21				See Note 7
			20						
strong hydrocarbon odor, with high amount of wood fragment, metal pieces, and other debris at 20 feet									
grades to blue-gray, silty below 23 feet	Firm				9	77	54		PP = 0.5
			25						
					8	74	56	1.1	PP = 1.0

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EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-1

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**14.4 Feet**	LOGGED BY	**NS**
DEPTH TO GROUND WATER	**15 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**2/26/02**

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						TESTS
BAY MUD: CLAY (CH), continued									
SAND (SM), dark green-gray, fine-grained, silty, some clay, trace shell fragment, wet	Loose				10	22	102		PP = 3.0
			35						
BAY MUD: CLAY (CH), blue-gray, silty, trace sand (fine- to medium-grained), wet	Firm				9	76	55	0.4*	PP = 1.5, See Note 8
			40						
SAND (SM/SC), blue-gray, fine- to medium-grained, silty, with clay, trace shell fragment, wet	Medium Dense				32	16	112		
			45						
	Very Dense				83/9"				
			50						
	Dense				37				
			55						
	Very Dense				63				

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Oakland, CA 94621

EXPLORATORY BORING LOG		
NEW ART BUILDING AT LANEY COLLEGE Oakland, CA		
PROJECT NO.	DATE	BORING NO.
1430.001	**February, 2002**	**EB-1**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	14.4 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	15 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SAND (SM/SC), continued grades to blue-gray-brown, trace gravel at 60 feet grades to brown, clayey below 63 feet	Dense		65	X	61	22	105	2.3	% of Passing #200 Sieve = 43 PP = 4.0
			70	X	67				PP = 4.5
			75	X	73				PP = 2.5

Bottom of Boring = 75 Feet

Notes:

1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of penetration resistance values, see first page of Appendix A.
3. A 140-lb safety hammer falling 30 inches was used to drive the sampler.
4. Ground water was encountered originally at depth of about 17 feet, and at depth of about 15 feet two hours later.
5. The borehole was backfilled with lean cement immediately upon completion of the drilling.
6. PP = Pocket Penetrometer Reading (tsf).
7. High value of blow count is due to localized encountering metal, brick, and/or concrete debris.
8. Low shear strength was probably caused by severe sample disturbance.

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425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	**February, 2002**	**EB-1**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	12.8 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	45 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						TESTS
FILL: CLAY (CL), dark brown, mottled, sandy (fine- to medium-grained), some silt, damp	Firm								
FILL: SAND (SM), brown, fine- to coarse-grained, silty, trace clay and gravel, damp	Medium Dense				15	13	110	1.3	PP = 2.0
					23				
	Loose		5		10				
grades to black, gravelly (subangular to subrounded) at 6 feet									
BAY MUD: CLAY (CH), blue-gray, silty, trace sand (fine- to coarse-grained) and wood fragmentl, moist	Soft				3				PP = 0.5
			10						
	Very Soft				2	50	74	0.2	PP < 0.5
			15						
grades to mottled shades of black-brown, trace shell fragment at 15 feet									
grades to dark gray-brown, mild hydrocarbon odor at 18 feet	Soft				4	78	54	0.3	PP = 0.5
			20						
					4				PP = 1.5

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Oakland, CA 94621

EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-2

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	12.8 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	45 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
BAY MUD: CLAY (CH), continued	Soft								
grades to gravelly (rounded to subrounded),wet at 28 feet									
CLAY (CL/GC), blue-gray, gravelly, some silt and sand, wet	Hard				57	17	114	9.1	
			30						
SAND (SP/SM), light brown, medium- to coarse-grained, trace gravel (subangular to subrounded) and silt, wet									
	Dense				37				
			35						
					32				% of Passing #200 Sieve = 19 between 29 feet to 59 feet
			40						
					32				
			45			▼			
					37				

FUGRO

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	**February, 2002**	**EB-2**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	12.8 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	45 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
SAND (SP/SM), continued	Dense								
			55						
CLAY (CL), olive-brown, silty, with sand (fine- to medium-grained), wet	Hard				67	21	109	12.3	PP = 4.5
			60						
			65						
grades to dark gray at 69 feet					76				
			70						

Bottom of Boring = 70 Feet

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see first page of Appendix A.
3. A 140-lb safety hammer falling 30 inches was used to drive the sampler.
4. Ground water was apparently encountered at depth of 45 feet at the time of drilling.
5. The borehole was backfilled with lean cement immediately upon completion of the drilling.
6. PP = Pocket Penetrometer Reading (tsf).

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425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-2

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**14.3 Feet**	LOGGED BY	**NS**
DEPTH TO GROUND WATER	**20 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**2/26/02**

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
FILL: CLAY (CL), dark brown, mottled, sandy (fine- to medium-grained), some silt, damp	Firm								
FILL: SAND (SM), dary gray-brown, mottled shades of green, fine- to coarse-grained, silty, some clay, trace gravel (subangular to subrounded), trace brick pieces, damp	Medium Dense		5	X	33	15	119	3.2	% of Passing #200 Sieve = 42
hard drilling due to encountering concrete or brick chunk			10	X	50/4"				No Recovery
BAY MUD: CLAY (CH), black, mottled shades of blue-gray, silty, mild hydrocarbon odor, moist	Very Soft		15						
			20	X	1	▽			PP < 0.5
			25						

FUGRO 425 Roland Way Oakland, CA 94621	EXPLORATORY BORING LOG **NEW ART BUILDING AT LANEY COLLEGE** **Oakland, CA**		
	PROJECT NO. **1430.001**	DATE **February, 2002**	BORING NO. **EB-3**

File Name: G:\ENGINEER\GINTW\PROJECTS\21127GI.GPJ Report Template: FUGRO Output Date: 3/26/02

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	14.3 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
BAY MUD: CLAY (CH), continued	Very Soft								
SAND (SM - SC), dark gray, medium- to coarse-grained, with silt, strong hydrocarbon odor, trace shell fragment, wet	Loose to medium dense				18				No Recovery
			30		8				
			35						
CLAY (CL), blue-gray, silty, with sand (fine- to coarse-grained), trace gravel, wet	Very Stiff				44	18	112	2.6	
			40						
SAND (SM - SC), dark brown, mottled shades of green, fine- to coarse-grained, clayey, some silt, trace gravel, wet	Dense								
			45						
					52				
			50						

Bottom of Boring = 50 Feet

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see first page of Appendix A.
3. A 140-lb safety hammer falling 30 inches was used to drive the sampler.
4. Ground water was encountered originally at depth of about 20 feet at the time of drilling.
5. The borehole was backfilled with lean cement immediately upon completion of the drilling.
6. PP = Pocket Penetrometer Reading (tsf).

FUGRO

425 Roland Way
Oakland, CA 94621

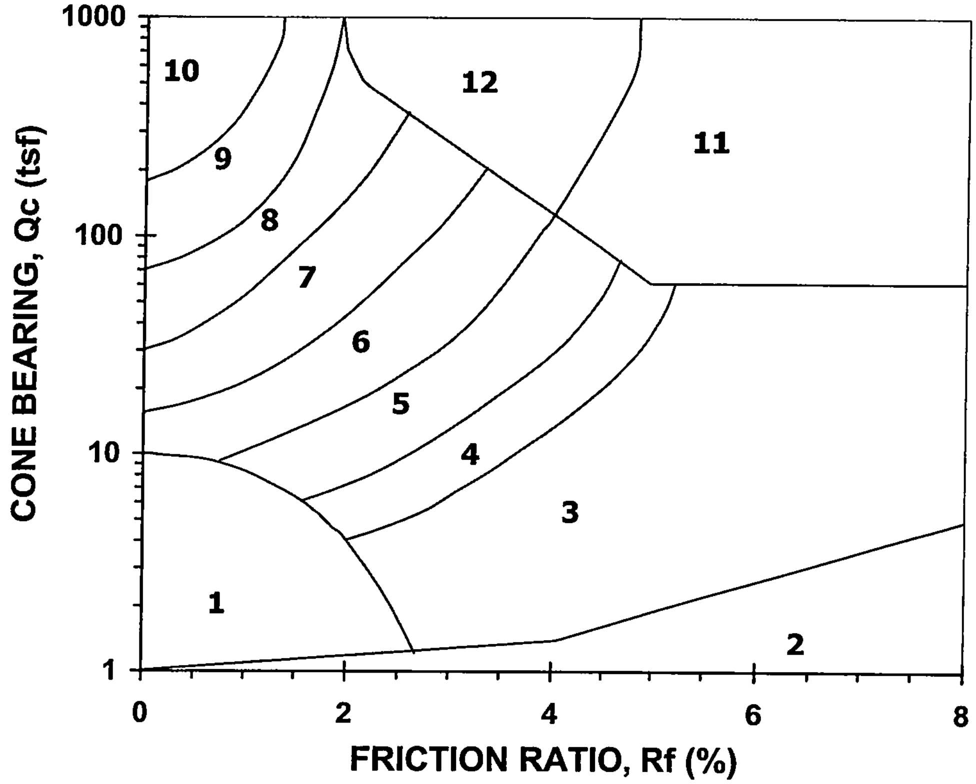
EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	**February, 2002**	**EB-3**

File Name: G:\ENGINEER\GINTW\PROJECTS\21127GI.GPJ Report Template: FUGRO Output Date: 3/26/02

SIMPLIFIED SOIL BEHAVIOR TYPE CLASSIFICATION FOR STANDARD ELECTRONIC CONE PENETROMETER



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2		Sensitive Fine Grained
2	1	for Zones 1 to 6	Organic Material
3	1	10 for Qc <= 9 tsf	CLAY
4	1.5	12 for Qc = 9 to 12 tsf	Silty CLAY to CLAY
5	2	15 for Qc > 12 tsf	Clayey SILT to Silty CLAY
6	2.5		Sandy SILT to Clayey SILT
7	3	---	Silty SAND to Sandy SILT
8	4	---	SAND to Silty SAND
9	5	---	SAND
10	6	---	Gravelly SAND to SAND
11	1	15	Very Stiff Fine Grained (*)
12	2	---	SAND to Clayey SAND (*)

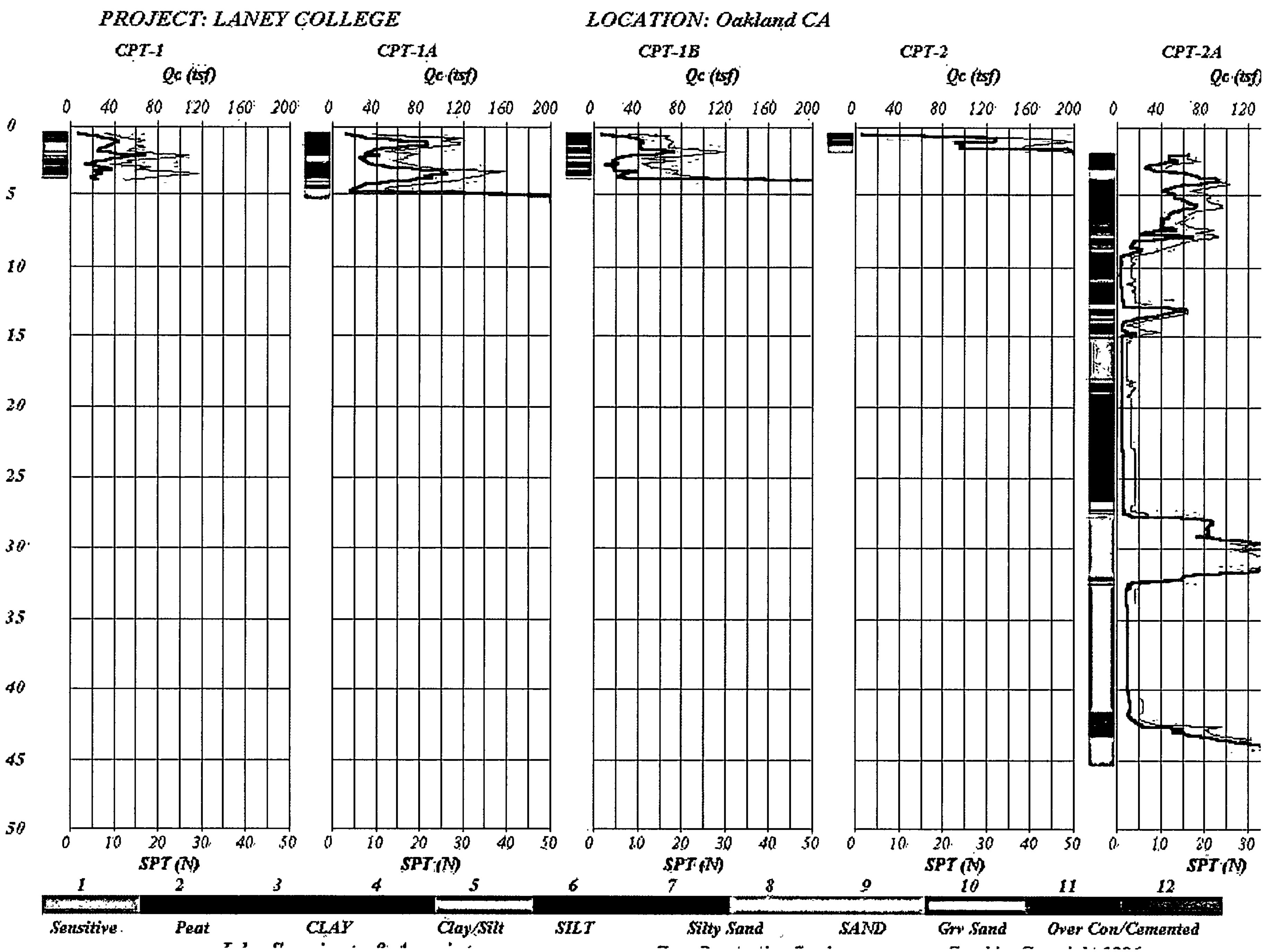
(*) Overconsolidated or Cemented

Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc*100 = Friction Ratio

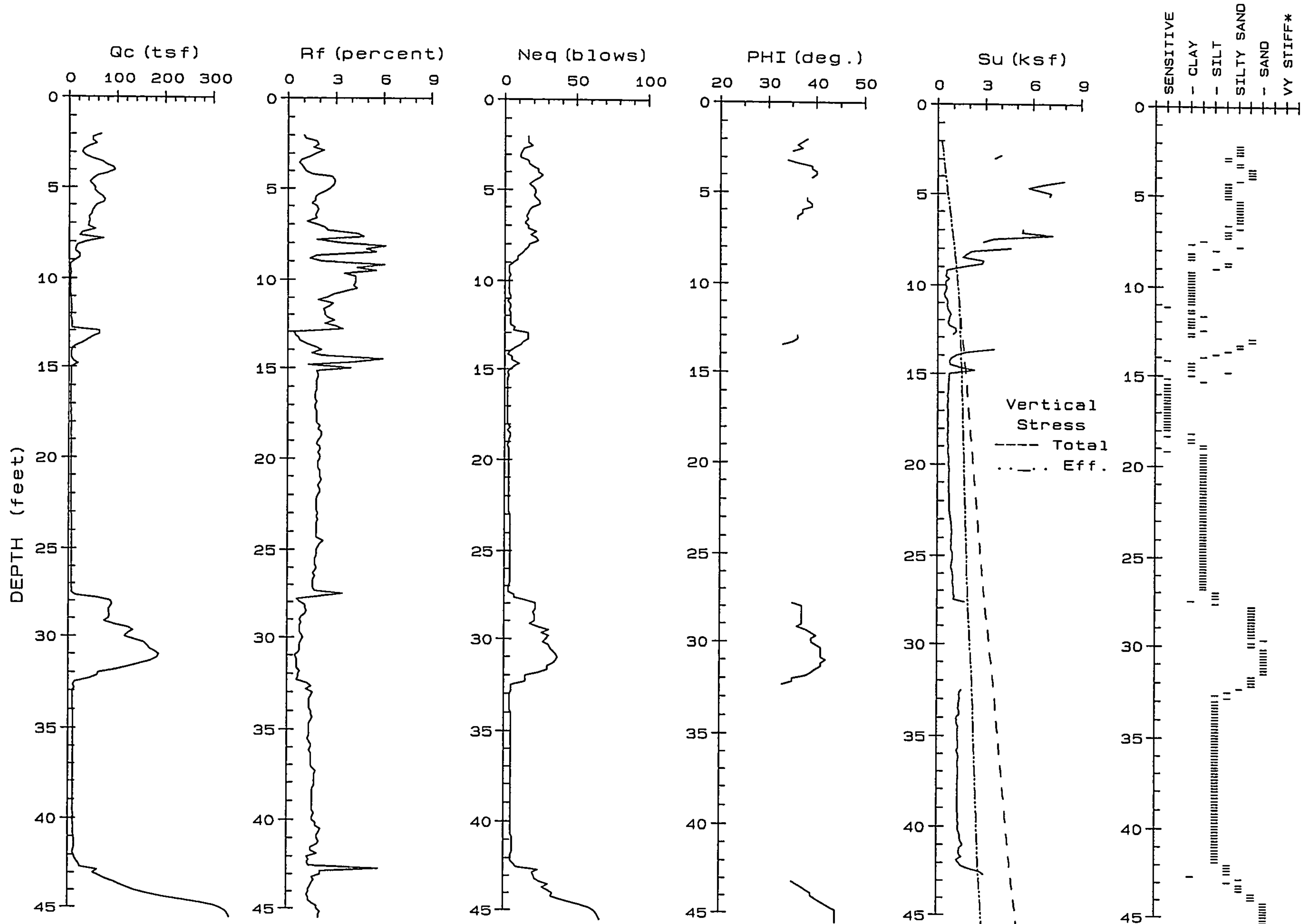
References: [1]Robertson, 1986, Olsen, 1988
[2]Bonaparte & Mitchell, 1979 (young bay mud Qc <= 9)
[2]Estimated from local experience (fine grained soils Qc > 9)

Note: Testing performed in accordance with ASTM D3441

John Sarmiento & Associates
Cone Penetrometer Testing Services



RESULTS OF CONE PENETROMETER TESTS (CPTs)



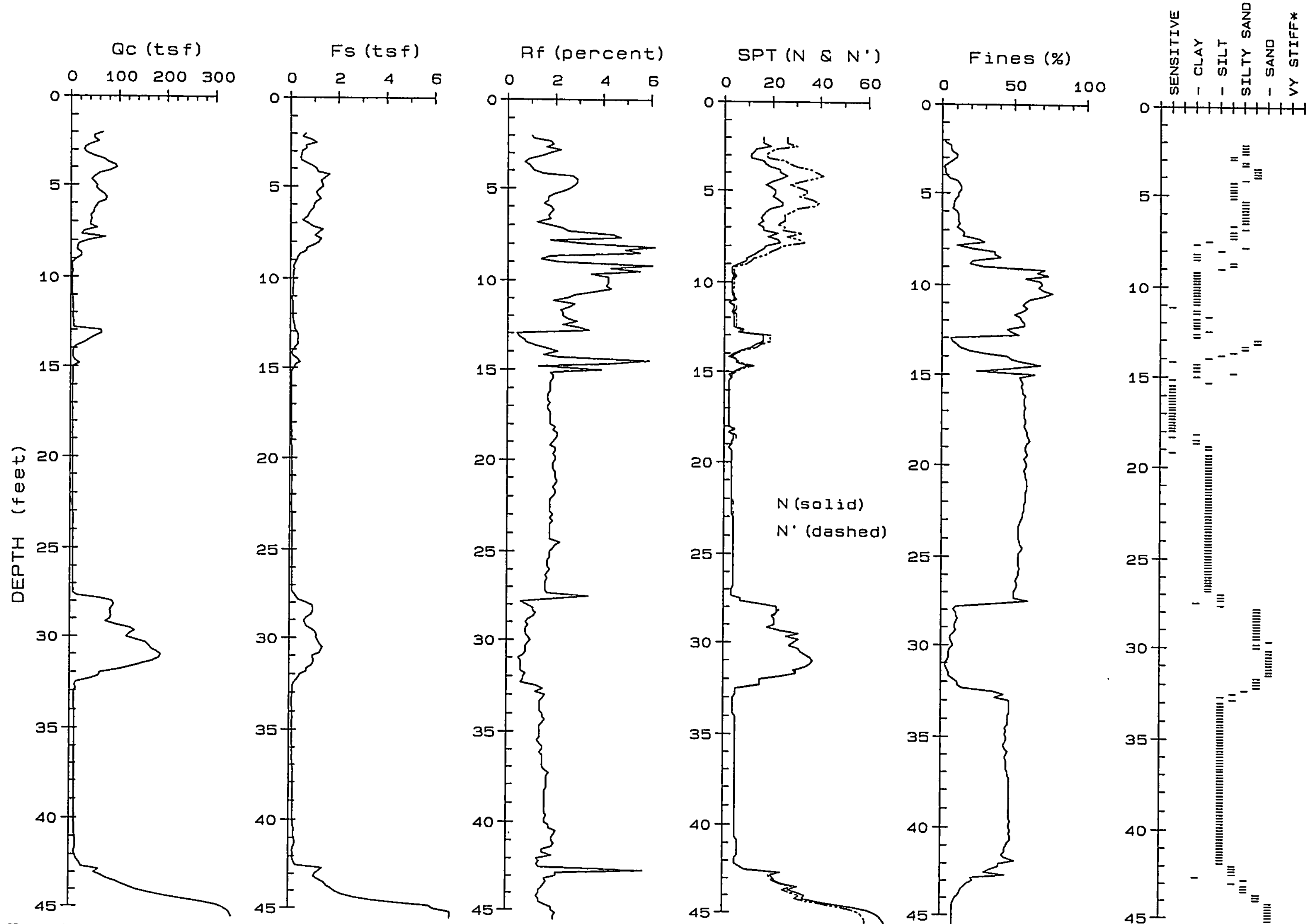
Terminated at 45.5 feet

Groundwater measured at 11.0 feet

PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1 (MWH-37)

CPT NO.: CPT-2A
DATE : 02-26-2002

John Sarmiento & Associates
Cone Penetration Testing Service



Terminated at 45.5 feet

Groundwater measured at 11.0 feet

PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1 (MWH-37)

CPT NO.: CPT-2A
DATE : 02-26-2002

John Sarmiento & Associates
Cone Penetration Testing Service

PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1(MWH-37)

CPT NO.: CPT-2A
DATE : 02-26-2002
Groundwater measured at 11.0 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
2.00	65.31	0.633	1.0	16	26	0.25	38	----	SAND to Silty SAND	"
2.50	55.84	1.069	1.9	19	30	0.31	37	----	Silty SAND to Sandy SILT	130-140
3.00	26.84	0.490	1.8	11	17	0.37	----	3.55	Sandy SILT to Clayey SILT	120-130
3.50	68.70	0.451	0.7	17	27	0.43	39	----	SAND to Silty SAND	"
4.00	93.79	1.068	1.1	23	38	0.50	40	----	"	"
4.50	49.78	1.437	2.9	20	32	0.56	----	6.60	Sandy SILT to Clayey SILT	130-140
5.00	53.17	1.359	2.6	21	34	0.63	----	7.05	"	"
5.50	67.05	1.135	1.7	22	36	0.70	38	----	Silty SAND to Sandy SILT	"
6.00	56.26	0.986	1.8	19	30	0.77	37	----	"	"
6.50	46.38	0.771	1.7	15	25	0.83	36	----	"	"
7.00	40.32	0.739	1.8	16	25	0.90	----	5.32	Sandy SILT to Clayey SILT	"
7.50	26.43	1.166	4.4	18	26	0.97	----	3.46	Silty CLAY to CLAY	"
8.00	34.45	1.180	3.4	17	24	1.03	----	4.52	Clayey SILT to Silty CLAY	"
8.50	12.15	0.668	5.5	12	17	1.10	----	1.55	CLAY	120-130
9.00	12.82	0.270	2.1	6	9	1.16	----	1.63	Clayey SILT to Silty CLAY	110-120
9.50	3.06	0.167	5.5	3	4	1.20	----	0.49	CLAY	90-100
10.00	3.14	0.131	4.2	3	4	1.25	----	0.50	"	"
10.50	2.67	0.114	4.3	3	3	1.30	----	0.40	"	"
11.00	3.82	0.096	2.5	4	5	1.35	----	0.63	"	"
11.50	4.21	0.110	2.6	4	5	1.39	----	0.70	"	"
12.00	4.25	0.097	2.3	4	5	1.44	----	0.71	"	"
12.50	6.40	0.149	2.3	4	5	1.49	----	1.13	Silty CLAY to CLAY	"
13.00	62.43	0.258	0.4	16	19	1.54	36	----	SAND to Silty SAND	120-130
13.50	38.26	0.318	0.8	13	15	1.60	33	----	Silty SAND to Sandy SILT	110-120
14.00	6.53	0.139	2.1	4	5	1.66	----	1.14	Silty CLAY to CLAY	90-100
14.50	4.72	0.276	5.9	5	6	1.70	----	0.77	CLAY	100-110
15.00	4.41	0.172	3.9	4	5	1.76	----	0.71	"	90-100
15.50	4.27	0.080	1.9	2	2	1.81	----	0.67	Sensitive Fine Grained	"
16.00	4.19	0.075	1.8	2	2	1.85	----	0.65	"	"
16.50	4.13	0.072	1.7	2	2	1.90	----	0.64	"	85-90
17.00	4.17	0.072	1.7	2	2	1.94	----	0.64	"	"
17.50	4.17	0.075	1.8	2	2	1.99	----	0.63	"	90-100
18.00	4.24	0.076	1.8	2	2	2.04	----	0.64	"	"
18.50	4.23	0.090	2.1	4	5	2.08	----	0.64	CLAY	"
19.00	4.39	0.085	1.9	3	3	2.13	----	0.67	Silty CLAY to CLAY	"
19.50	4.45	0.089	2.0	3	3	2.18	----	0.67	"	"
20.00	4.47	0.089	2.0	3	3	2.23	----	0.67	"	"
20.50	4.70	0.091	1.9	3	3	2.27	----	0.71	"	"
21.00	4.68	0.093	2.0	3	3	2.32	----	0.70	"	"
21.50	4.74	0.092	1.9	3	3	2.37	----	0.71	"	"
22.00	4.80	0.093	1.9	3	3	2.42	----	0.72	"	"
22.50	4.86	0.089	1.8	3	3	2.46	----	0.73	"	"
23.00	5.38	0.100	1.9	4	4	2.51	----	0.82	"	"
23.50	5.77	0.106	1.8	4	4	2.56	----	0.90	"	"
24.00	5.81	0.104	1.8	4	4	2.61	----	0.90	"	"
24.50	5.73	0.125	2.2	4	4	2.65	----	0.88	"	"
25.00	5.95	0.108	1.8	4	4	2.70	----	0.92	"	"
25.50	5.87	0.102	1.7	4	4	2.75	----	0.90	"	"
26.00	6.01	0.109	1.8	4	4	2.80	----	0.92	"	"
26.50	6.11	0.102	1.7	4	4	2.84	----	0.94	"	"
27.00	6.52	0.105	1.6	3	3	2.89	----	1.02	Clayey SILT to Silty CLAY	"
27.50	6.50	0.224	3.4	7	7	2.94	----	1.01	CLAY	100-110
28.00	86.18	0.708	0.8	22	22	3.00	37	----	SAND to Silty SAND	120-130
28.50	81.76	0.985	1.2	20	21	3.06	37	----	"	"
29.00	82.38	0.631	0.8	21	21	3.13	37	----	"	"
29.50	122.59	0.928	0.8	31	31	3.19	39	----	"	"
30.00	116.31	1.119	1.0	29	29	3.25	39	----	"	"
30.50	161.98	1.367	0.8	32	32	3.31	41	----	SAND	"
31.00	185.64	0.963	0.5	37	37	3.38	42	----	"	"
31.50	147.89	0.941	0.6	30	29	3.44	40	----	"	"
32.00	61.40	0.446	0.7	15	15	3.50	35	----	SAND to Silty SAND	"
32.50	13.03	0.172	1.3	5	5	3.56	----	1.50	Sandy SILT to Clayey SILT	100-110
33.00	8.97	0.147	1.6	4	4	3.61	----	1.43	Clayey SILT to Silty CLAY	"
33.50	8.49	0.122	1.4	4	4	3.66	----	1.33	"	90-100
34.00	9.09	0.134	1.5	5	5	3.71	----	1.21	"	"

John Sarmiento & Associates
Cone Penetration Testing Service

PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1(MWH-37)

CPT NO.: CPT-2A
DATE : 02-26-2002
Groundwater measured at 11.0 feet

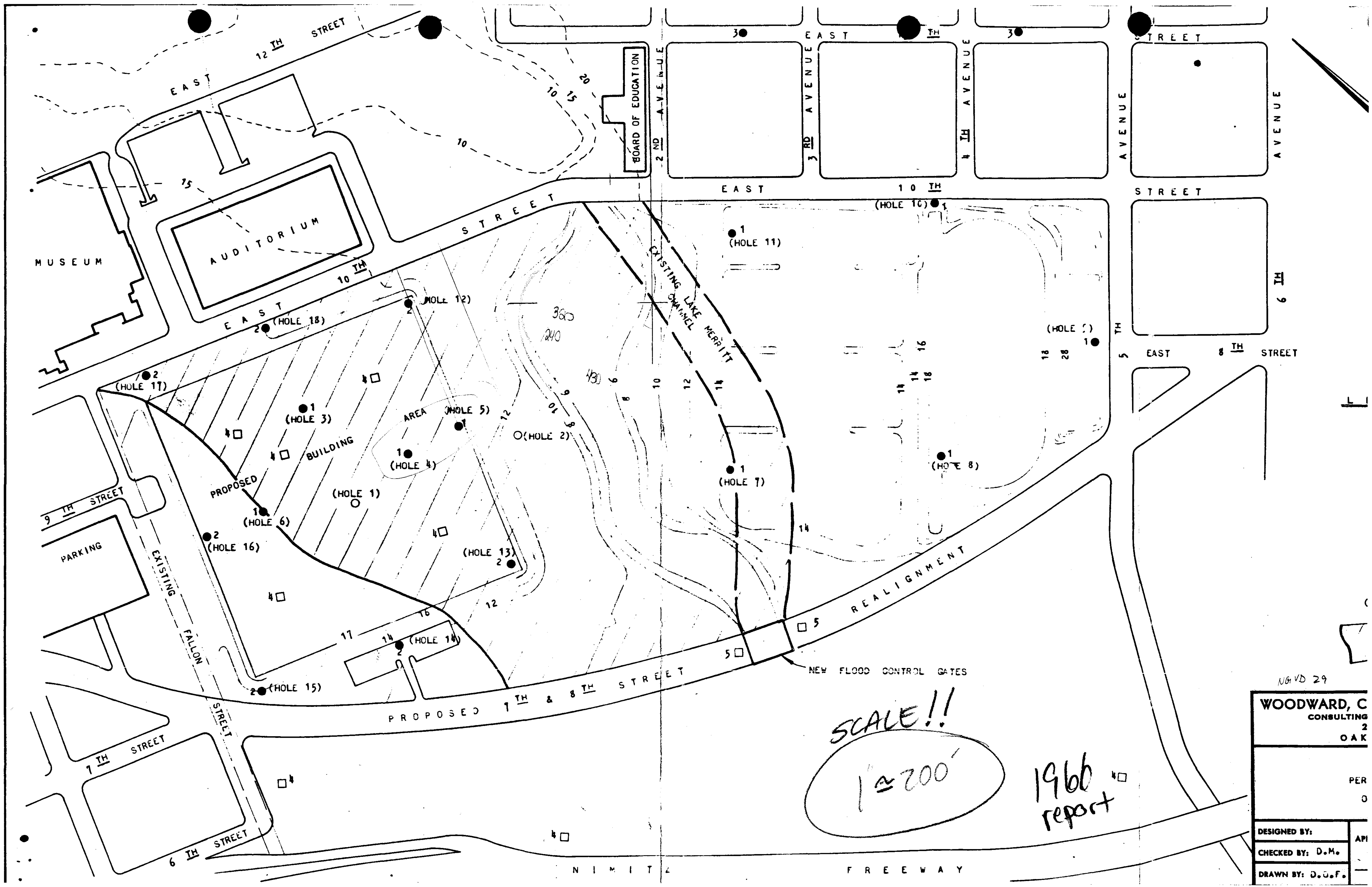
DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
34.50	9.78	0.143	1.5	5	5	3.75	----	1.32	"	"
35.00	9.60	0.138	1.4	5	5	3.80	----	1.28	"	"
35.50	9.93	0.126	1.3	5	5	3.85	----	1.33	"	"
36.00	9.83	0.147	1.5	5	5	3.90	----	1.31	"	100-110
36.50	9.82	0.144	1.5	5	5	3.95	----	1.31	"	90-100
37.00	9.93	0.153	1.5	5	5	4.00	----	1.32	"	100-110
37.50	9.82	0.168	1.7	5	5	4.05	----	1.30	"	"
38.00	9.79	0.163	1.7	5	5	4.10	----	1.29	"	"
38.50	9.87	0.161	1.6	5	5	4.16	----	1.30	"	"
39.00	9.87	0.161	1.6	5	5	4.21	----	1.29	"	"
39.50	10.14	0.167	1.6	5	5	4.26	----	1.33	"	"
40.00	10.14	0.165	1.6	5	5	4.31	----	1.33	"	"
40.50	10.91	0.227	2.1	5	5	4.37	----	1.46	"	"
41.00	11.94	0.232	1.9	6	6	4.42	----	1.62	"	"
41.50	12.51	0.191	1.5	6	6	4.48	----	1.37	"	"
42.00	12.66	0.161	1.3	5	5	4.53	----	1.39	Sandy SILT to Clayey SILT	"
42.50	22.52	0.304	1.4	9	9	4.58	----	2.70	"	110-120
43.00	52.04	1.108	2.1	21	20	4.65	----	6.63	"	130-140
43.50	92.46	1.483	1.6	31	29	4.72	37	----	Silty SAND to Sandy SILT	"
44.00	137.71	1.911	1.4	34	32	4.78	39	----	SAND to Silty SAND	"
44.50	232.01	3.349	1.4	46	42	4.85	42	----	SAND	"
45.00	314.60	5.980	1.9	63	57	4.92	44	----	"	"
45.50	332.90	6.687	2.0	67	59	4.99	44	----	"	"

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance
Fs = Sleeve friction resistance
Rf = Tip/Sleeve ratio
SPT = Equivalent Standard Penetration Test*
TotStr = Total Stress using est. density**
Phi = Soil friction angle*
Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf) (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)

References: * Robertson and Campanella, 1988
** Olsen, 1989 *** Durgunoglu & Mitchell, 1975

John Sarmiento & Associates
Cone Penetration Testing Service

C.2 Exploratory Boring Logs and Lab Results by WCS, November 1965, WCS No. S10312

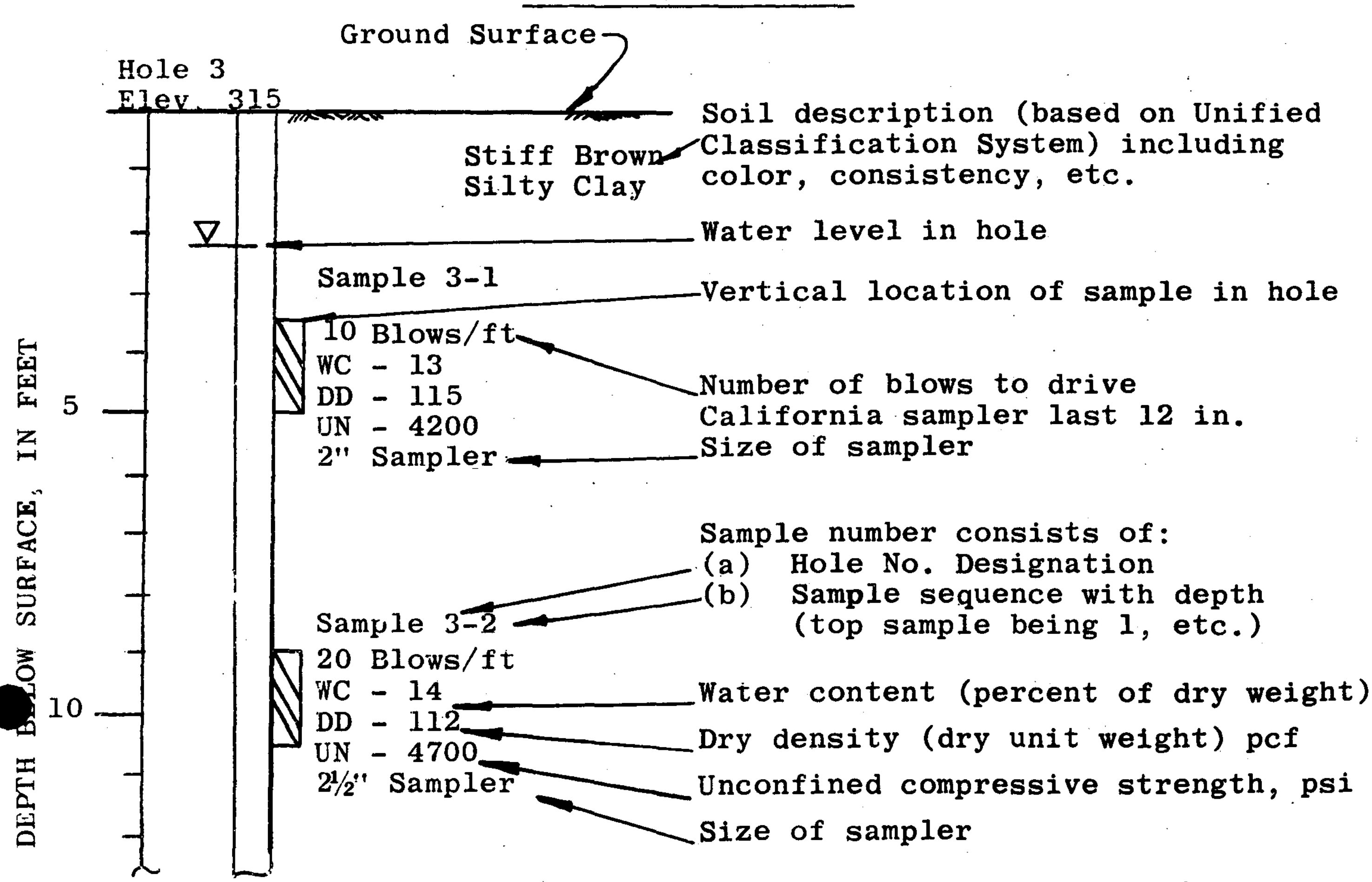

EAST 12TH STREET
MUSEUM
AUDITORIUM
EAST 10TH STREET
BOARD OF EDUCATION
2ND AVENUE
3RD AVENUE
4TH AVENUE
AVENUE
AVENUE
EAST TH STREET
EAST 10TH STREET
6TH
5TH
EAST 8TH STREET
EXISTING LAKE MERRITT CHANNEL
(HOLE 1)
(HOLE 2)
(HOLE 3)
(HOLE 4)
(HOLE 5)
(HOLE 6)
(HOLE 7)
(HOLE 8)
(HOLE 10)
(HOLE 11)
(HOLE 12)
(HOLE 13)
(HOLE 14)
(HOLE 15)
(HOLE 16)
(HOLE 17)
(HOLE 18)
PROPOSED BUILDING AREA
9TH STREET
PARKING
EXISTING FALLON STREET
7TH STREET
6TH STREET
REALIGNMENT
PROPOSED 7TH & 8TH STREET
NEW FLOOD CONTROL GATES
NIMITZ FREEWAY
SCALE!!
1" ≈ 200'
1966 report
NGVD 29
WOODWARD, C
CONSULTING
OAK
PER
DESIGNED BY:
CHECKED BY: D.M.
DRAWN BY: D.O.F.

A P P E N D I X

NOTES ON FIELD INVESTIGATION

1. Borings were advanced with a 6-in. diameter continuous flight power auger and by wash boring.
2. The Engineering Geologist were M. Conant, R. Russell and C. Taylor
3. In-place samples of the soils were obtained with either drive samplers or Shelby tube samplers. The size of sampler used is indicated at the sample location on the logs of borings.
   a) The 2-in. sampler measures 2-in. I.D. and 2½-in. O.D.. Thin brass liners are enclosed in the sampler. The sampler is driven 18-in. into the soil at the bottom of the holes with a 140 lb. hammer falling 30 in.
   b) The 2½-in. sampler measures 2½-in. I.D. and 2¾-in. O.D. and also contains brass liners. This sampler is driven 24-in. into the soil with a 140 lb. hammer falling 30 in.
   c) Shelby tube samplers are thin-walled brass tubes, measuring either 2.8 or 3.2 I.D., and are pushed into the soil by hydraulic mechanism. Loss of the sample is prevented by either a fixed piston in the Osterberg type sampler or by ball check valve in the open type sampler.
4. When the sampler was withdrawn from the test holes, the brass tubes containing the soils samples were removed, carefully sealed to preserve the natural moisture content, and returned to the laboratory for testing.
5. Classifications are based on the Unified Classification System and are made in the field by our Engineer or Geologist. Classifications of in-place samples are verified by an examination by the Staff Engineer.

KEY TO BORING LOGS



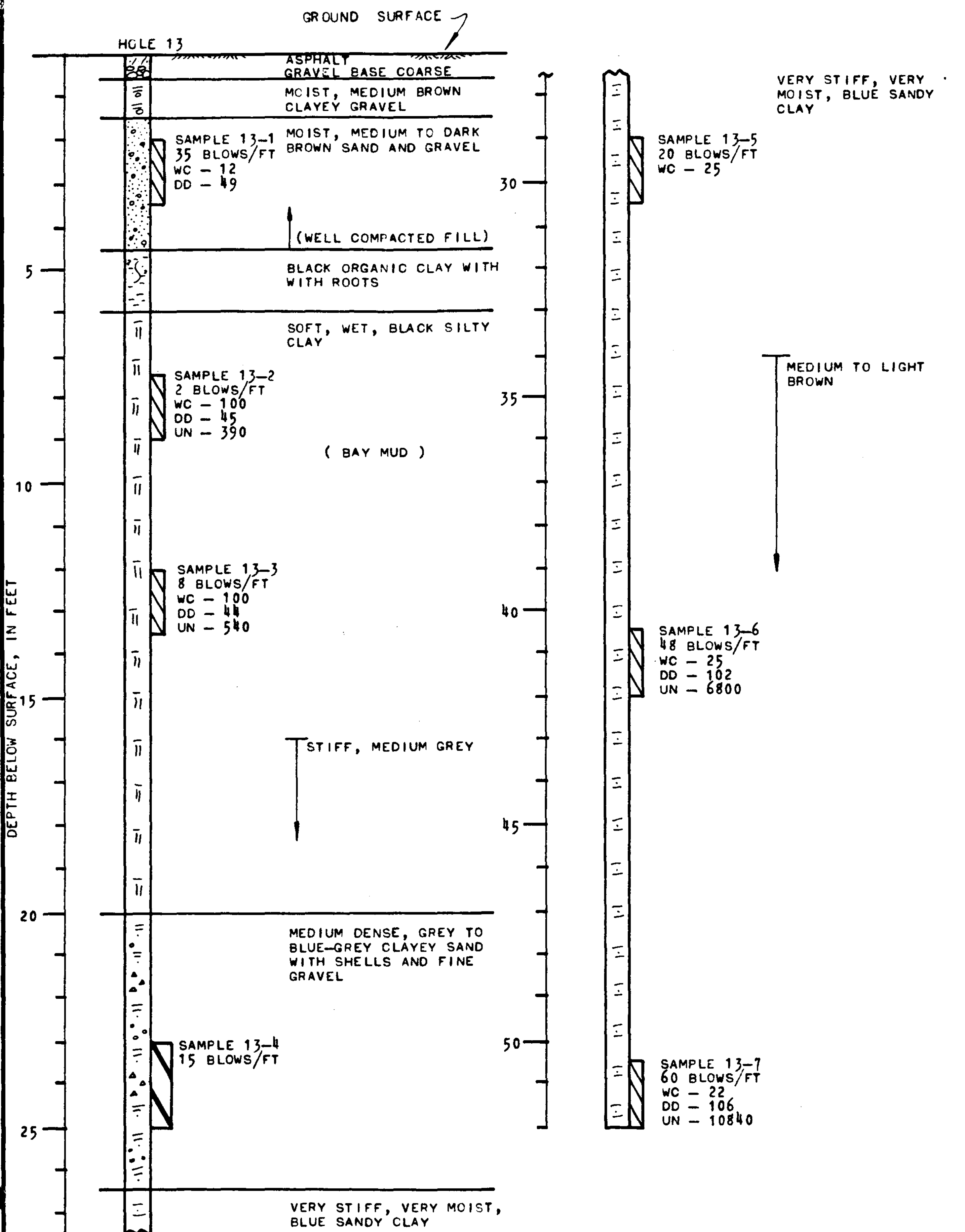


FIG. 20 — LOGS OF BORINGS

WOODWARD-CLYDE-SHERARD & ASSOCIATES
Consulting Soil and Foundation Engineers

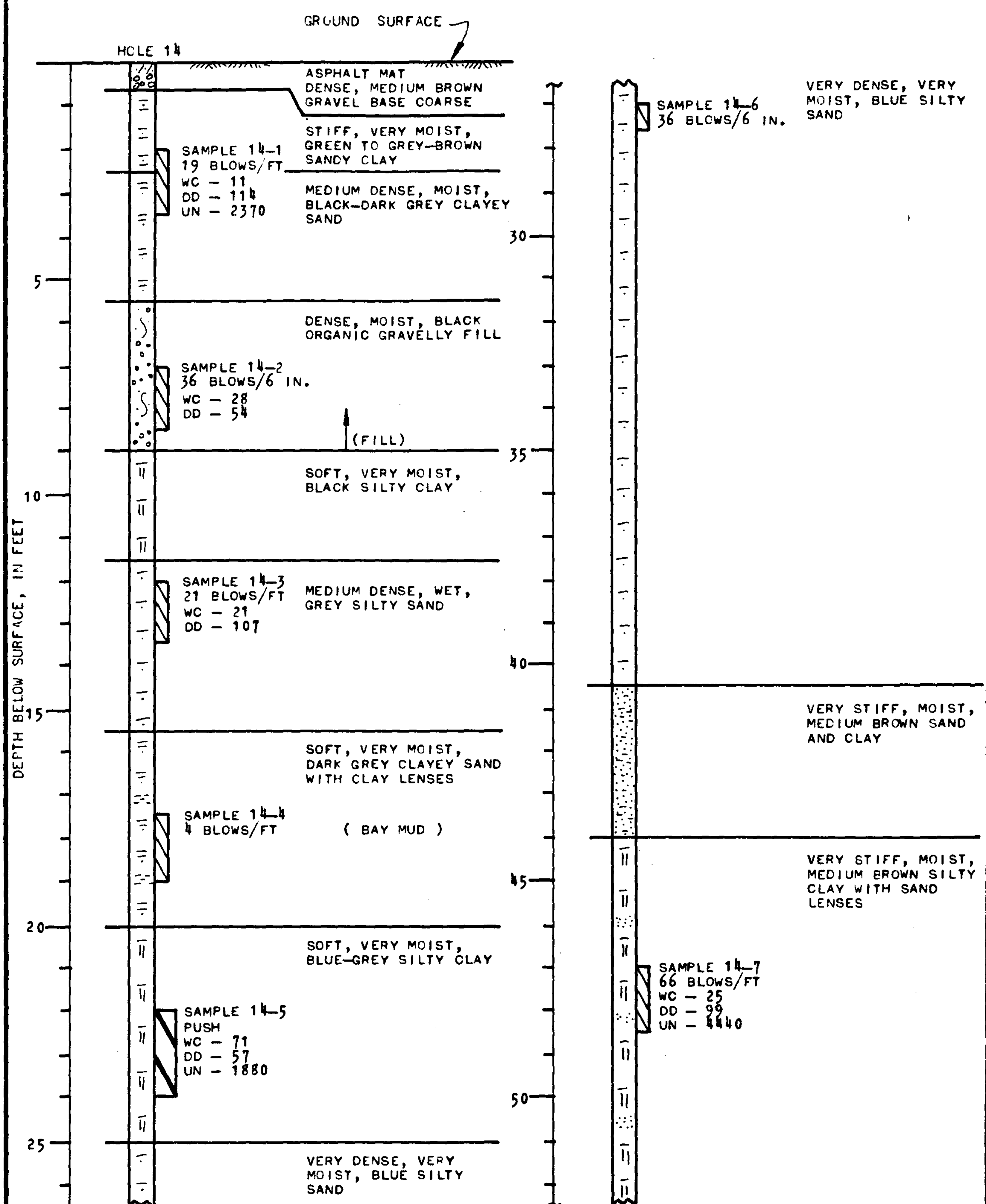


FIG. 21 — LOGS OF BORINGS

WOODWARD-CLYDE-SHERARD & ASSOCIATES
Consulting Soil and Foundation Engineers

HOLE 14 (CONT'D)

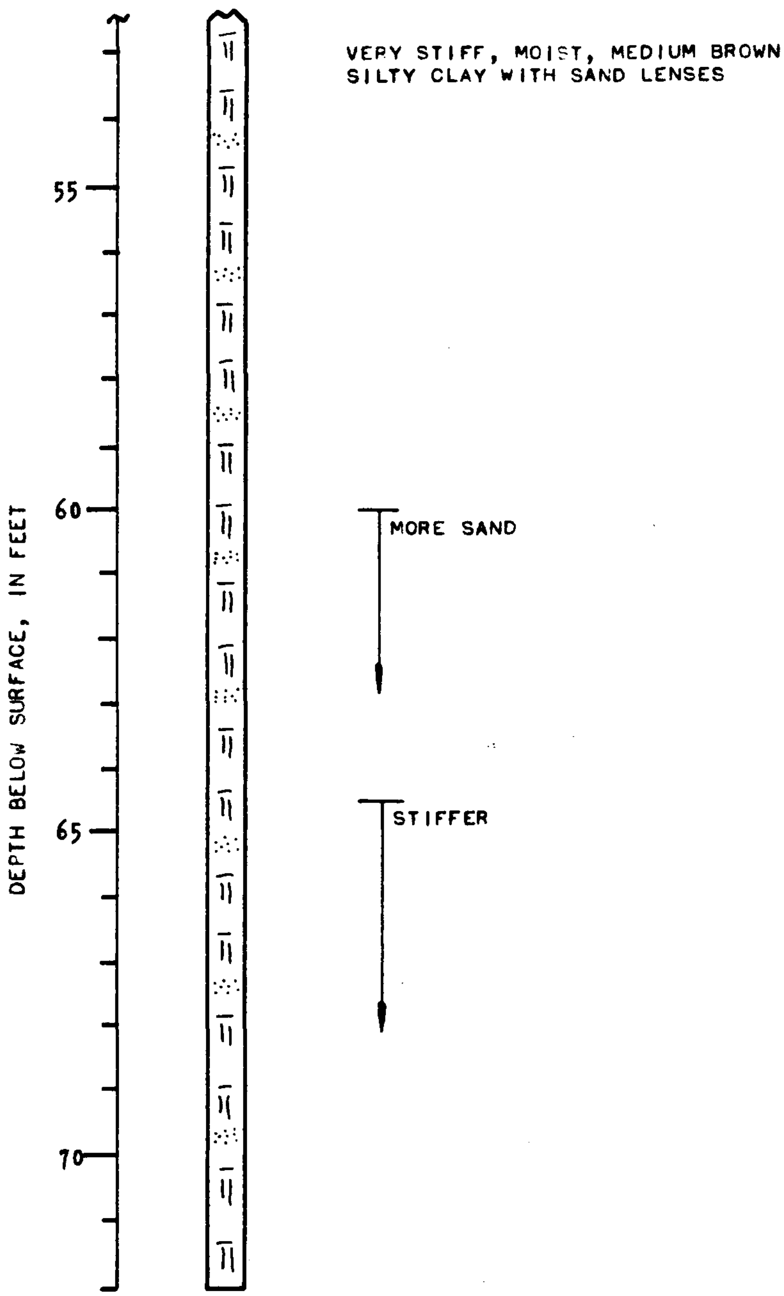
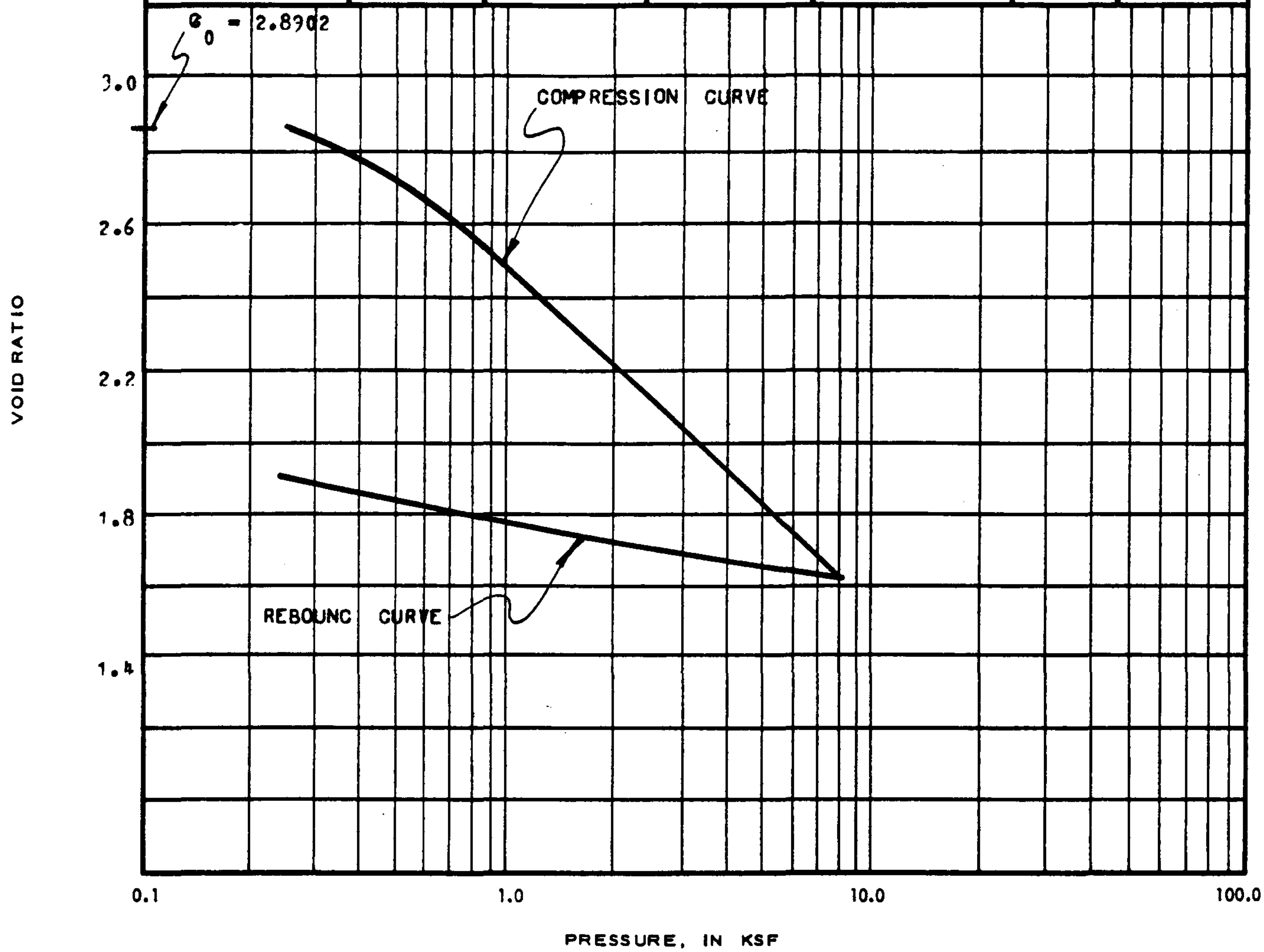
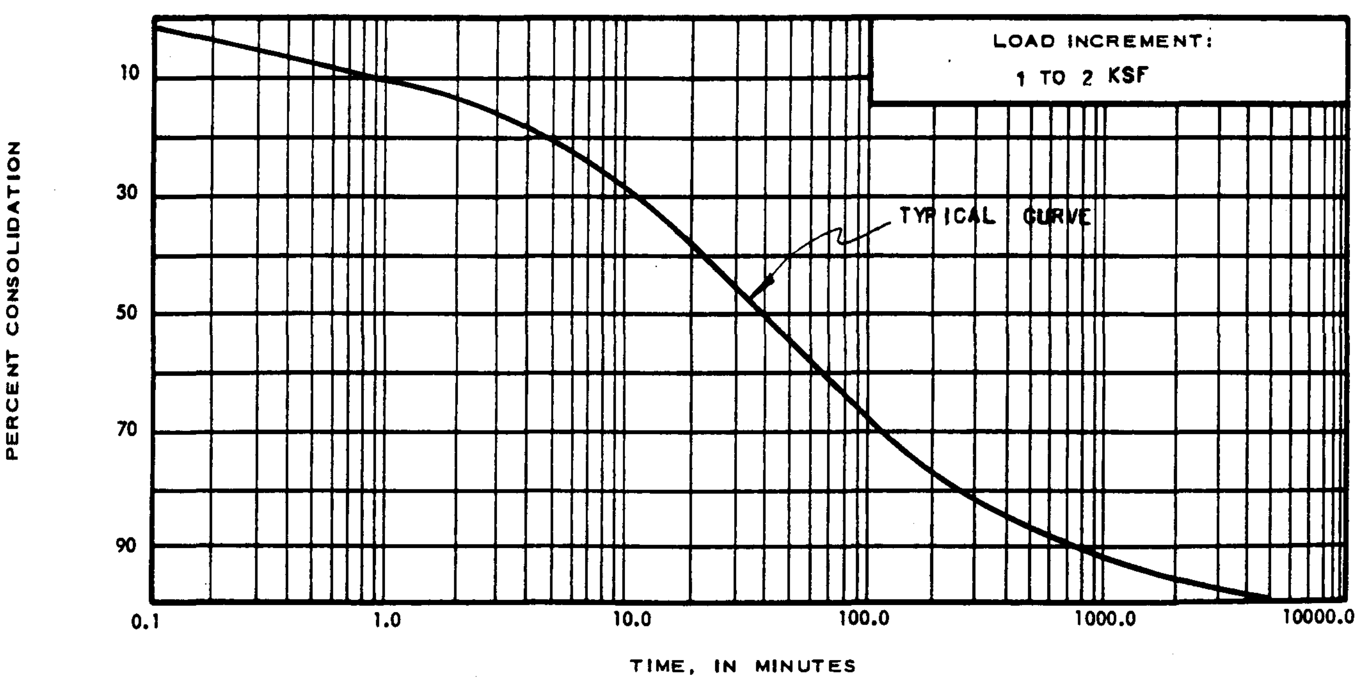


FIG. 22 — LOGS OF BORINGS

SAMPLE NO. 13-3-9	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY, (PCF)	PERCENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.75	102.24	44.10	97.28	.8981	1.94
FINAL		68.96	52.17	99.81	.6694	





CONSOLIDATION TEST

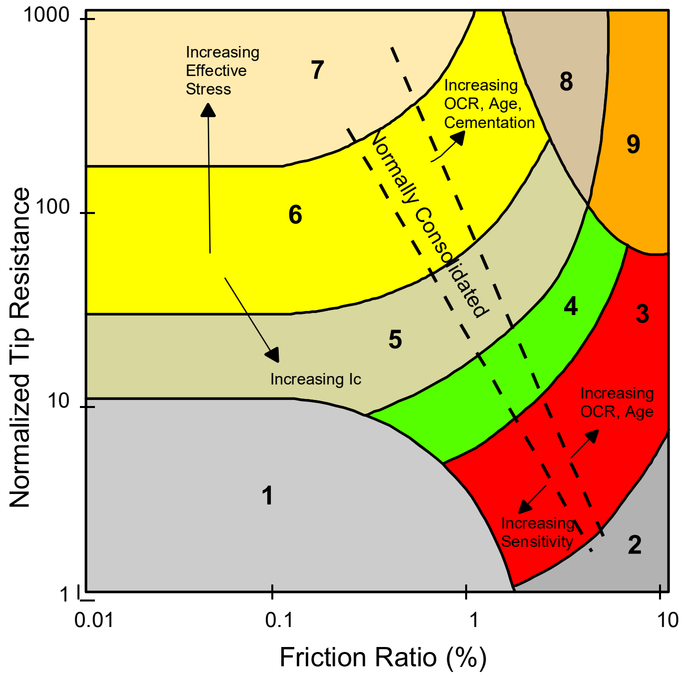
FIG. 26

Appendix D

Liquefaction Triggering and

Post-Liquefaction Deformation Analyses

CPT CORRELATION CHART
(Robertson 1990)



Zone	Soil Behavior Type
1	Sensitive Fine-grained
2	Peats
3	Silty Clay to Clay
4	Clayey Silt to Silty Clay
5	Silty Sand to Sandy Silt
6	Clean Sand to Silty Sand
7	Gravelly Sand to Dense Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine-Grained*

*heavily overconsolidated or cemented

CPT LOG COMPONENTS

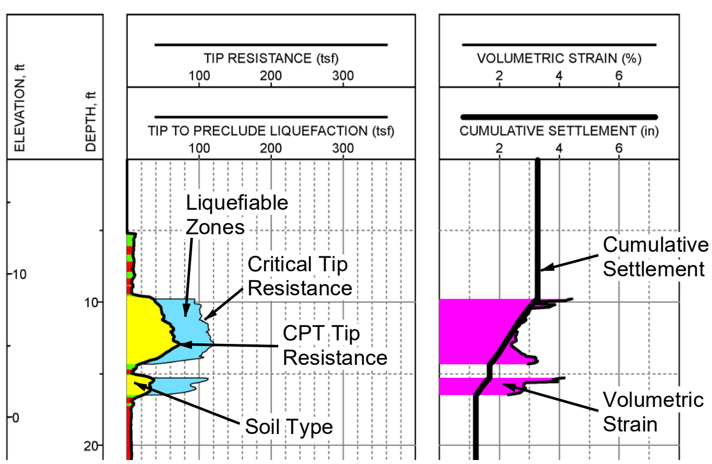
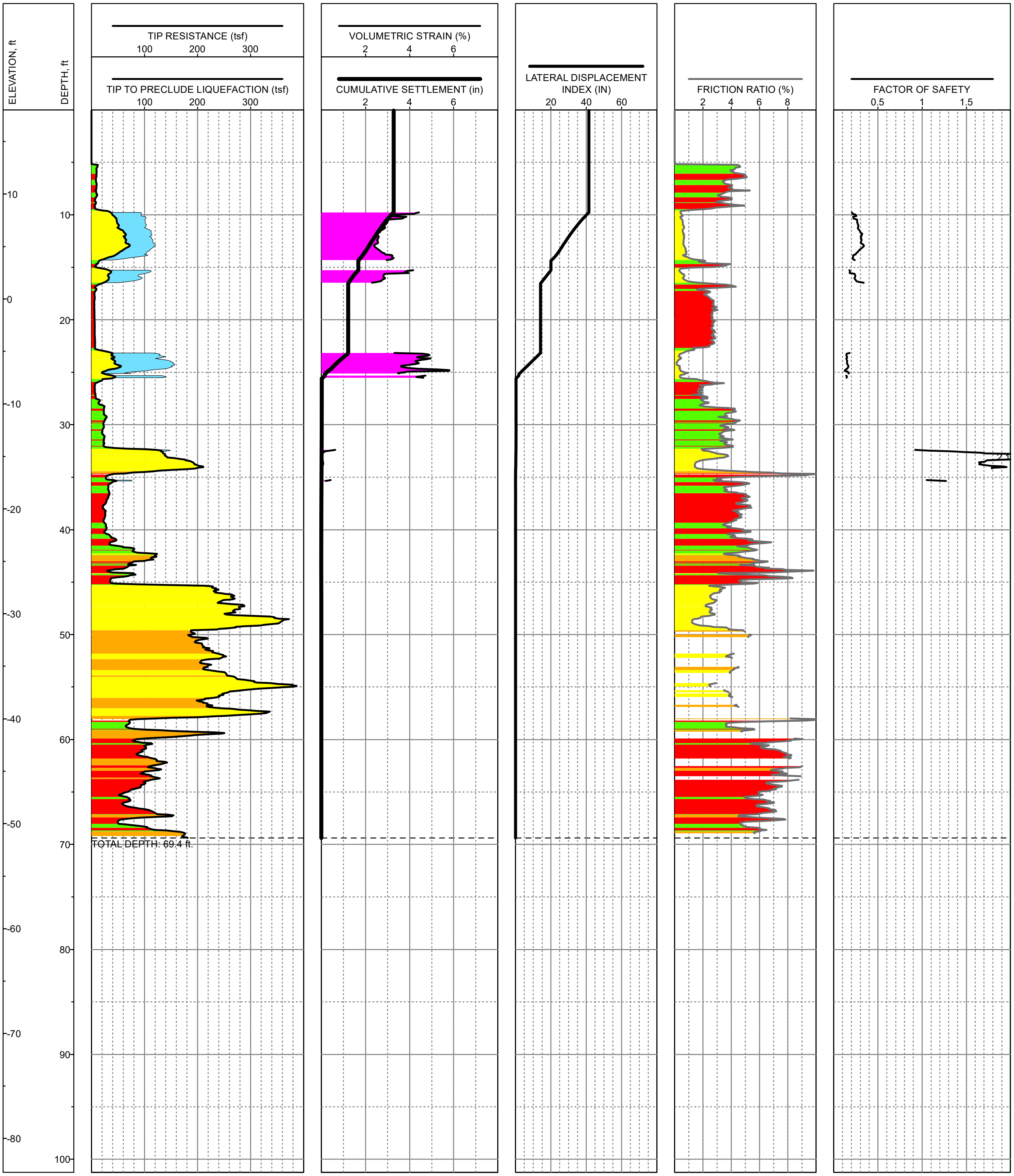


PLATE D-0: KEY TO LIQUEFACTION LOGS

FUGRO

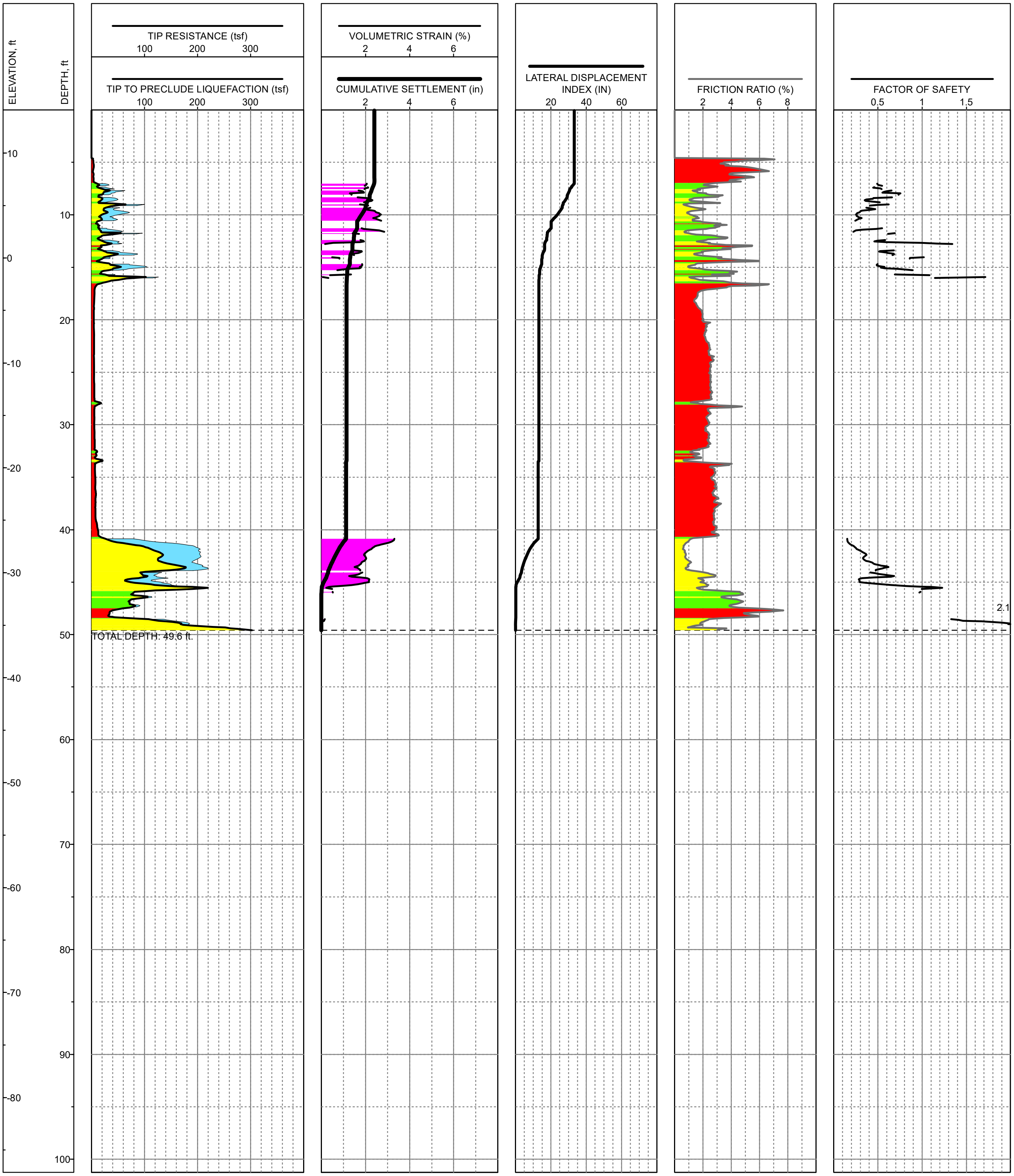


LOCATION: E6,052,365, N2,116,794, NAD83 SP CA Z3 FT
SURFACE EL: 18.02 ft
COMPLETION DEPTH: 69.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_N_TL_TR\MXD\CPT_Logs_M7_0_a0_810_N_TL_TR.6/21/2019.A.Ramirez

PLATE D-1: LOG OF 2019-CPT-01 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,593, N2,116,694, NAD83 SP CA Z3 FT
SURFACE EL: 14.11 ft
COMPLETION DEPTH: 49.6 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-2: LOG OF 2019-CPT-02 – M=7.0, PGA=0.810, N, TL, TR

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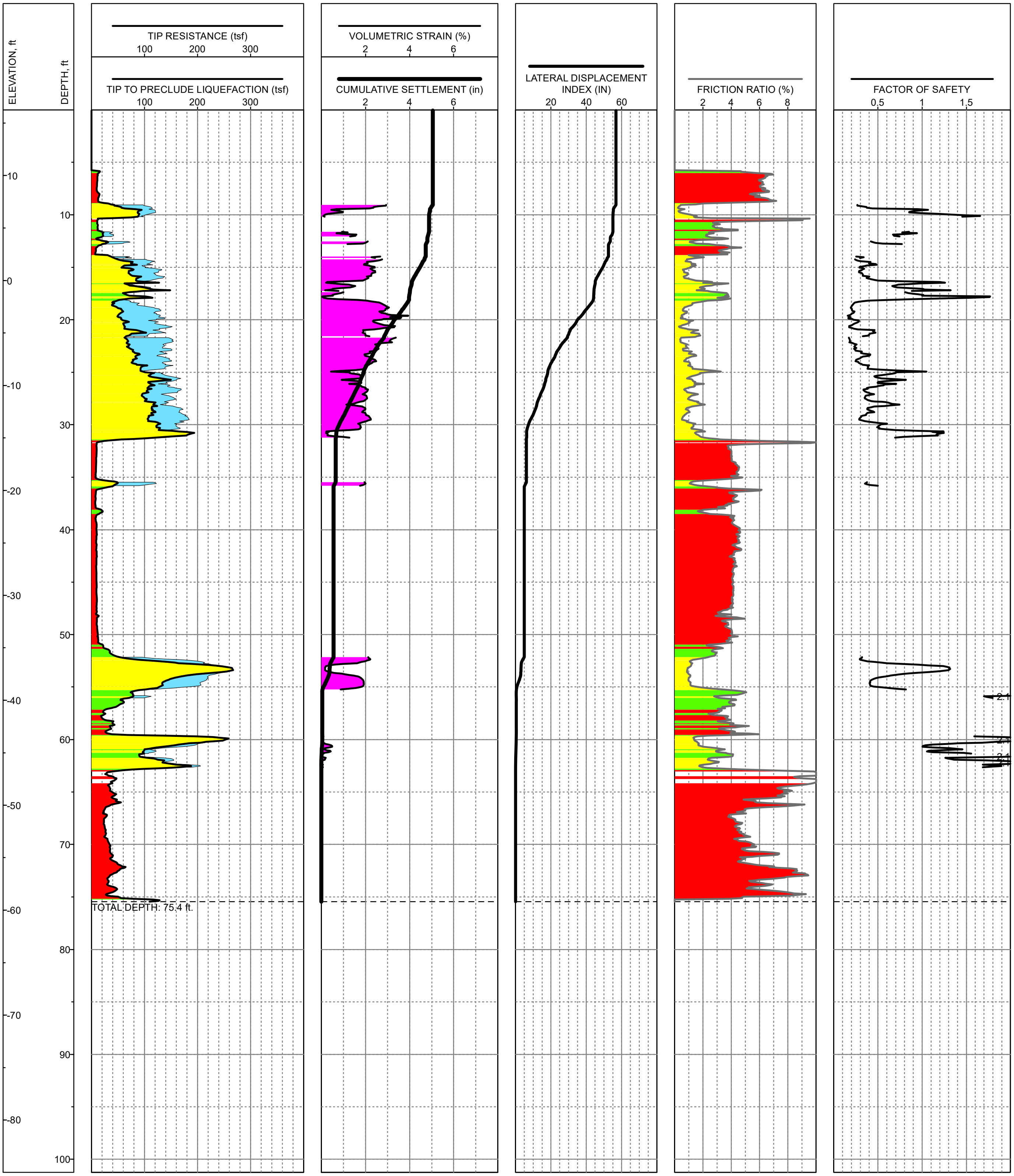

LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16.26 ft
COMPLETION DEPTH: 75.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-3: LOG OF 2019-CPT-03 – M=7.0, PGA=0.810, N, TL, TR

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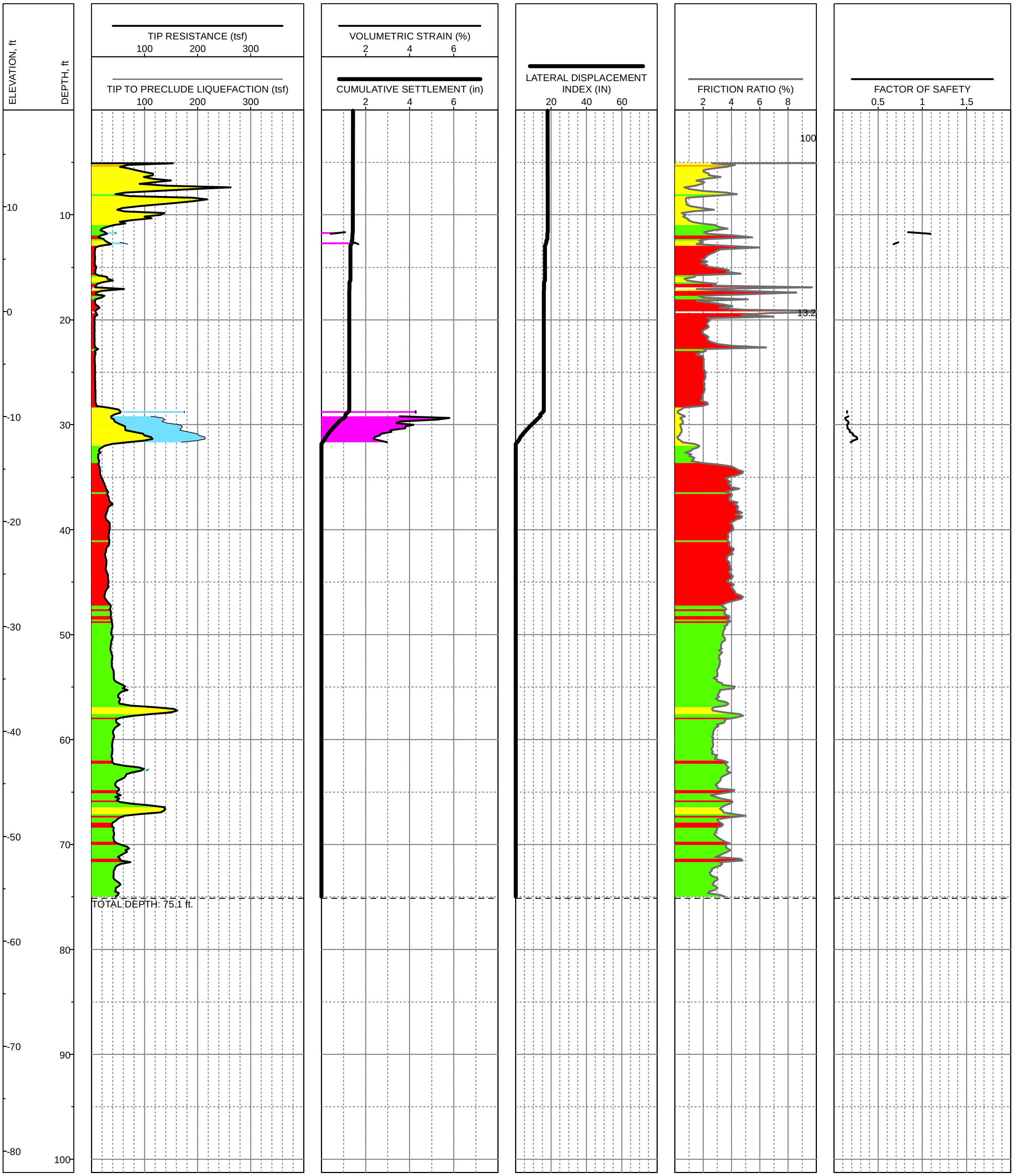
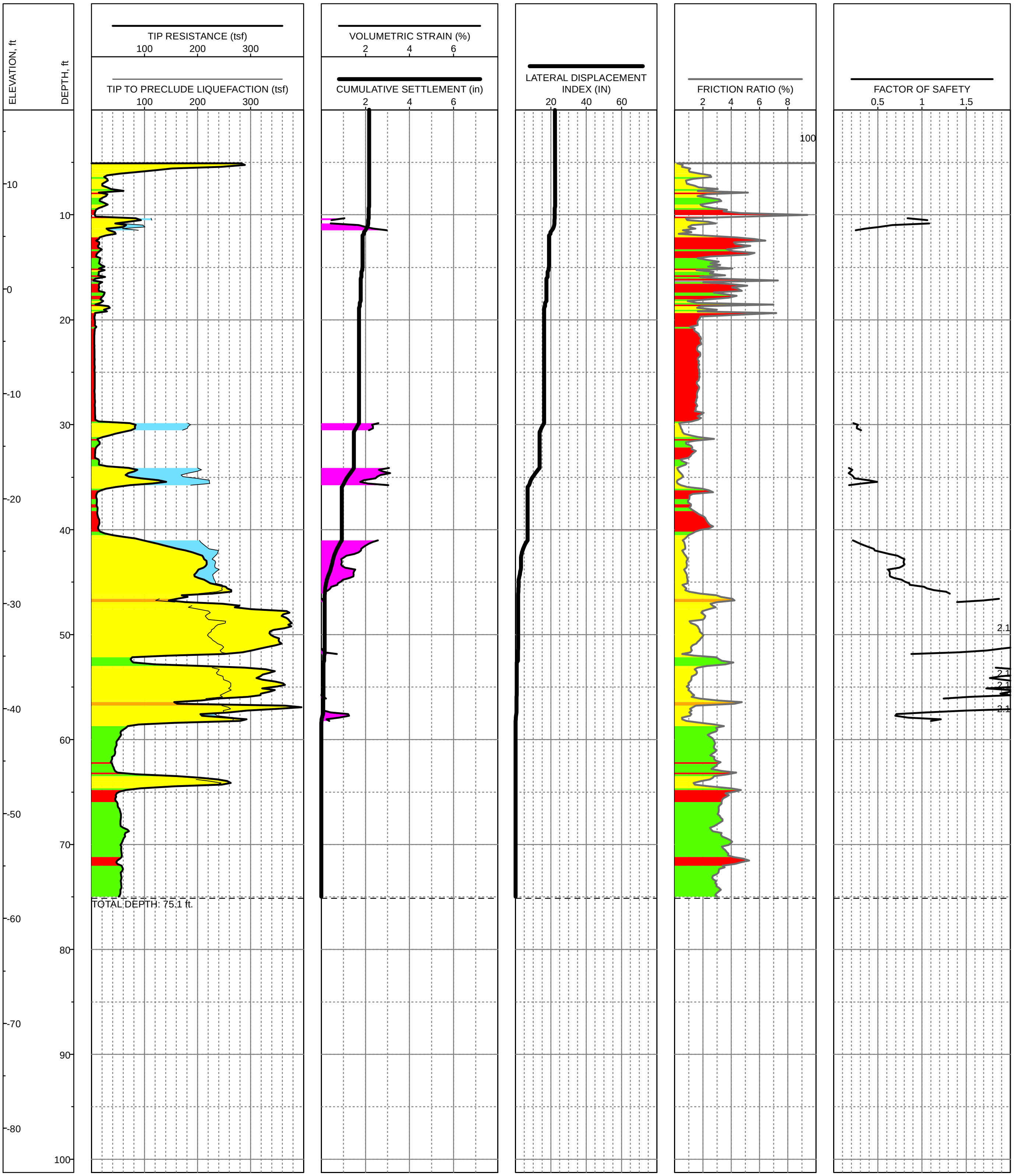


LOCATION: E6,052,490, N2,116,767, NAD83 SP CA Z3 FT
SURFACE EL: 19.2 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

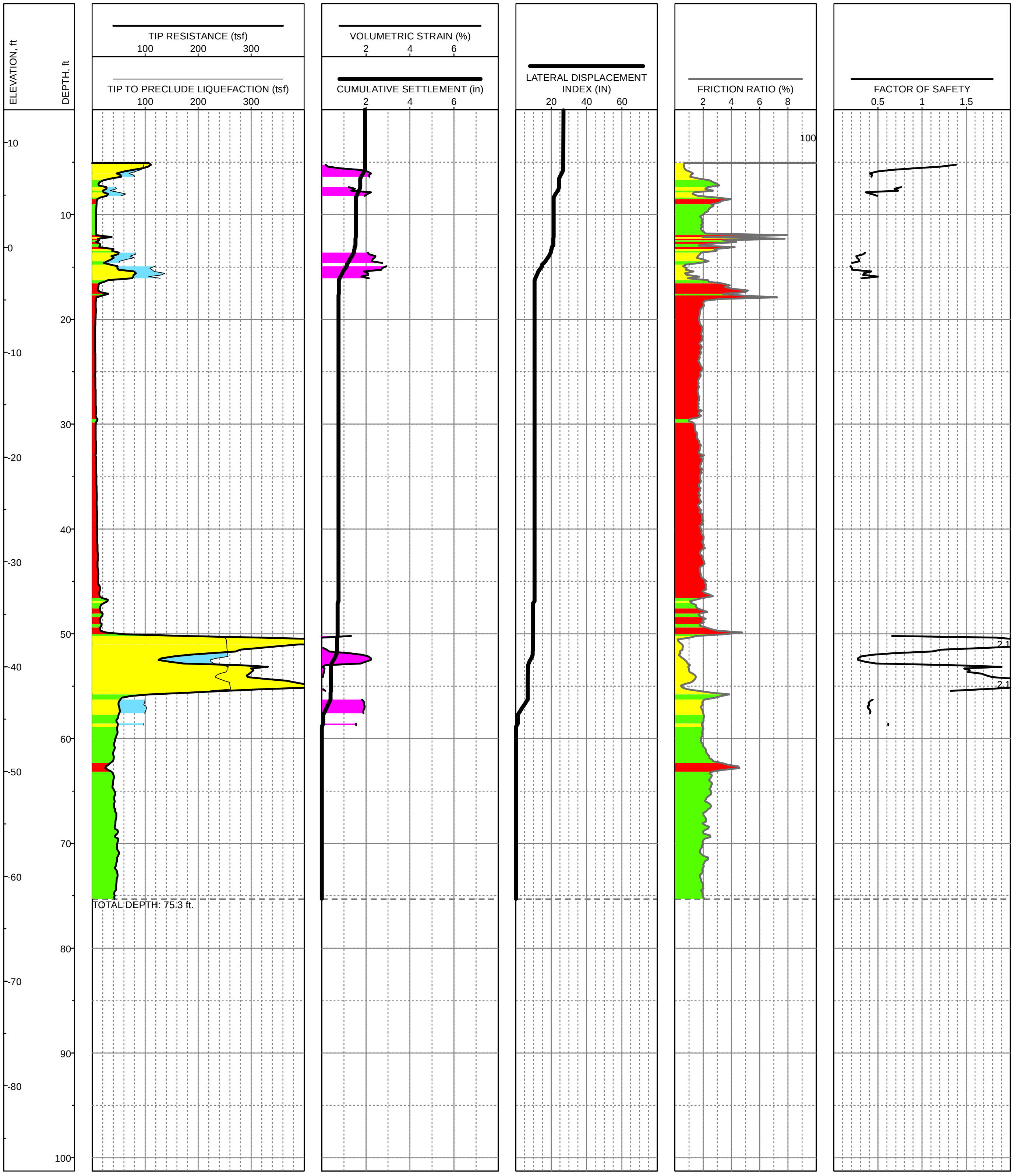
PLATE D-4: LOG OF 2020-CPT-04 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,557, N2,116,734, NAD83 SP CA Z3 FT
SURFACE EL: 17.1 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

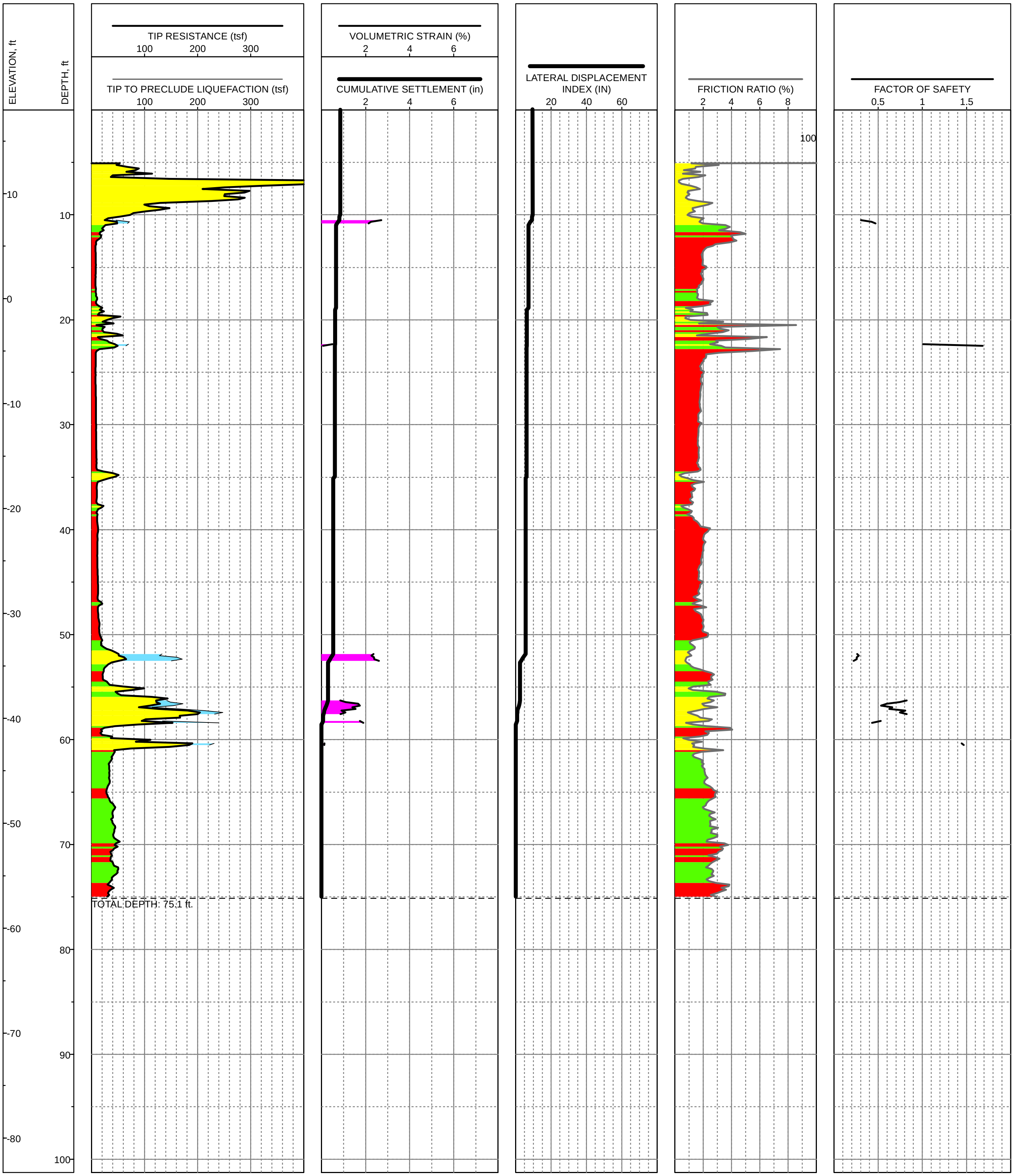
PLATE D-5: LOG OF 2020-CPT-05 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,632, N2,116,632, NAD83 SP CA Z3 FT
SURFACE EL: 13.1 ft
COMPLETION DEPTH: 75.3 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

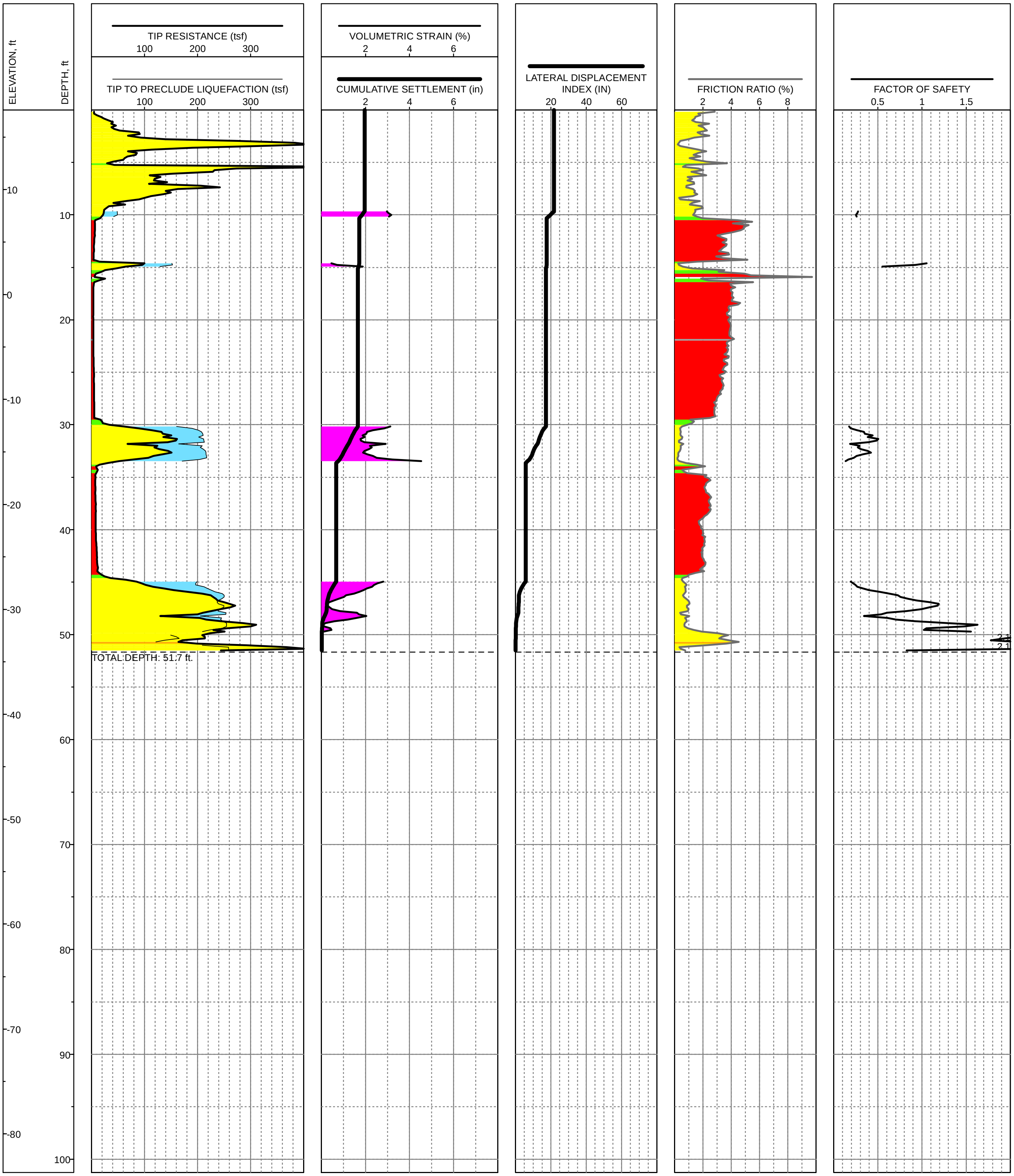
PLATE D-6: LOG OF 2020-CPT-06 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,572, N2,116,598, NAD83 SP CA Z3 FT
SURFACE EL: 18.0 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

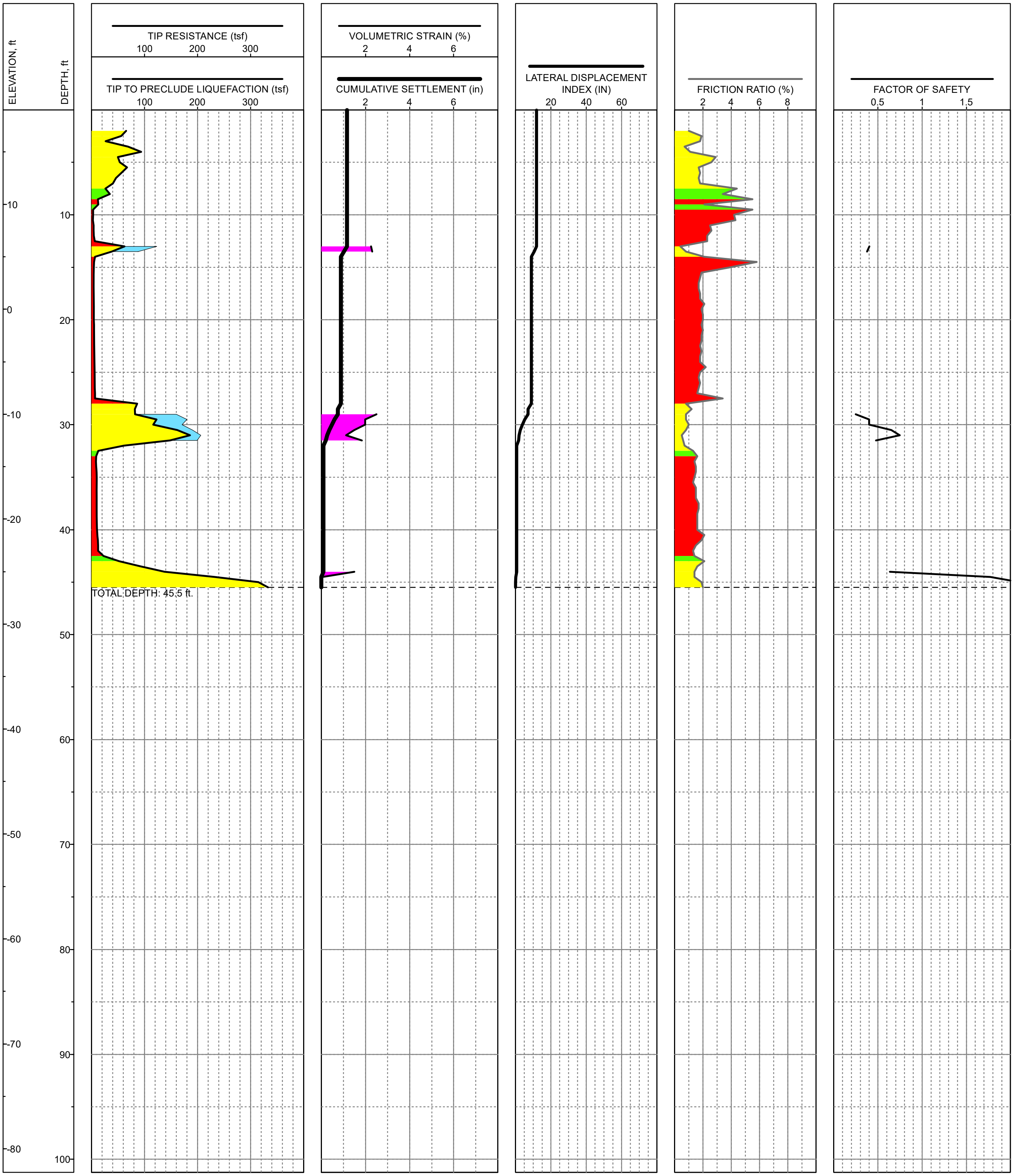
PLATE D-7: LOG OF 2020-CPT-07 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,485, N2,116,625, NAD83 SP CA Z3 FT
SURFACE EL: 17.6 ft
COMPLETION DEPTH: 51.7 ft
TESTDATE: 2.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-8: LOG OF 2020-CPT-08 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,497, N2,116,672, NAD83 SP CA Z3 FT
SURFACE EL: 19.00 ft
COMPLETION DEPTH: 45.5 ft
TESTDATE: 2/26/2002

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER DEPTH: 8 ft

PLATE D-9: LOG OF 2002-CPT-2 – M=7.0, PGA=0.810, N, TL, TR

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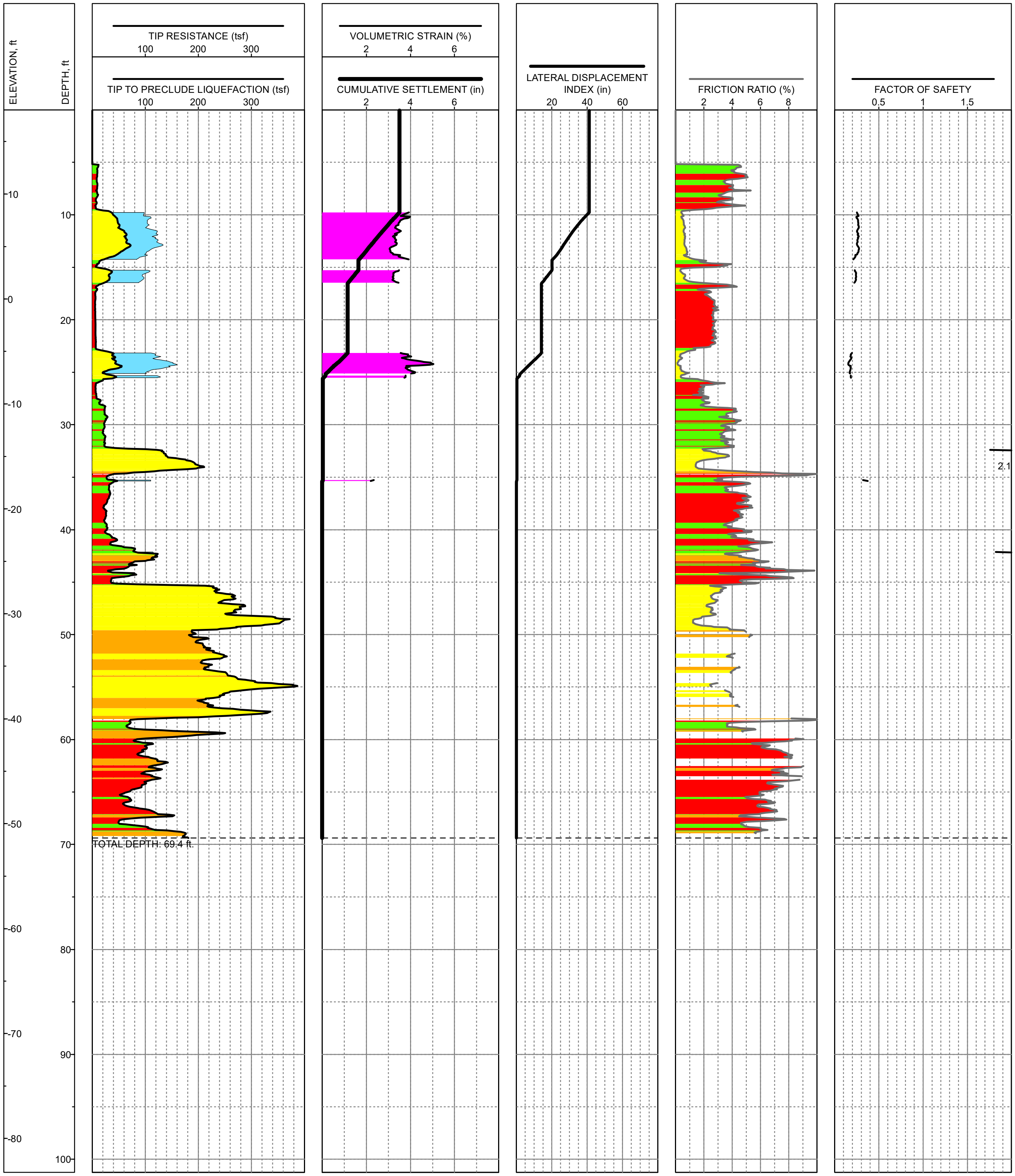
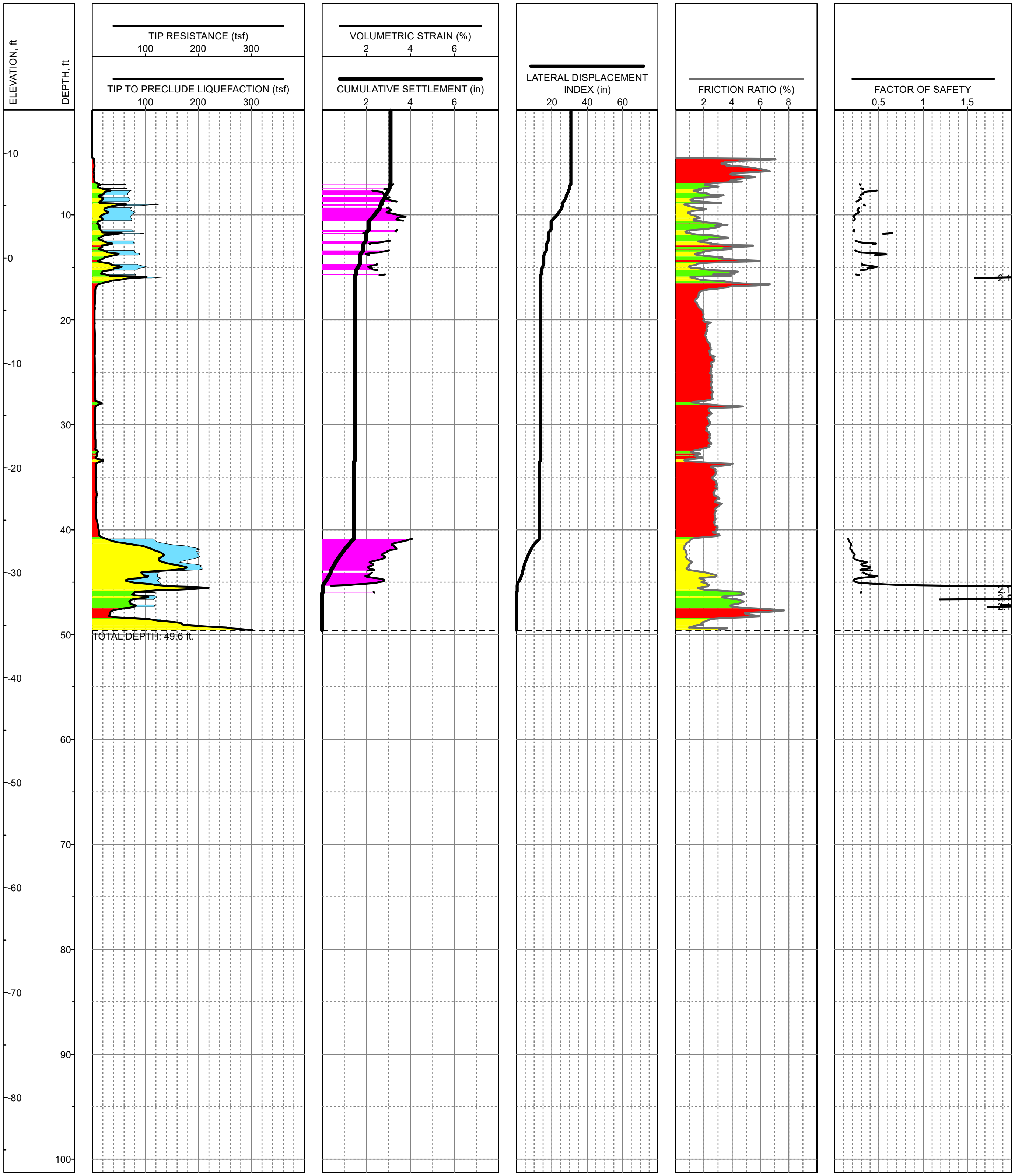


LOCATION: E6,052,365, N2,116,794, NAD83 SP CA Z3 FT
SURFACE EL: 18.02 ft
COMPLETION DEPTH: 69.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_B_IB_TL_TR\MXD\CPT_Logs_M7_0_a0_810_B_IB_TL_TR,6/21/2019,A.Ramirez

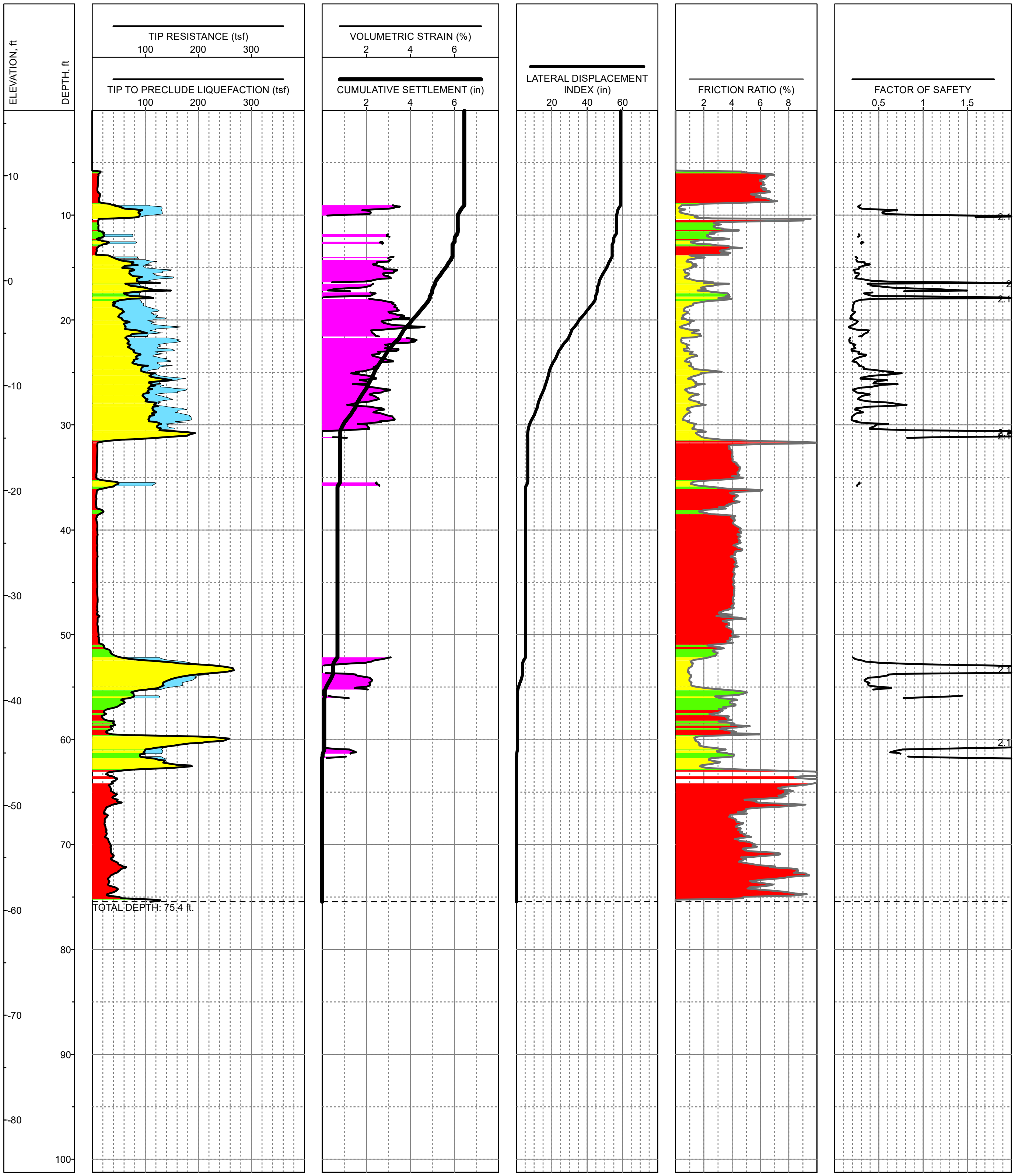
PLATE D-10: LOG OF 2019-CPT-01 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,593, N2,116,694, NAD83 SP CA Z3 FT
SURFACE EL: 14.11 ft
COMPLETION DEPTH: 49.6 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-11: LOG OF 2019-CPT-02 – M=7.0, PGA=0.810, B, TL, TR

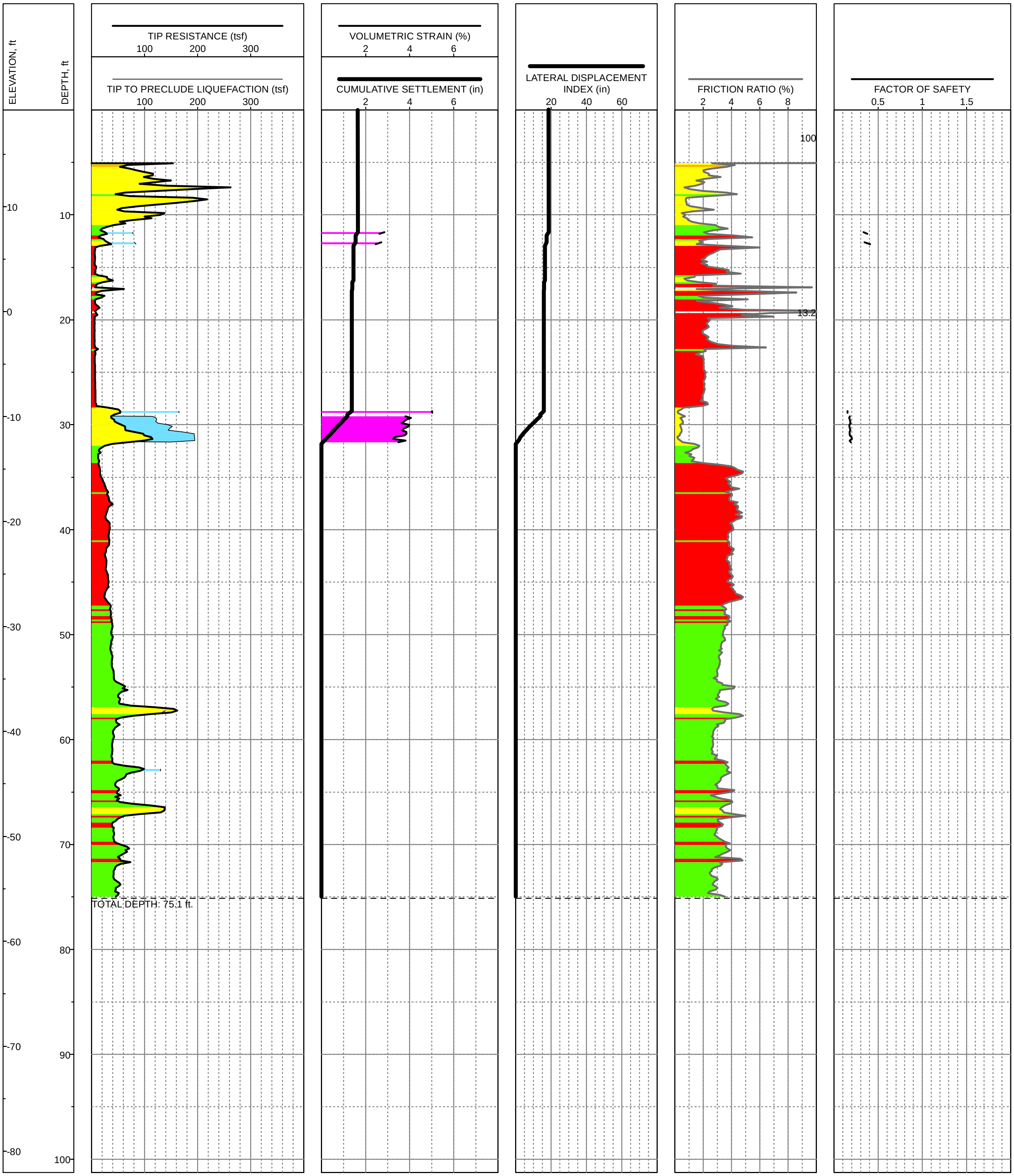


LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16.26 ft
COMPLETION DEPTH: 75.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-12: LOG OF 2019-CPT-03 – M=7.0, PGA=0.810, B, TL, TR

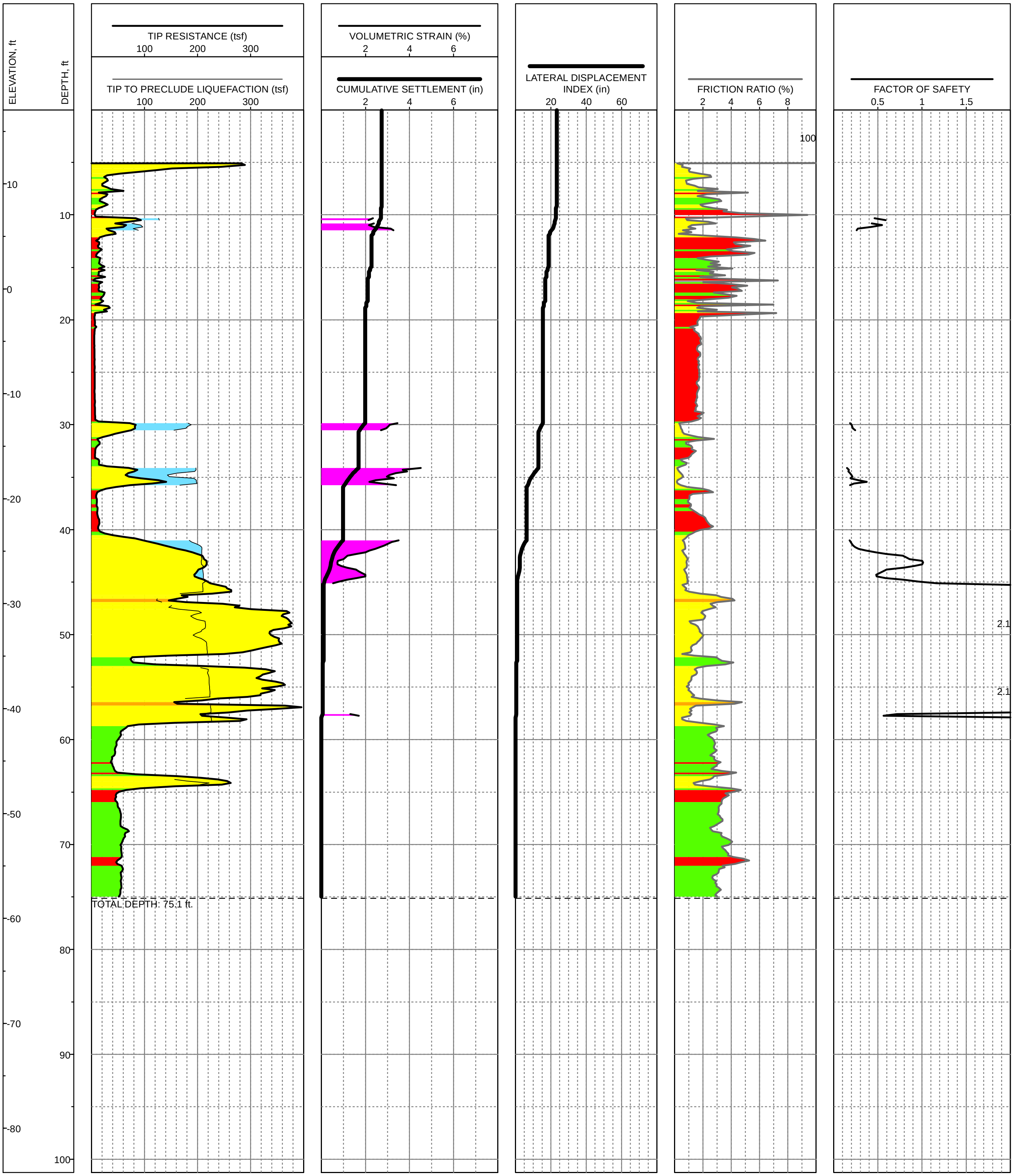
W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_B_IB_TL_TRIMXD\CPT_Logs_M7_0_a0_810_B_IB_TL_TR,6/21/2019,A.Ramirez



LOCATION: E6,052,490, N2,116,767, NAD83 SP CA Z3 FT
SURFACE EL: 19.2 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

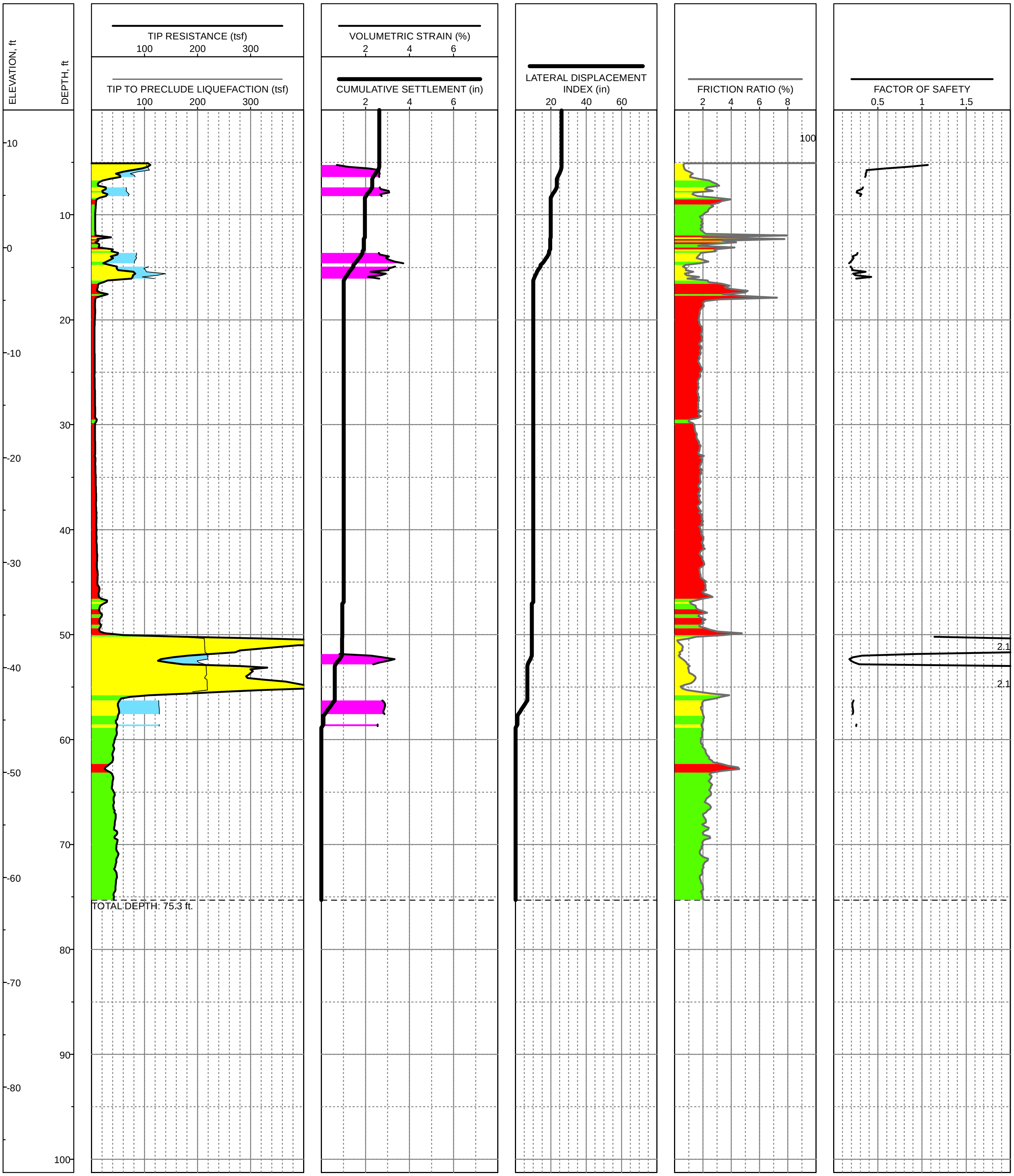
PLATE D-13: LOG OF 2020-CPT-04 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,557, N2,116,734, NAD83 SP CA Z3 FT
SURFACE EL: 17.1 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

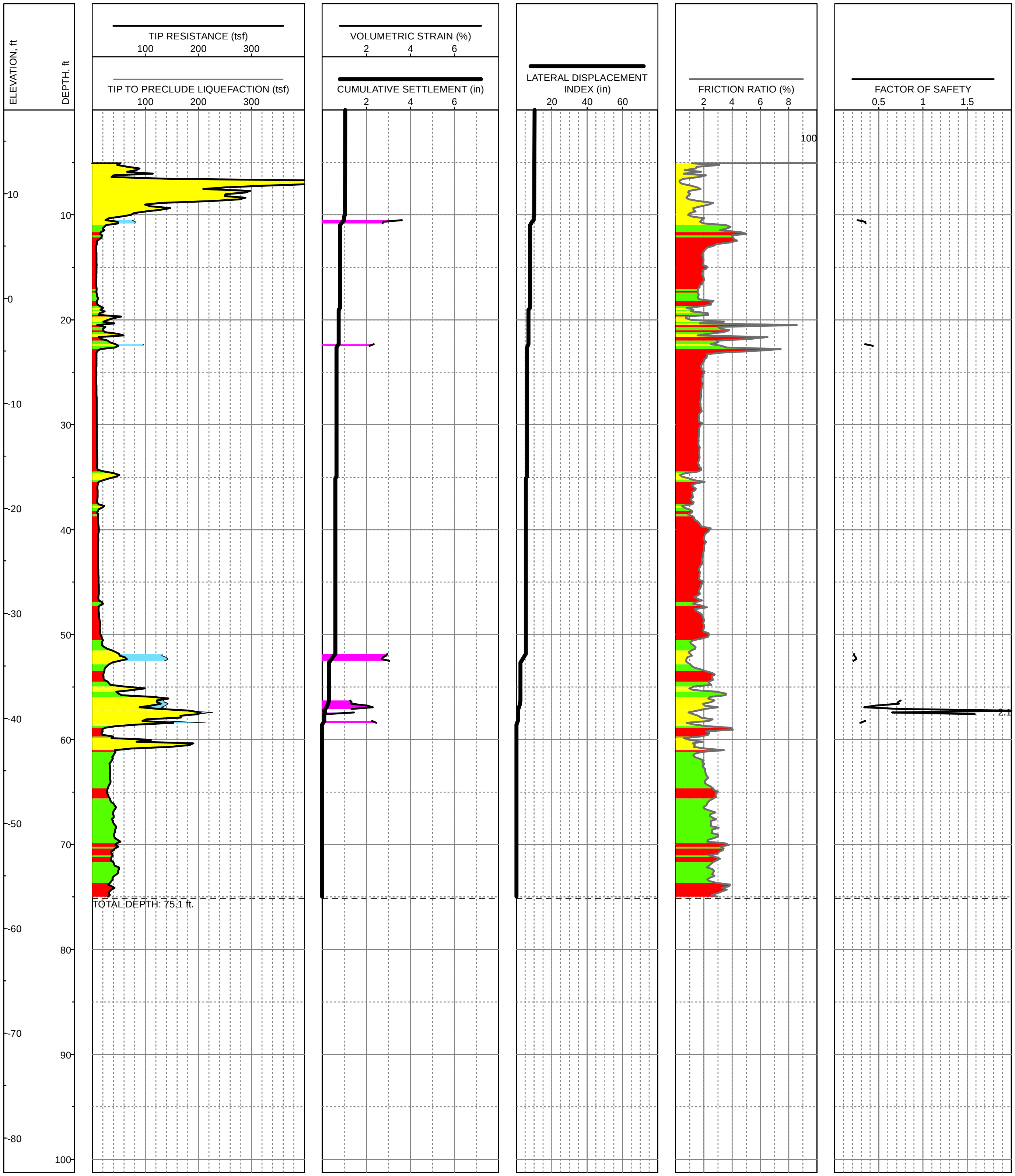
PLATE D-14: LOG OF 2020-CPT-05 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,632, N2,116,632, NAD83 SP CA Z3 FT
SURFACE EL: 13.1 ft
COMPLETION DEPTH: 75.3 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

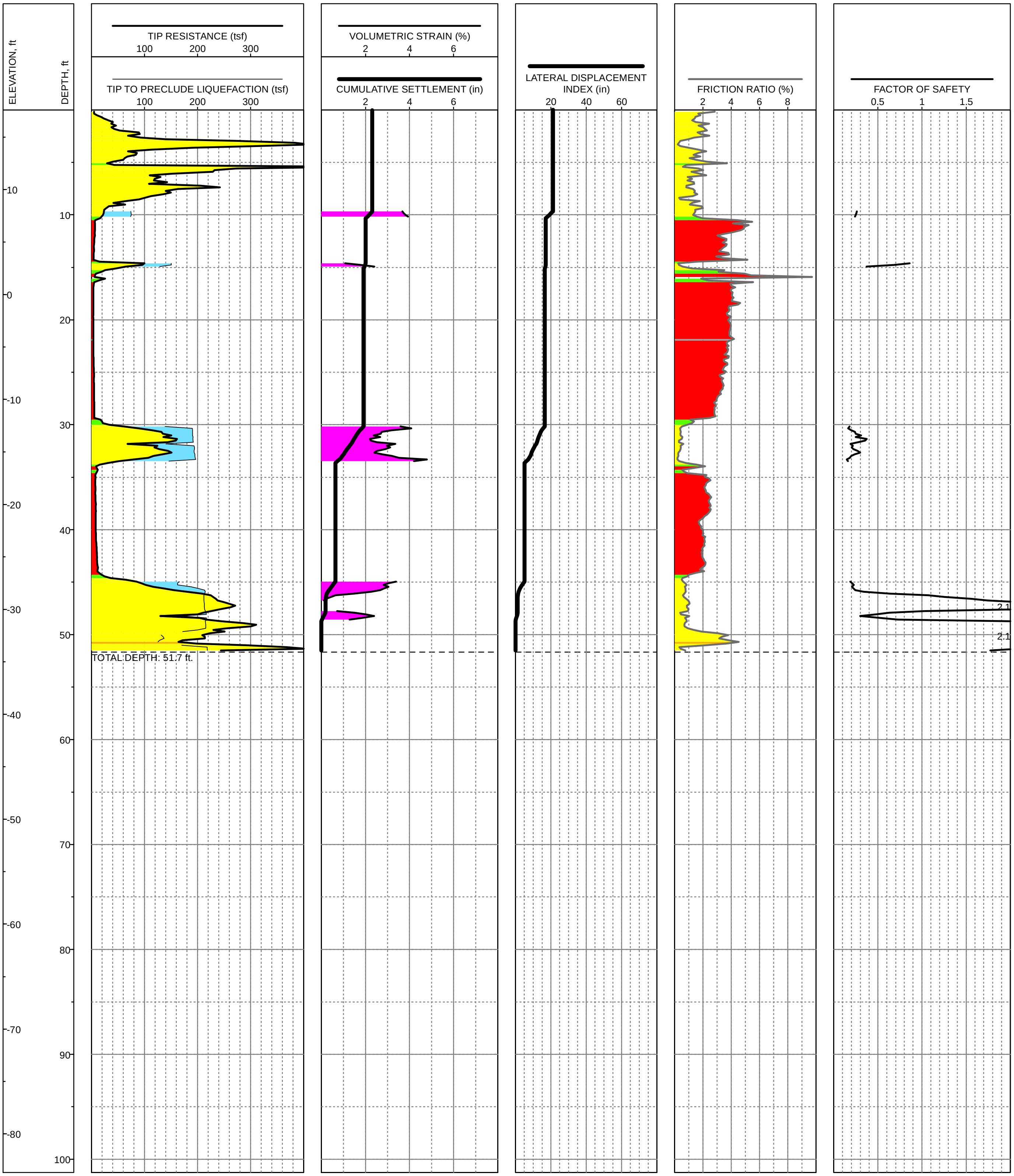
PLATE D-15: LOG OF 2020-CPT-06 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,572, N2,116,598, NAD83 SP CA Z3 FT
SURFACE EL: 18.0 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

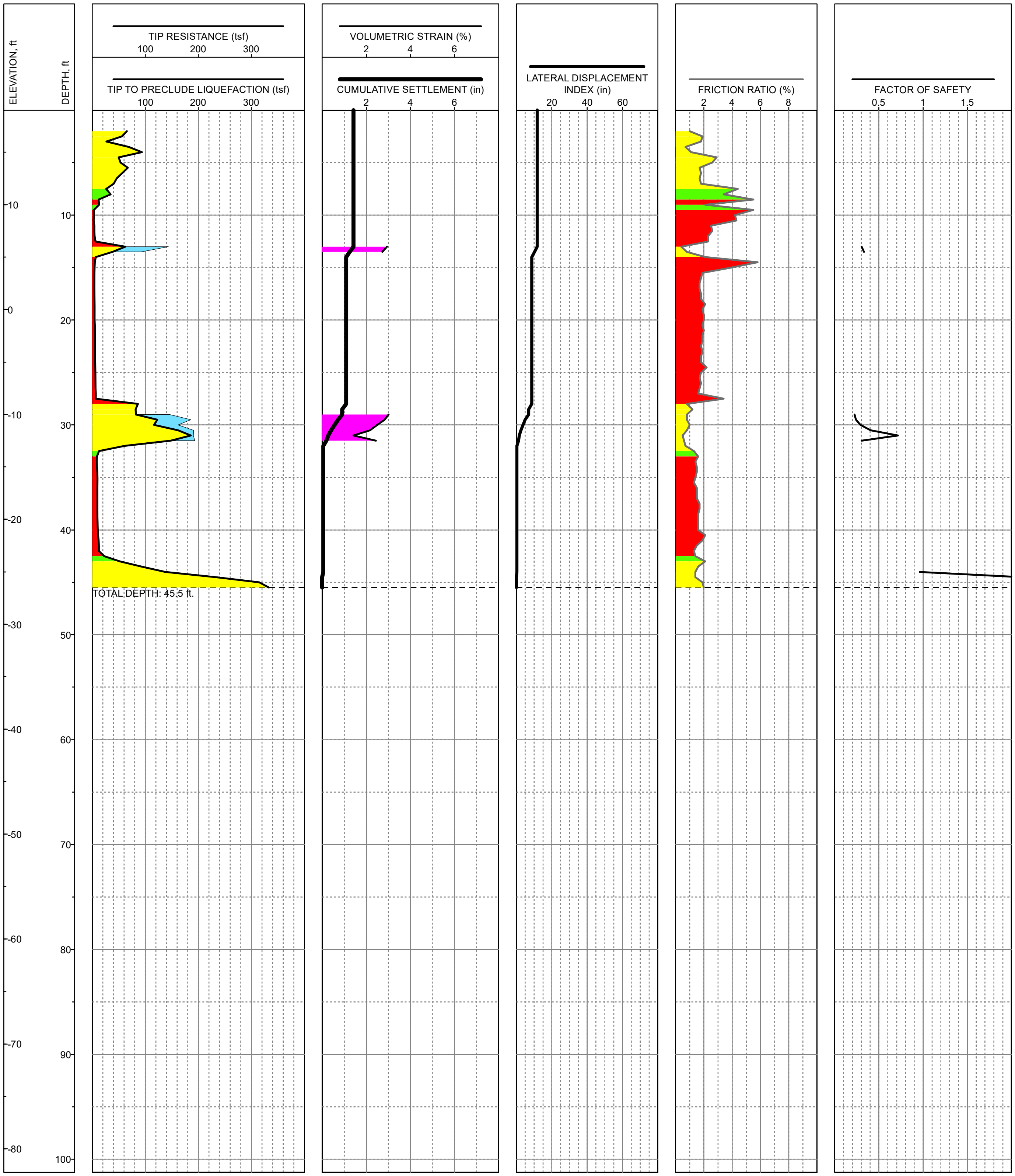
PLATE D-16: LOG OF 2020-CPT-07 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,485, N2,116,625, NAD83 SP CA Z3 FT
SURFACE EL: 17.6 ft
COMPLETION DEPTH: 51.7 ft
TESTDATE: 2.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-17: LOG OF 2020-CPT-08 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,497, N2,116,672, NAD83 SP CA Z3 FT
SURFACE EL: 19.00 ft
COMPLETION DEPTH: 45.5 ft
TESTDATE: 2/26/2002

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2002\Logs\2019_06_18_Logs_M7_0_a0_810_B_IB_TL_TR\MXD\CPT_Logs_M7_0_a0_810_B_IB_TL_TR,6/21/2019,A.Ramirez

PLATE D-18: LOG OF 2002-CPT-2 – M=7.0, PGA=0.810, B, TL, TR

FUGRO

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
1/28/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-B-01

Ground Elevation =		17.5	ft			
Depth to Ground Water Table =		9.5	ft	= EL	8	ft
γ =	110	pcf				
γ_{sat} =	120	pcf				
Boring Diameter =	4	inch =	101.6	mm		
Rod Length Above Ground =	3	ft =	0.9	m		

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1\,atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft											%	ft	in	
6.5	11.0	3.4	9	1,225.0	58.7	1,131.4	54.2	0.85	N	1.00	1	1.4	1.41	15	21	5	20	0.97	0.55	1.141	1.08	0.20	0.25	0.5	0.164	0.531	0.164	2.3	1.0	0.3	2.0
5	12.5	3.8	8	1,405.0	67.4	1,217.8	58.4	0.85	Y	1.10	1	1.4	1.36	14	21	5	19	0.96	0.58	1.141	1.07	0.19	0.24	0.4	0.179	0.574	0.179	2.4	2.5	0.7	5.4
1.5	16.0	4.9	16	1,825.0	87.5	1,419.4	68.0	0.95	N	1.00	1	1.4	1.21	26	6	0	26	0.95	0.64	1.141	1.07	0.31	0.37	0.6	0.081	0.188	0.081	1.8	3.0	0.7	2.9
-13.5	31.0	9.4	14	3,625.0	173.8	2,283.4	109.5	1.00	N	1.00	1	1.4	0.96	19	8	0	19	0.87	0.72	1.141	0.99	0.20	0.22	0.3	0.174	0.559	0.174	2.4	5.5	1.6	11.5
-33.5	51.0	15.5	24	6,025.0	288.8	3,435.4	164.7	1.00	N	1.00	1	1.4	0.82	28	16	4	31	0.76	0.70	1.141	0.90	0.56	0.58	0.8	0.040	-0.163	0.040	0.8	3.0	0.3	1.4
-37.5	55.0	16.8	96	6,505.0	311.8	3,665.8	175.7	1.00	N	1.00	1	1.4	0.86	116	17	4	120	0.74	0.69	1.141	0.84	2.00	1.91	2.0	0.000	-8.012	0.000	0.0	5.0	0.0	0.0
-47.5	65.0	19.8	64	7,705.0	369.4	4,241.8	203.4	1.00	N	1.00	1	1.4	0.83	75	19	4	79	0.69	0.66	1.141	0.79	2.00	1.81	2.0	0.000	-4.090	0.000	0.0	11.5	0.0	0.0
																											Total	2.0	15.0	3.5	23.2

PLATE D-19: BORING 2020-B-01

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
5/15/19 TC

a_{max} = 0.81 g ASCE 7-16
Mw = 7.0

2002-EB-1
Ground Elevation = 19.8 ft
Depth to Ground Water Table = 11.8 ft = EL 8 ft
γ = 110 pcf
γ_{sat} = 120 pcf
Boring Diameter = 8 inch = 203.2 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	Assumed FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1\ atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft											%	ft	in	
-14.7	34.5	10.5	6	4,022.0	192.8	2,605.5	124.9	1.00	N	1.00	1.15	1	0.88	6	30	5	11	0.85	0.69	1.141	0.98	0.13	0.14	0.2	0.404	0.879	0.404	3.4	5.0	2.1	24.2
-24.7	44.5	13.6	20	5,222.0	250.3	3,181.5	152.5	1.00	N	1.00	1.15	1	0.83	19	30	5	25	0.80	0.68	1.141	0.93	0.28	0.30	0.4	0.094	0.260	0.094	1.9	5.0	1.2	5.6
-29.2	49	14.9	100	5,762.0	276.2	3,440.7	164.9	1.00	N	1.00	1.15	1	0.88	101	30	5	106	0.77	0.68	1.141	0.86	2.00	1.95	2.0	0.000	-6.686	0.000	0.0	5.0	0.0	0.0
-34.7	54.5	16.6	37	6,422.0	307.9	3,757.5	180.1	1.00	Y	1.30	1.15	1	0.83	46	30	5	51	0.74	0.66	1.141	0.83	2.00	1.89	2.0	0.000	-1.704	0.000	0.0	5.0	0.0	0.0
-39.7	59.5	18.1	63	7,022.0	336.6	4,045.5	193.9	1.00	Y	1.30	1.15	1	0.84	79	30	5	85	0.72	0.65	1.141	0.81	2.00	1.84	2.0	0.000	-4.628	0.000	0.0	5.0	0.0	0.0
-44.7	64.5	19.7	38	7,622.0	365.4	4,333.5	207.7	1.00	N	1.00	1.15	1	0.80	35	30	5	40	0.69	0.64	1.141	0.79	2.00	1.80	2.0	0.008	-0.826	0.000	0.0	5.0	0.0	0.0
-49.7	69.5	21.2	42	8,222.0	394.2	4,621.5	221.6	1.00	N	1.00	1.15	1	0.80	39	30	5	44	0.67	0.62	1.141	0.77	2.00	1.75	2.0	0.003	-1.106	0.000	0.0	5.0	0.0	0.0
-54.7	74.5	22.7	46	8,822.0	422.9	4,909.5	235.4	1.00	N	1.00	1.15	1	0.80	42	30	5	48	0.65	0.61	1.141	0.75	2.00	1.71	2.0	0.001	-1.404	0.000	0.0	5.0	0.0	0.0
																											Total	2.7	10.0	3.2	29.9

PLATE D-20: BORING 2002-EB-1

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
5/15/19 TC

a_{max} = 0.81 g ASCE 7-16
Mw = 7.0

2002-EB-2
Ground Elevation = 18.2 ft
Depth to Ground Water Table = 10.2 ft = EL 8 ft
γ = 110 pcf
γ_{sat} = 120 pcf
Boring Diameter = 8 inch = 203.2 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	Assumed FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1 atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft											%	ft	in	
-16.3	34.5	10.5	37	4,038.0	193.6	2,521.7	120.9	1.00	Y	1.30	1.15	1	0.94	52	15	3	56	0.85	0.71	1.141	0.95	2.00	2.00	2.0	0.000	-2.044	0.000	0.0	5.0	0.0	0.0
-21.3	39.5	12.0	32	4,638.0	222.3	2,809.7	134.7	1.00	Y	1.30	1.15	1	0.90	43	15	3	47	0.83	0.71	1.141	0.92	2.00	2.00	2.0	0.002	-1.310	0.000	0.0	5.0	0.0	0.0
-26.3	44.5	13.6	32	5,238.0	251.1	3,097.7	148.5	1.00	Y	1.30	1.15	1	0.87	42	15	3	45	0.80	0.71	1.141	0.89	2.00	2.00	2.0	0.002	-1.195	0.000	0.0	5.0	0.0	0.0
-31.3	49.5	15.1	37	5,838.0	279.9	3,385.7	162.3	1.00	Y	1.30	1.15	1	0.86	48	15	3	51	0.77	0.69	1.141	0.86	2.00	1.96	2.0	0.000	-1.659	0.000	0.0	5.0	0.0	0.0
																											Total	0.0	0.0	0.0	0.0

PLATE D-21: BORING 2002-EB-2

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
5/15/19 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-EB-3

Ground Elevation =			19.2	ft			
Depth to Ground Water Table =			11.2	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	Assumed FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1 atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft											%	ft	in	
-10.3	29.5	9.0	11	3,428.0	164.3	2,286.1	109.6	1.00	N	1.00	1.15	1	0.96	12	15	3	15	0.88	0.69	1.141	0.99	0.16	0.18	0.3	0.264	0.739	0.264	2.8	1.0	0.3	3.2
-11.8	31.0	9.4	8	3,608.0	173.0	2,372.5	113.7	1.00	Y	1.10	1.15	1	0.93	9	15	3	13	0.87	0.69	1.141	0.99	0.14	0.16	0.2	0.352	0.839	0.352	3.2	5.0	1.9	21.1
-30.3	49.5	15.1	33	5,828.0	279.4	3,438.1	164.8	1.00	N	1.00	1.15	1	0.85	32	15	3	35	0.77	0.68	1.141	0.87	1.19	1.18	1.7	0.021	-0.461	0.006	0.1	5.0	0.1	0.4
																											Total	1.8	11.0	2.3	24.7

PLATE D-22: BORING 2002-EB-3

Appendix E

Dynamic Densification Analyses

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2019-CPT-01					
Ground Elevation =	18.0	ft			
Depth to Ground Water Table =	10.0	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.59				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 6 feet																
6.04	9.75	0.42	0.46	19,501.4	19,874.7	843.0	910.4	664.4	0.0	664.4	0.99	10.7	4.4	1.04	10.3	3.1	2.92E+05	442.9	0.1	16,357.3	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
6.10	9.46	0.44	0.43	18,925.0	19,279.6	880.0	864.8	671.0	0.0	671.0	0.99	10.8	4.7	1.02	11.1	3.0	3.06E+05	447.3	0.1	16,260.6	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
6.17	9.42	0.47	0.38	18,839.8	19,154.4	937.8	767.2	678.7	0.0	678.7	0.99	10.9	5.1	1.02	11.5	3.0	3.16E+05	452.5	0.1	16,149.6	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
6.23	9.55	0.48	0.35	19,098.6	19,382.2	956.6	691.8	685.3	0.0	685.3	0.99	11.1	5.1	1.03	11.5	3.0	3.26E+05	456.9	0.1	16,056.1	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
6.30	9.52	0.47	0.31	19,045.8	19,302.7	941.8	626.6	693.0	0.0	693.0	0.99	11.2	5.1	1.05	11.0	3.1	3.30E+05	462.0	0.1	15,948.8	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
6.36	9.27	0.47	0.30	18,537.4	18,786.4	939.2	607.2	699.6	0.0	699.6	0.99	11.3	5.2	1.06	10.6	3.1	3.32E+05	466.4	0.1	15,858.4	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.43	8.88	0.45	0.34	17,767.2	18,047.6	905.6	684.0	707.3	0.0	707.3	0.99	11.4	5.2	1.07	10.5	3.2	3.36E+05	471.5	0.1	15,754.6	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.49	8.62	0.42	0.36	17,241.4	17,539.5	845.2	727.0	713.9	0.0	713.9	0.98	11.5	5.0	1.05	10.9	3.1	3.43E+05	475.9	0.1	15,667.0	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.56	8.41	0.37	0.38	16,825.4	17,141.0	741.2	769.8	721.6	0.0	721.6	0.98	11.6	4.5	1.04	11.3	3.1	3.45E+05	481.1	0.1	15,566.5	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.63	8.31	0.32	0.44	16,610.0	16,966.9	640.2	870.6	729.3	0.0	729.3	0.98	11.7	3.9	1.03	11.1	3.1	3.39E+05	486.2	0.1	15,467.7	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.69	8.30	0.30	0.46	16,595.0	16,975.6	602.2	928.2	735.9	0.0	735.9	0.98	11.9	3.7	1.03	11.0	3.0	3.33E+05	490.6	0.1	15,384.3	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.76	8.52	0.30	0.51	17,030.0	17,446.6	590.2	1,016.2	743.6	0.0	743.6	0.98	12.0	3.5	1.02	10.9	3.0	3.34E+05	495.7	0.1	15,288.5	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
6.82	8.63	0.29	0.54	17,265.2	17,704.1	586.0	1,070.4	750.2	0.0	750.2	0.98	12.1	3.5	1.04	11.2	3.1	3.57E+05	500.1	0.1	15,207.7	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
6.89	9.00	0.31	0.52	17,995.2	18,422.9	625.8	1,043.2	757.9	0.0	757.9	0.98	12.2	3.5	0.99	13.5	2.9	3.88E+05	505.3	0.1	15,114.8	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
6.95	9.41	0.33	0.50	18,828.8	19,237.3	657.6	996.4	764.5	0.0	764.5	0.98	12.3	3.6	0.92	18.9	2.8	4.67E+05	509.7	0.1	15,036.3	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.02	9.84	0.35	0.49	19,684.4	20,085.9	700.2	979.2	772.2	0.0	772.2	0.98	12.4	3.6	0.87	24.1	2.6	5.38E+05	514.8	0.1	14,946.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.08	9.97	0.38	0.49	19,941.8	20,340.8	754.2	973.2	778.8	0.0	778.8	0.98	12.5	3.9	0.83	29.3	2.5	5.98E+05	519.2	0.1	14,870.1	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.15	9.80	0.40	0.54	19,603.8	20,048.1	795.2	1,083.6	786.5	0.0	786.5	0.98	12.7	4.1	0.83	30.7	2.5	6.20E+05	524.3	0.1	14,782.6	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
7.22	9.61	0.39	0.57	19,224.2	19,694.9	781.4	1,148.0	794.2	0.0	794.2	0.98	12.8	4.1	0.87	26.3	2.6	5.88E+05	529.5	0.1	14,696.4	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
7.28	9.48	0.38	0.60	18,960.6	19,455.5	760.4	1,207.0	800.8	0.0	800.8	0.98	12.9	4.1	0.91	22.2	2.7	5.55E+05	533.9	0.1	14,623.6	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
7.35	9.39	0.38	0.62	18,787.6	19,295.8	757.4	1,239.6	808.5	0.0	808.5	0.98	13.0	4.1	0.88	25.1	2.6	5.87E+05	539.0	0.1	14,539.9	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
7.41	9.18	0.35	0.67	18,367.0	18,917.1	694.8	1,341.6	815.1	0.0	815.1	0.98	13.1	3.8	0.85	27.6	2.6	6.13E+05	543.4	0.1	14,469.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
7.48	9.16	0.37	0.67	18,319.6	18,872.0	733.2	1,347.2	822.8	0.0	822.8	0.98	13.2	4.1	0.80	34.3	2.4	6.77E+05	548.5	0.1	14,387.7	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
7.54	9.10	0.37	0.68	18,192.6	18,747.2	745.4	1,352.8	829.4	0.0	829.4	0.98	13.3	4.2	0.76	41.2	2.3	7.48E+05	552.9	0.1	14,318.9	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
7.61	9.06	0.37	0.69	18,114.4	18,683.1	746.8	1,387.0	837.1	0.0	837.1	0.98	13.5	4.2	0.75	48.6	2.3	8.70E+05	558.1	0.1	14,239.7	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
7.68	7.03	0.37	0.74	14,052.6	14,655.6	747.0	1,470.8	844.8	0.0	844.8	0.98	13.6	5.4	0.69	61.3	2.1	9.37E+05	563.2	0.1	14,161.7	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
7.74	9.04	0.35	0.88	18,086.8	18,808.2	708.6	1,759.4	851.4	0.0	851.4	0.98	13.7	3.9	0.71	53.7	2.2	8.78E+05	567.6	0.1	14,095.8	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
7.81	8.81	0.33	0.99	17,621.8	18,430.4	650.2	1,972.2	859.1	0.0	859.1	0.98	13.8	3.7	0.76	43.2	2.3	7.95E+05	572.7	0.1	14,019.8	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
7.87	9.11	0.32	1.11	18,228.8	19,137.1	641.6	2,215.4	865.7	0.0	865.7	0.98	13.9	3.5	0.76	41.7	2.3	7.81E+05	577.1	0.1	13,955.6	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
7.94	9.41	0.33	1.15	18,812.2	19,751.1	652.6	2,290.0	873.4	0.0	873.4	0.98	14.0	3.5	0.80	41.5	2.4	8.50E+05	582.3	0.1	13,881.6	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
8.00	9.84	0.33	1.12	19,677.4	20,599.2	651.4	2,248.4	880.0	0.0	880.0	0.98	14.1	3.3	0.80	40.1	2.4	8.36E+05	586.7	0.1	13,819.1	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.001
8.07	11.12	0.35	0.83	22,230.0	22,913.6	690.0	1,667.2	887.7	0.0	887.7	0.98	14.3	3.1	0.83	35.7	2.5	7.98E+05	591.8	0.1	13,747.0	1.79E-05	0.0018	2.8	101.1	26.2	10.8	0.0013	0.0012	0.001
8.13	10.85	0.33	0.86	21,707.4	22,413.5	659.0	1,722.2	894.3	0.0	894.3	0.98	14.4	3.1	0.90	26.4	2.7	7.10E+05	596.2	0.1	13,686.1	2.02E-05	0.0021	4.0	106.4	30.4	10.8	0.0013	0.0011	0.001
8.20	10.28	0.32	0.92	20,555.0	21,306.1	637.4	1,832.0	902.0	0.0	902.0	0.98	14.5	3.1	0.90	25.9	2.7	6.93E+05	601.3	0.1	13,615.9	2.09E-05	0.0022	3.9	101.7	28.8	10.8	0.0014	0.0012	0.001
8.27	9.44	0.32	0.91	18,871.4	19,614.2	641.4	1,811.6	909.7	0.0	909.7	0.98	14.6	3.4	0.90	25.4	2.7	6.82E+05	606.5	0.1	13,546.6	2.14E-05	0.0022	3.9	99.2	28.1	10.8	0.0015	0.0013	0.001
8.33	8.27	0.31	0.98	16,536.4	17,341.1	629.6	1,962.8	916.3	0.0	916.3	0.98	14.7	3.8	0.83	31.8	2.5	7.34E+05	610.9	0.1	13,488.0	2.00E-05	0.0021	2.9	91.3	23.7	10.8	0.0017	0.0015	0.001
8.40	7.56	0.31	1.07	15,126.0	16,002.1	610.8	2,136.8	924.0	0.0	924.0	0.98	14.8	4.1	0.77	34.3	2.4	6.89E+05	616.0	0.1	13,420.4	2.15E-05	0.0022	2.2	75.0	18.2	10.8	0.0025	0.0022	0.002
8.46	7.26	0.29	1.09	14,522.8	15,419.1	586.6	2,186.0	930.6	0.0	930.6	0.98	14.9	4.0	0.76	36.4	2.3	7.08E+05	620.4	0.1	13,363.2	2.11E-05	0.0022	2.0	74.3	17.7	10.8	0.0025	0.0022	0.002
8.53	6.86	0.28	1.18	13,716.2	14,679.7	553.6	2,350.0	938.3	0.0	938.3	0.98	15.0	4.0	0.75	33.9	2.3	6.52E+05	625.5	0.1	13,297.3	2.31E-05	0.0024	2.0	67.1	15.8	10.8	0.0032	0.0028	0.002
8.59	6.70	0.25	1.26	13,399.0	14,435.6	496.2	2,528.4	944.9	0.0	944.9	0.98	15.2	3.7	0.76	31.0	2.3	6.18E+05	629.9	0.1	13,241.5	2.45E-05	0.0026	2.1	64.8	15.5	10.8	0.0035	0.0030	0.002
8.66	6.79	0.22	1.31	13,584.4	14,661.8	435.0	2,627.8	952.6	0.0	952.6	0.98	15.3	3.2	0.79	28.7	2.4	6.07E+05	635.1	0.1	13,177.2	2.52E-05	0.0026	2.3	66.0	16.2	10.8	0.0034	0.0029	0.002
8.72	7.43	0.22	1.41	14,852.8	16,011.3	448.2	2,825.6	959.2	0.0	959.2	0.98	15.4	3.0	0.85	25.1	2.6	6.30E+05	639.5	0.1	13,122.7	2.44E-05	0.0025	3.1	78.7	20.9	10.8	0.0024	0.0021	0.002
8.79	8.36	0.24	1.38	16,714.6	17,847.5	488.6	2,763.2	966.9	0.0	966.9	0.98	15.5	2.9	0.89	23.4	2.7	6.44E+05	644.6	0.1	13,059.9	2.41E-05	0.0025	3.7	86.5	24.1	10.8	0.0020	0.0017	0.001
8.86	8.49	0.29	1.26	16,988.8	18,023.0	583.4	2,522.4	974.6	0.0	974.6	0.98	15.6	3.4	0.90	25.5	2.7	7.39E+05	649.7	0.1	12,997.9	2.11E-05	0.0022	4.0	102.3	29.2	10.8	0.0014	0.0012	0.001
8.92	8.39	0.29	0.97	16,778.0	17,570.9	588.0	1,933.8	981.2	0.0	981.2	0.98	15.7	3.5	0.67	62.0	2.1	1.00E+06	654.1	0.1	12,945.4	1.57E-05	0.0016	1.4	89.7	19.4	10.8	0.0017	0.0014	0.001
8.99	7.23	0.29	0.68	14,450.4	15,005.3	583.6	1,353.4	988.9	0.0	988.9	0.98	15.8	4.2	0.56	92.8	1.8	1.14E+06	659.3	0.1	12,884.8	1.38E-05	0.0014	1.1	103.7	20.1	10.8	0.0014	0.0012	0.001
9.05	6.48	0.28	0.59	12,963.2	13,446.5	561.0	1,178.8	995.5	0.0	995.5	0.98	15.9	4.5	0.59	80.3	1.9	1.07E+06	663.7	0.1	12,833.5	1.49E-05	0.0015	1.2	94.8	18.9	10.8	0.0016	0.0014	0.001
9.12	5.77	0.28	0.47	11,546.2	11,934.0	569.6	945.8	1,003.2	0.0	1,003.2	0.98	16.1	5.2	0.63	67.8	2.0	1.00E+06	668.8	0.1	12,774.3	1.60E-05	0.0016	1.3	87.5	18.2	10.8	0.0018	0.0016	0.001
9.18	6.38	0.26	0.57	12,750.6	13,216.7	524.0	1,136.8	1,009.8	0.0	1,009.8	0.98	16.2	4.3	0.69	55.3	2.1	9.46E+05	673.2	0.1	12,724.1	1.71E-05	0.0018	1.5	84.2	18.5	10.8	0.0019	0.0017	0.001
9.25	6.61	0.23	0.72	13,223.0	13,816.0	460.8	1,446.4	1,017.5	0.0	1,017.5	0.98	16.3	3.6	0.72	48.8	2.2	9.04E+05	678.3	0.1	12,666.3	1.80E-05	0.0019	1.7	82.7	18.7	10.8	0.0020	0.0017	0.001
9.32	6.65	0.19	0.71	13,298.0	13,880.4	383.6	1,420.4	1,025.2	0.0	1,025.2	0.98	16.4	3.0	0.76	42.5	2.3	8.76E+05	683.5	0.1	12,609.1	1.87E-05	0.0019	2.0	84.7	20.1	10.8	0.0019	0.0017	0.001
9.38	6.50	0.17	0.62	13,001.2	13,508.9	339.8	1,238.2	1,031.8	0.0	1,031.8	0.98	16.5	2.7	0.79	38.8	2.4	8.87E+05	687.9	0.1	12,560.6	1.86E-05	0.0019	2.4	91.6	22.7	10.8	0.0017	0.0014	0.001
9.45	8.44	0.18	0.67	16,881.4	17,427.4	359.6	1,331.8	1,039.5	0.0	1,039.5	0.98	16.6	2.2	0.81	38.0	2.5	9.13E+05	693.0	0.1	12,504.7	1.82E-05	0.0019	2.6	97.2	24.5	10.8	0.0015	0.0013	0.001
9.51	13.91	0.18	0.78	27,812.6	28,449.7	358.2	1,554.0	1,046.1	0.0	1,046.1	0.98	16.7	1.3	0.80	36.7	2.4	8.69E+05	697.4	0.1	12,457.3	1.93E-05	0.0020	2.5	90.7	22.7	10.8	0.0017	0.0015	0.001
9.58	21.73	0.17	0.80	43,453.0	44,108.3	334.2	1,598.2	1,053.8	0.0	1,053.8	0.98	16.9	0.8	0.77	37.0	2.4	8.08E+05	702.5	0.1	12,402.6	2.09E-05	0.0022	2.1	78.6	18.9	10.8	0.0023	0.0020	0.002
9.64	30.54	0.14	0.58	61,077.0	61,554.5	286.4	1,164.6	1,060.4	0.0	1,060.4	0.98	17.0	0.5	0.74	40.0	2.3	8.15E+05	706.9	0.1	12,356.3	2.08E-05	0.0022	1.9	75.1	17.5	10.8	0.0025	0.0022	0.002

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
9.71	34.78	0.14	0.39	69,551.4	69,869.4	279.6	775.6	1,068.1	0.0	1,068.1	0.98	17.1	0.4	0.71	45.1	2.2	8.41E+05	712.1	0.1	12,302.7	2.03E-05	0.0021	1.6	73.3	16.4	10.8	0.0027	0.0023	0.002
9.77	36.08	0.15	0.32	72,156.2	72,422.3	291.2	649.0	1,074.7	0.0	1,074.7	0.98	17.2	0.4	0.70	45.9	2.2	8.37E+05	716.5	0.1	12,257.3	2.05E-05	0.0021	1.6	72.0	15.9	10.8	0.0028	0.0024	0.002
9.84	37.82	0.16	0.24	75,636.2	75,829.7	328.0	472.0	1,082.4	0.0	1,082.4	0.98	17.3	0.4	0.71	40.9	2.2	7.81E+05	721.6	0.1	12,205.0	2.22E-05	0.0023	1.7	67.9	15.3	10.8	0.0032	0.0027	0.002
9.91	38.66	0.18	0.17	77,325.4	77,461.7	362.8	332.4	1,090.1	0.0	1,090.1	0.98	17.4	0.5	0.75	34.9	2.3	7.35E+05	726.7	0.1	12,153.2	2.37E-05	0.0025	1.9	66.9	15.7	10.8	0.0033	0.0029	0.002
9.97	39.27	0.20	0.13	78,543.6	78,649.9	391.0	259.2	1,096.7	0.0	1,096.7	0.98	17.5	0.5	0.77	30.5	2.4	6.90E+05	731.1	0.1	12,109.2	2.54E-05	0.0026	2.1	65.3	15.7	10.8	0.0035	0.0031	0.000
																										Total Estimated Settlement		2 x $\Sigma\Delta s$	0.2

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2019-CPT-02					
Ground Elevation =	14.1	ft			
Depth to Ground Water Table =	6.1	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.59				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in

Hand Auger from 0 to 6 feet - Ground Water Table is at 5 feet below ground surface

Total Estimated Settlement 2 x $\Sigma\Delta s$ **0.0**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} = 0.81 g ASCE 7-16
Mw = 7.0

2019-CPT-03
Ground Elevation = 16.3 ft
Depth to Ground Water Table = 8.3 ft = EL 8 ft
γ = 110 pcf
γ_{sat} = 120 pcf
Atmospheric pressure 2,116.2 psf
Cone Area Ratio 0.59

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 5 feet																
5.84	15.47	0.73	0.22	30,930.0	31,112.3	1,450.2	444.6	642.4	0.0	642.4	0.99	10.4	4.8	0.88	41.0	2.7	7.92E+05	428.3	0.1	16,691.1	1.31E-05	0.0013	3.7	150.9	42.0	10.8	0.0006	0.0005	0.000
5.90	15.85	0.74	0.23	31,702.8	31,892.9	1,485.6	463.6	649.0	0.0	649.0	0.99	10.5	4.8	0.88	41.5	2.7	8.07E+05	432.7	0.1	16,589.1	1.30E-05	0.0013	3.7	151.8	42.1	10.8	0.0005	0.0005	0.000
5.97	14.51	0.75	0.02	29,012.8	29,025.9	1,507.2	32.0	656.7	0.0	656.7	0.99	10.6	5.3	0.90	38.3	2.7	7.90E+05	437.8	0.1	16,472.1	1.34E-05	0.0014	4.1	155.9	44.6	10.8	0.0005	0.0005	0.000
6.04	13.42	0.78	0.07	26,840.4	26,895.8	1,563.0	135.2	664.4	0.0	664.4	0.99	10.7	6.0	0.92	36.0	2.8	7.83E+05	442.9	0.1	16,357.3	1.37E-05	0.0014	4.5	161.6	47.6	10.8	0.0005	0.0004	0.000
6.10	11.61	0.79	-0.09	23,226.8	23,151.6	1,578.4	-183.4	671.0	0.0	671.0	0.99	10.8	7.0	0.95	31.7	2.9	7.51E+05	447.3	0.1	16,260.6	1.44E-05	0.0015	5.3	167.0	51.8	10.8	0.0005	0.0004	0.000
6.17	11.34	0.79	-0.06	22,676.8	22,626.9	1,578.6	-121.8	678.7	0.0	678.7	0.99	10.9	7.2	0.96	30.9	2.9	7.48E+05	452.5	0.1	16,149.6	1.46E-05	0.0015	5.4	167.0	52.2	10.8	0.0005	0.0004	0.000
6.23	12.02	0.77	0.07	24,038.4	24,093.8	1,542.0	135.0	685.3	0.0	685.3	0.99	11.1	6.6	0.94	32.1	2.8	7.59E+05	456.9	0.1	16,056.1	1.46E-05	0.0015	5.1	162.1	49.6	10.8	0.0005	0.0004	0.000
6.30	11.37	0.75	0.12	22,735.2	22,833.3	1,492.8	239.2	693.0	0.0	693.0	0.99	11.2	6.7	0.95	30.3	2.9	7.40E+05	462.0	0.1	15,948.8	1.51E-05	0.0016	5.3	159.9	49.6	10.8	0.0005	0.0005	0.000
6.36	11.13	0.72	0.16	22,255.2	22,383.7	1,439.4	313.4	699.6	0.0	699.6	0.99	11.3	6.6	0.96	29.5	2.9	7.28E+05	466.4	0.1	15,858.4	1.55E-05	0.0016	5.3	156.4	48.6	10.8	0.0006	0.0005	0.000
6.43	10.99	0.70	0.25	21,980.6	22,184.3	1,404.4	496.8	707.3	0.0	707.3	0.99	11.4	6.5	0.96	28.9	2.9	7.23E+05	471.5	0.1	15,754.6	1.58E-05	0.0016	5.3	153.8	47.8	10.8	0.0006	0.0005	0.000
6.49	11.04	0.71	0.22	22,079.2	22,261.2	1,415.8	444.0	713.9	0.0	713.9	0.98	11.5	6.6	0.96	28.8	2.9	7.28E+05	475.9	0.1	15,667.0	1.58E-05	0.0016	5.3	153.9	47.9	10.8	0.0006	0.0005	0.000
6.56	11.04	0.69	0.31	22,075.2	22,328.8	1,381.6	618.6	721.6	0.0	721.6	0.98	11.6	6.4	0.96	28.5	2.9	7.25E+05	481.1	0.1	15,566.5	1.60E-05	0.0017	5.3	151.0	46.9	10.8	0.0006	0.0005	0.000
6.63	11.19	0.68	0.39	22,380.0	22,696.5	1,360.0	772.0	729.3	0.0	729.3	0.98	11.7	6.2	0.95	28.6	2.8	7.27E+05	486.2	0.1	15,467.7	1.62E-05	0.0017	5.2	148.5	45.8	10.8	0.0006	0.0005	0.000
6.69	11.25	0.67	0.37	22,498.4	22,804.8	1,339.4	747.4	735.9	0.0	735.9	0.98	11.9	6.1	0.95	28.5	2.8	7.26E+05	490.6	0.1	15,384.3	1.63E-05	0.0017	5.2	146.6	45.1	10.8	0.0006	0.0005	0.000
6.76	11.10	0.66	0.41	22,193.6	22,530.6	1,329.4	822.0	743.6	0.0	743.6	0.98	12.0	6.1	0.95	27.9	2.8	7.24E+05	495.7	0.1	15,288.5	1.65E-05	0.0017	5.2	145.7	45.0	10.8	0.0006	0.0006	0.000
6.82	10.83	0.67	0.26	21,660.8	21,873.0	1,340.0	517.6	750.2	0.0	750.2	0.98	12.1	6.3	0.96	27.1	2.9	7.21E+05	500.1	0.1	15,207.7	1.68E-05	0.0017	5.4	146.6	45.9	10.8	0.0006	0.0006	0.000
6.89	10.68	0.67	0.42	21,354.6	21,698.4	1,331.0	838.6	757.9	0.0	757.9	0.98	12.2	6.4	0.96	26.6	2.9	7.20E+05	505.3	0.1	15,114.8	1.70E-05	0.0018	5.5	145.7	45.7	10.8	0.0006	0.0006	0.000
6.95	10.50	0.65	0.23	20,997.2	21,187.0	1,303.0	463.0	764.5	0.0	764.5	0.98	12.3	6.4	0.97	25.9	2.9	7.11E+05	509.7	0.1	15,036.3	1.73E-05	0.0018	5.6	144.0	45.5	10.8	0.0007	0.0006	0.000
7.02	10.46	0.64	0.19	20,912.6	21,065.8	1,289.0	373.6	772.2	0.0	772.2	0.98	12.4	6.4	0.97	25.5	2.9	7.10E+05	514.8	0.1	14,946.2	1.75E-05	0.0018	5.6	142.7	45.2	10.8	0.0007	0.0006	0.000
7.08	10.29	0.65	0.27	20,582.0	20,803.4	1,295.8	540.0	778.8	0.0	778.8	0.98	12.5	6.5	0.97	25.0	2.9	7.10E+05	519.2	0.1	14,870.1	1.77E-05	0.0018	5.7	142.9	45.5	10.8	0.0007	0.0006	0.000
7.15	10.30	0.63	0.21	20,606.4	20,781.2	1,263.2	426.4	786.5	0.0	786.5	0.98	12.7	6.3	0.97	24.7	2.9	7.06E+05	524.3	0.1	14,782.6	1.79E-05	0.0019	5.7	140.3	44.6	10.8	0.0007	0.0006	0.001
7.22	10.52	0.63	0.22	21,042.0	21,221.9	1,267.2	438.8	794.2	0.0	794.2	0.98	12.8	6.2	0.97	25.0	2.9	7.13E+05	529.5	0.1	14,696.4	1.79E-05	0.0019	5.6	139.6	44.2	10.8	0.0007	0.0006	0.000
7.28	10.42	0.63	0.43	20,842.4	21,192.7	1,256.4	854.4	800.8	0.0	800.8	0.98	12.9	6.2	0.97	24.7	2.9	7.13E+05	533.9	0.1	14,623.6	1.81E-05	0.0019	5.6	138.5	43.8	10.8	0.0007	0.0006	0.000
7.35	10.32	0.62	0.32	20,631.6	20,895.5	1,245.8	643.6	808.5	0.0	808.5	0.98	13.0	6.2	0.97	24.2	2.9	7.10E+05	539.0	0.1	14,539.9	1.83E-05	0.0019	5.7	137.6	43.8	10.8	0.0007	0.0006	0.000
7.41	9.85	0.62	0.41	19,705.8	20,040.3	1,235.0	815.8	815.1	0.0	815.1	0.98	13.1	6.4	0.98	23.2	2.9	7.00E+05	543.4	0.1	14,469.1	1.87E-05	0.0019	5.9	137.4	44.3	10.8	0.0007	0.0006	0.001
7.48	9.65	0.62	0.32	19,300.0	19,566.4	1,239.2	649.8	822.8	0.0	822.8	0.98	13.2	6.6	0.99	22.6	2.9	6.98E+05	548.5	0.1	14,387.7	1.89E-05	0.0020	6.1	137.6	44.9	10.8	0.0007	0.0006	0.001
7.54	9.76	0.60	0.41	19,514.4	19,853.3	1,203.6	826.6	829.4	0.0	829.4	0.98	13.3	6.3	0.98	22.6	2.9	6.96E+05	552.9	0.1	14,318.9	1.91E-05	0.0020	6.0	134.7	43.5	10.8	0.0008	0.0007	0.001
7.61	9.86	0.61	0.16	19,728.8	19,856.1	1,226.4	310.4	837.1	0.0	837.1	0.98	13.5	6.4	0.99	22.5	2.9	7.03E+05	558.1	0.1	14,239.7	1.91E-05	0.0020	6.0	135.6	44.0	10.8	0.0008	0.0007	0.001
7.68	9.97	0.65	0.49	19,943.2	20,348.0	1,308.0	987.2	844.8	0.0	844.8	0.98	13.6	6.7	0.99	22.9	2.9	7.26E+05	563.2	0.1	14,161.7	1.87E-05	0.0019	6.1	139.5	45.5	10.8	0.0007	0.0006	0.000
7.74	10.54	0.71	0.48	21,086.4	21,477.3	1,411.0	953.4	851.4	0.0	851.4	0.98	13.7	6.8	0.99	23.9	2.9	7.60E+05	567.6	0.1	14,095.8	1.80E-05	0.0019	6.0	144.0	46.7	10.8	0.0007	0.0006	0.000
7.81	11.13	0.74	0.43	22,251.4	22,603.4	1,482.4	858.6	859.1	0.0	859.1	0.98	13.8	6.8	0.98	24.9	2.9	7.88E+05	572.7	0.1	14,019.8	1.75E-05	0.0018	5.9	146.5	47.1	10.8	0.0006	0.0006	0.000
7.87	12.28	0.76	0.5	24,550.6	24,924.2	1,519.6	911.2	865.7	0.0	865.7	0.98	13.9	6.3	0.96	26.9	2.9	8.21E+05	577.1	0.1	13,955.6	1.69E-05	0.0017	5.4	145.9	45.7	10.8	0.0006	0.0006	0.000
7.94	13.90	0.77	0.57	27,809.2	28,278.5	1,545.2	1,144.6	873.4	0.0	873.4	0.98	14.0	5.6	0.94	29.7	2.8	8.61E+05	582.3	0.1	13,881.6	1.63E-05	0.0017	4.8	143.7	43.4	10.8	0.0007	0.0006	0.000
8.00	15.32	0.81	0.78	30,635.0	31,271.2	1,628.8	1,551.8	880.0	0.0	880.0	0.98	14.1	5.4	0.92	32.3	2.8	9.07E+05	586.7	0.1	13,819.1	1.56E-05	0.0016	4.5	145.1	42.8	10.8	0.0006	0.0006	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta$s	**0.0**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-04					
Ground Elevation =	19.2	ft			
Water Table Depth from ground surface	11.2	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 5 feet																
5.09	153.74	4.04	0.10	307,472.0	307,511.9	8,084.0	199.7	559.4	0.0	559.4	0.99	9.0	2.6	1.04	513.9	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0041	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	68.61	2.91	0.18	137,216.0	137,287.9	5,812.0	359.6	577.4	0.0	577.4	0.99	9.3	4.3	1.02	11.1	3.0	3.06E+05	385.0	0.1	17,793.9	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
5.41	53.82	1.96	0.62	107,634.0	107,883.0	3,922.0	1,245.0	595.5	0.0	595.5	0.99	9.6	3.7	1.02	11.5	3.0	3.16E+05	397.0	0.1	17,468.3	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
5.58	70.64	1.86	0.40	141,284.0	141,442.5	3,722.0	792.3	613.5	0.0	613.5	0.99	9.9	2.6	1.03	11.5	3.0	3.26E+05	409.0	0.1	17,158.2	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
5.74	83.60	1.68	1.31	167,190.0	167,715.9	3,368.0	2,629.7	631.6	0.0	631.6	0.99	10.2	2.0	1.05	11.0	3.1	3.30E+05	421.0	0.1	16,862.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
5.91	97.58	1.98	1.93	195,156.0	195,926.9	3,954.0	3,854.7	649.6	0.0	649.6	0.99	10.5	2.0	1.06	10.6	3.1	3.32E+05	433.1	0.1	16,579.8	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.07	116.10	2.78	1.01	232,204.0	232,606.1	5,554.0	2,010.7	667.7	0.0	667.7	0.99	10.8	2.4	1.07	10.5	3.2	3.36E+05	445.1	0.1	16,309.4	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.23	115.46	2.77	1.46	230,924.0	231,509.9	5,548.0	2,929.4	685.7	0.0	685.7	0.99	11.1	2.4	1.05	10.9	3.1	3.43E+05	457.1	0.1	16,050.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.40	98.72	3.19	1.73	197,442.0	198,133.1	6,378.0	3,455.3	703.7	0.0	703.7	0.99	11.3	3.2	1.04	11.3	3.1	3.45E+05	469.2	0.1	15,802.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.56	116.88	2.48	1.43	233,766.0	234,336.6	4,956.0	2,852.8	721.8	0.0	721.8	0.98	11.6	2.1	1.03	11.1	3.1	3.39E+05	481.2	0.1	15,564.1	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.73	149.89	2.29	1.27	299,782.0	300,288.6	4,570.0	2,533.2	739.8	0.0	739.8	0.98	11.9	1.5	1.03	11.0	3.0	3.33E+05	493.2	0.1	15,335.2	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.89	119.92	2.50	0.81	239,838.0	240,162.9	5,004.0	1,624.5	757.9	0.0	757.9	0.98	12.2	2.1	1.02	10.9	3.0	3.34E+05	505.2	0.1	15,115.1	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
7.05	90.70	1.74	0.68	181,396.0	181,667.0	3,484.0	1,354.9	775.9	0.0	775.9	0.98	12.5	1.9	1.04	11.2	3.1	3.57E+05	517.3	0.1	14,903.2	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
7.22	140.06	2.02	1.87	280,116.0	280,865.7	4,042.0	3,748.3	794.0	0.0	794.0	0.98	12.8	1.4	0.99	13.5	2.9	3.88E+05	529.3	0.1	14,699.0	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
7.38	262.07	1.75	1.52	524,132.0	524,739.2	3,492.0	3,036.0	812.0	0.0	812.0	0.98	13.1	0.7	0.92	18.9	2.8	4.67E+05	541.3	0.1	14,502.2	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.55	194.71	1.99	0.83	389,422.0	389,753.5	3,988.0	1,657.7	830.1	0.0	830.1	0.98	13.3	1.0	0.87	24.1	2.6	5.38E+05	553.4	0.1	14,312.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.71	127.33	2.48	0.57	254,656.0	254,883.7	4,968.0	1,138.5	848.1	0.0	848.1	0.98	13.6	2.0	0.83	29.3	2.5	5.98E+05	565.4	0.1	14,128.7	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.87	63.85	2.33	0.48	127,690.0	127,883.1	4,664.0	965.4	866.1	0.0	866.1	0.98	13.9	3.7	0.83	30.7	2.5	6.20E+05	577.4	0.1	13,951.3	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
8.04	45.35	1.99	1.57	90,698.0	91,327.2	3,978.0	3,145.8	884.2	0.0	884.2	0.98	14.2	4.4	0.87	26.3	2.6	5.88E+05	589.5	0.1	13,779.8	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
8.20	72.17	1.74	5.35	144,348.0	146,487.1	3,476.0	10,695.6	902.2	0.0	902.2	0.98	14.5	2.4	0.91	22.2	2.7	5.55E+05	601.5	0.1	13,613.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
8.37	190.78	1.58	3.87	381,566.0	383,112.6	3,168.0	7,732.9	920.3	0.0	920.3	0.98	14.8	0.8	0.88	25.1	2.6	5.87E+05	613.5	0.1	13,453.0	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
8.53	217.78	1.66	1.97	435,552.0	436,338.3	3,318.0	3,931.3	938.3	0.0	938.3	0.98	15.0	0.8	0.85	27.6	2.6	6.13E+05	625.5	0.1	13,297.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
8.69	194.85	1.52	1.17	389,700.0	390,168.0	3,048.0	2,340.1	956.4	0.0	956.4	0.98	15.3	0.8	0.80	34.3	2.4	6.77E+05	637.6	0.1	13,146.0	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
8.86	159.11	1.32	0.76	318,224.0	318,528.2	2,634.0	1,521.2	974.4	0.0	974.4	0.98	15.6	0.8	0.76	41.2	2.3	7.48E+05	649.6	0.1	12,999.4	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
9.02	123.48	1.03	0.51	246,968.0	247,171.7	2,052.0	1,018.7	992.5	0.0	992.5	0.98	15.9	0.8	0.75	48.6	2.3	8.70E+05	661.6	0.1	12,857.1	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
9.19	88.89	0.99	0.37	177,774.0	177,923.8	1,970.0	748.9	1,010.5	0.0	1,010.5	0.98	16.2	1.1	0.69	61.3	2.1	9.37E+05	673.7	0.1	12,718.8	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
9.35	57.52	1.12	0.11	115,044.0	115,087.3	2,248.0	216.4	1,028.5	0.0	1,028.5	0.98	16.5	2.0	0.71	53.7	2.2	8.78E+05	685.7	0.1	12,584.5	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
9.51	48.55	1.34	0.11	97,106.0	97,148.6	2,674.0	213.1	1,046.6	0.0	1,046.6	0.98	16.7	2.8	0.76	43.2	2.3	7.95E+05	697.7	0.1	12,453.8	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
9.68	62.20	0.78	0.45	124,404.0	124,582.4	1,558.0	892.1	1,064.6	0.0	1,064.6	0.98	17.0	1.3	0.76	41.7	2.3	7.81E+05	709.8	0.1	12,326.8	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
9.84	137.80	0.66	0.94	275,604.0	275,979.5	1,310.0	1,877.5	1,082.7	0.0	1,082.7	0.98	17.3	0.5	0.80	41.5	2.4	8.50E+05	721.8	0.1	12,203.1	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
10.01	129.81	0.97	0.29	259,614.0	259,731.8	1,942.0	589.2	1,100.7	0.0	1,100.7	0.98	17.6	0.8	0.80	40.1	2.4	8.36E+05	733.8	0.1	12,082.7	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.001
10.17	100.17	0.59	0.00	200,338.0	200,336.0	1,180.0	-9.9	1,118.8	0.0	1,118.8	0.98	17.9	0.6	0.83	35.7	2.5	7.98E+05	745.8	0.1	11,965.3	1.79E-05	0.0018	2.8	101.1	26.2	10.8	0.0013	0.0012	0.001
10.33	113.29	0.97	0.13	226,578.0	226,630.6	1,940.0	262.9	1,136.8	0.0	1,136.8	0.98	18.2	0.9	0.90	26.4	2.7	7.10E+05	757.9	0.1	11,851.0	2.02E-05	0.0021	4.0	106.4	30.4	10.8	0.0013	0.0011	0.001
10.50	76.72	0.73	0.12	153,430.0	153,477.9	1,450.0	239.6	1,154.9	0.0	1,154.9	0.98	18.4	1.0	0.90	25.9	2.7	6.93E+05	769.9	0.1	11,739.6	2.09E-05	0.0022	3.9	101.7	28.8	10.8	0.0014	0.0012	0.001
10.66	53.37	0.64	-0.05	106,742.0	106,722.7	1,286.0	-96.5	1,172.9	0.0	1,172.9	0.98	18.7	1.2	0.90	25.4	2.7	6.82E+05	781.9	0.1	11,630.9	2.14E-05	0.0022	3.9	99.2	28.1	10.8	0.0015	0.0013	0.001
10.83	64.01	1.14	-0.17	128,024.0	127,956.8	2,278.0	-336.2	1,190.9	0.0	1,190.9	0.97	19.0	1.8	0.83	31.8	2.5	7.34E+05	794.0	0.1	11,524.8	2.00E-05	0.0021	2.9	91.3	23.7	10.8	0.0017	0.0015	0.001
10.99	41.70	1.20	-0.02	83,400.0	83,391.3	2,406.0	-43.3	1,209.0	0.0	1,209.0	0.97	19.3	2.9	0.77	34.3	2.4	6.89E+05	806.0	0.1	11,421.3	2.15E-05	0.0022	2.2	75.0	18.2	10.8	0.0025	0.0022	0.002
11.15	28.75	0.89	0.02	57,494.0	57,502.0	1,784.0	40.0	1,227.0	0.0	1,227.0	0.97	19.6	3.2	0.76	36.4	2.3	7.08E+05	818.0	0.1	11,320.2	2.11E-05	0.0022	2.0	74.3	17.7	10.8	0.0025	0.0022	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta$s	**0.1**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-05					
Ground Elevation =	17.1	ft			
Water Table Depth from ground surface	9.1	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf			
5.09	282.09	0.50	0.20	564,188.0	564,269.2	1,000.0	406.1	559.4	0.0	559.4	0.99	9.0	0.2	1.04	10.3
5.25	289.53	1.63	2.63	579,064.0	580,116.6	3,266.0	5,262.9	577.4	0.0	577.4	0.99	9.3	0.6	1.02	11.1
5.41	243.79	1.24	1.54	487,584.0	488,199.2	2,484.0	3,075.8	595.5	0.0	595.5	0.99	9.6	0.5	1.02	11.5
5.58	152.12	1.67	0.24	304,240.0	304,335.9	3,330.0	479.4	613.5	0.0	613.5	0.99	9.9	1.1	1.03	11.5
5.74	122.82	1.23	0.39	245,632.0	245,788.4	2,452.0	782.2	631.6	0.0	631.6	0.99	10.2	1.0	1.05	11.0
5.91	90.84	0.96	0.71	181,674.0	181,959.6	1,920.0	1,428.0	649.6	0.0	649.6	0.99	10.5	1.1	1.06	10.6
6.07	53.54	0.96	0.40	107,078.0	107,237.8	1,916.0	798.9	667.7	0.0	667.7	0.99	10.8	1.8	1.07	10.5
6.23	29.72	0.75	0.23	59,444.0	59,537.2	1,494.0	466.0	685.7	0.0	685.7	0.99	11.1	2.5	1.05	10.9
6.40	24.07	0.62	0.12	48,134.0	48,183.9	1,240.0	249.7	703.7	0.0	703.7	0.99	11.3	2.6	1.04	11.3
6.56	29.50	0.39	0.09	58,998.0	59,033.3	774.0	176.4	721.8	0.0	721.8	0.98	11.6	1.3	1.03	11.1
6.73	31.14	0.25	0.07	62,284.0	62,311.3	500.0	136.5	739.8	0.0	739.8	0.98	11.9	0.8	1.03	11.0
6.89	25.82	0.22	0.03	51,644.0	51,658.0	432.0	69.8	757.9	0.0	757.9	0.98	12.2	0.8	1.02	10.9
7.05	20.50	0.18	0.01	41,004.0	41,008.7	352.0	23.3	775.9	0.0	775.9	0.98	12.5	0.9	1.04	11.2
7.22	20.36	0.29	0.04	40,726.0	40,742.6	576.0	83.2	794.0	0.0	794.0	0.98	12.8	1.4	0.99	13.5
7.38	29.86	0.51	0.06	59,722.0	59,744.6	1,016.0	113.2	812.0	0.0	812.0	0.98	13.1	1.7	0.92	18.9
7.55	36.38	1.11	0.01	72,760.0	72,766.0	2,220.0	30.0	830.1	0.0	830.1	0.98	13.3	3.1	0.87	24.1
7.71	60.59	0.99	-0.01	121,172.0	121,168.7	1,986.0	-16.7	848.1	0.0	848.1	0.98	13.6	1.7	0.83	29.3
7.87	13.98	0.73	-0.02	27,966.0	27,956.7	1,450.0	-46.7	866.1	0.0	866.1	0.98	13.9	5.4	0.83	30.7
8.04	30.17	0.69	0.09	60,336.0	60,373.9	1,374.0	189.6	884.2	0.0	884.2	0.98	14.2	2.3	0.87	26.3
8.20	27.35	0.44	0.01	54,708.0	54,713.3	886.0	26.6	902.2	0.0	902.2	0.98	14.5	1.6	0.91	22.2
8.37	20.98	0.51	0.04	41,950.0	41,966.0	1,018.0	79.9	920.3	0.0	920.3	0.98	14.8	2.5	0.88	25.1
8.53	15.91	0.50	0.07	31,812.0	31,838.6	1,006.0	133.1	938.3	0.0	938.3	0.98	15.0	3.3	0.85	27.6
8.69	15.74	0.52	0.26	31,476.0	31,578.5	1,040.0	512.6	956.4	0.0	956.4	0.98	15.3	3.4	0.80	34.3
8.86	24.32	0.63	0.25	48,636.0	48,735.2	1,250.0	496.1	974.4	0.0	974.4	0.98	15.6	2.6	0.76	41.2
9.02	30.67	0.56	0.04	61,338.0	61,355.3	1,110.0	86.5	992.5	0.0	992.5	0.98	15.9	1.8	0.75	48.6

Hand Auger from 0 to 5 feet

Depth	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft		psf												in
5.09	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	3.0	3.06E+05	385.0	0.1	17,793.9	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
5.41	3.0	3.16E+05	397.0	0.1	17,468.3	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
5.58	3.0	3.26E+05	409.0	0.1	17,158.2	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
5.74	3.1	3.30E+05	421.0	0.1	16,862.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
5.91	3.1	3.32E+05	433.1	0.1	16,579.8	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.07	3.2	3.36E+05	445.1	0.1	16,309.4	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.23	3.1	3.43E+05	457.1	0.1	16,050.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.40	3.1	3.45E+05	469.2	0.1	15,802.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.56	3.1	3.39E+05	481.2	0.1	15,564.1	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.73	3.0	3.33E+05	493.2	0.1	15,335.2	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.89	3.0	3.34E+05	505.2	0.1	15,115.1	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
7.05	3.1	3.57E+05	517.3	0.1	14,903.2	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
7.22	2.9	3.88E+05	529.3	0.1	14,699.0	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
7.38	2.8	4.67E+05	541.3	0.1	14,502.2	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.55	2.6	5.38E+05	553.4	0.1	14,312.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.71	2.5	5.98E+05	565.4	0.1	14,128.7	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.87	2.5	6.20E+05	577.4	0.1	13,951.3	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
8.04	2.6	5.88E+05	589.5	0.1	13,779.8	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
8.20	2.7	5.55E+05	601.5	0.1	13,613.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
8.37	2.6	5.87E+05	613.5	0.1	13,453.0	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
8.53	2.6	6.13E+05	625.5	0.1	13,297.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
8.69	2.4	6.77E+05	637.6	0.1	13,146.0	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
8.86	2.3	7.48E+05	649.6	0.1	12,999.4	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
9.02	2.3	8.70E+05	661.6	0.1	12,857.1	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.000
										Total Estimated Settlement			2 x $\Sigma\Delta s$	**0.1**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-06			
Ground Elevation =	13.1	ft	
Water Table Depth from ground surface	5.1	ft	= EL 8 ft
γ =	110	pcf	
γ_{sat} =	120	pcf	
Atmospheric pressure	2,116.2	psf	
Cone Area Ratio	0.8		

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 5 feet																
5.09	107.30	0.75	0.41	214,600.0	214,765.1	1,506.0	825.6	559.4	0.0	559.4	0.99	9.0	0.7	1.04	10.3	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	110.92	0.68	0.36	221,842.0	221,985.1	1,352.0	715.7	578.9	9.3	569.6	0.99	9.4	0.6	1.02	11.1	3.0	3.06E+05	385.9	0.1	17,766.3	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta s$	**0.1**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

FUGRO

04.72190021
2/20/2020 TC

a_{max} = 0.81 g ASCE 7-16
Mw = 7.0

2020-CPT-07
Ground Elevation = 18.0 ft
Water Table Depth from ground surface 10.0 ft = EL 8 ft
γ = 110 pcf
γ_{sat} = 120 pcf
Atmospheric pressure 2,116.2 psf
Cone Area Ratio 0.8

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 5 feet																
5.09	53.37	0.62	0.23	106,742.0	106,832.5	1,246.0	452.7	559.4	0.0	559.4	0.99	9.0	1.2	1.04	10.3	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	47.10	1.46	-0.12	94,208.0	94,158.7	2,924.0	-246.4	577.4	0.0	577.4	0.99	9.3	3.1	1.02	11.1	3.0	3.06E+05	385.0	0.1	17,793.9	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
5.41	67.27	1.00	-0.14	134,542.0	134,484.7	1,996.0	-286.3	595.5	0.0	595.5	0.99	9.6	1.5	1.02	11.5	3.0	3.16E+05	397.0	0.1	17,468.3	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
5.58	89.19	1.27	0.03	178,388.0	178,400.0	2,544.0	59.9	613.5	0.0	613.5	0.99	9.9	1.4	1.03	11.5	3.0	3.26E+05	409.0	0.1	17,158.2	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
5.74	83.04	0.52	0.04	166,076.0	166,094.0	1,040.0	89.9	631.6	0.0	631.6	0.99	10.2	0.6	1.05	11.0	3.1	3.30E+05	421.0	0.1	16,862.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
5.91	65.68	1.18	0.23	131,368.0	131,461.9	2,364.0	469.3	649.6	0.0	649.6	0.99	10.5	1.8	1.06	10.6	3.1	3.32E+05	433.1	0.1	16,579.8	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.07	114.29	0.65	0.34	228,584.0	228,721.8	1,290.0	689.0	667.7	0.0	667.7	0.99	10.8	0.6	1.07	10.5	3.2	3.36E+05	445.1	0.1	16,309.4	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.23	39.92	0.86	-0.07	79,834.0	79,806.0	1,716.0	-139.8	685.7	0.0	685.7	0.99	11.1	2.2	1.05	10.9	3.1	3.43E+05	457.1	0.1	16,050.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.40	36.27	0.61	-0.06	72,536.0	72,513.4	1,218.0	-113.2	703.7	0.0	703.7	0.99	11.3	1.7	1.04	11.3	3.1	3.45E+05	469.2	0.1	15,802.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.56	146.91	0.82	0.03	293,822.0	293,834.0	1,648.0	59.9	721.8	0.0	721.8	0.98	11.6	0.6	1.03	11.1	3.1	3.39E+05	481.2	0.1	15,564.1	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.73	446.81	1.28	1.50	893,610.0	894,209.2	2,554.0	2,995.9	739.8	0.0	739.8	0.98	11.9	0.3	1.03	11.0	3.0	3.33E+05	493.2	0.1	15,335.2	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.89	471.79	1.31	1.08	943,582.0	944,012.1	2,622.0	2,150.5	757.9	0.0	757.9	0.98	12.2	0.3	1.02	10.9	3.0	3.34E+05	505.2	0.1	15,115.1	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
7.05	419.87	1.99	0.33	839,736.0	839,869.8	3,970.0	669.2	775.9	0.0	775.9	0.98	12.5	0.5	1.04	11.2	3.1	3.57E+05	517.3	0.1	14,903.2	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
7.22	326.25	3.58	0.40	652,492.0	652,651.1	7,166.0	795.6	794.0	0.0	794.0	0.98	12.8	1.1	0.99	13.5	2.9	3.88E+05	529.3	0.1	14,699.0	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
7.38	251.04	3.79	1.33	502,070.0	502,600.6	7,586.0	2,653.1	812.0	0.0	812.0	0.98	13.1	1.5	0.92	18.9	2.8	4.67E+05	541.3	0.1	14,502.2	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.55	209.17	3.69	4.16	418,336.0	419,999.7	7,374.0	8,318.7	830.1	0.0	830.1	0.98	13.3	1.8	0.87	24.1	2.6	5.38E+05	553.4	0.1	14,312.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.71	297.86	2.80	2.87	595,720.0	596,867.8	5,598.0	5,739.0	848.1	0.0	848.1	0.98	13.6	0.9	0.83	29.3	2.5	5.98E+05	565.4	0.1	14,128.7	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.87	288.31	2.54	0.72	576,612.0	576,899.6	5,070.0	1,438.1	866.1	0.0	866.1	0.98	13.9	0.9	0.83	30.7	2.5	6.20E+05	577.4	0.1	13,951.3	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
8.04	250.90	2.60	0.33	501,792.0	501,923.2	5,202.0	655.8	884.2	0.0	884.2	0.98	14.2	1.0	0.87	26.3	2.6	5.88E+05	589.5	0.1	13,779.8	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
8.20	250.34	2.14	0.65	500,678.0	500,937.0	4,272.0	1,294.8	902.2	0.0	902.2	0.98	14.5	0.9	0.91	22.2	2.7	5.55E+05	601.5	0.1	13,613.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
8.37	288.84	2.20	0.81	577,670.0	577,992.2	4,392.0	1,611.2	920.3	0.0	920.3	0.98	14.8	0.8	0.88	25.1	2.6	5.87E+05	613.5	0.1	13,453.0	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
8.53	276.66	2.62	1.37	553,326.0	553,873.3	5,240.0	2,736.3	938.3	0.0	938.3	0.98	15.0	0.9	0.85	27.6	2.6	6.13E+05	625.5	0.1	13,297.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
8.69	226.36	3.69	1.37	452,710.0	453,257.3	7,374.0	2,736.3	956.4	0.0	956.4	0.98	15.3	1.6	0.80	34.3	2.4	6.77E+05	637.6	0.1	13,146.0	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
8.86	129.14	3.43	1.27	258,278.0	258,786.0	6,862.0	2,539.9	974.4	0.0	974.4	0.98	15.6	2.7	0.76	41.2	2.3	7.48E+05	649.6	0.1	12,999.4	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
9.02	99.58	2.34	3.16	199,168.0	200,431.6	4,682.0	6,318.1	992.5	0.0	992.5	0.98	15.9	2.3	0.75	48.6	2.3	8.70E+05	661.6	0.1	12,857.1	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
9.19	108.75	1.99	2.91	217,498.0	218,661.8	3,976.0	5,818.8	1,010.5	0.0	1,010.5	0.98	16.2	1.8	0.69	61.3	2.1	9.37E+05	673.7	0.1	12,718.8	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
9.35	146.97	1.83	0.69	293,934.0	294,210.3	3,668.0	1,381.5	1,028.5	0.0	1,028.5	0.98	16.5	1.3	0.71	53.7	2.2	8.78E+05	685.7	0.1	12,584.5	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
9.51	129.36	1.65	0.05	258,724.0	258,744.6	3,292.0	103.2	1,046.6	0.0	1,046.6	0.98	16.7	1.3	0.76	43.2	2.3	7.95E+05	697.7	0.1	12,453.8	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
9.68	101.79	1.44	-0.02	203,570.0	203,562.0	2,870.0	-40.0	1,064.6	0.0	1,064.6	0.98	17.0	1.4	0.76	41.7	2.3	7.81E+05	709.8	0.1	12,326.8	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
9.84	79.03	0.81	-0.02	158,054.0	158,047.3	1,624.0	-33.3	1,082.7	0.0	1,082.7	0.98	17.3	1.0	0.80	41.5	2.4	8.50E+05	721.8	0.1	12,203.1	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
10.01	73.01	0.65	-0.06	146,020.0	145,994.0	1,300.0	-129.9	1,100.8	0.4	1,100.4	0.98	17.6	0.9	0.80	40.1	2.4	8.36E+05	733.9	0.1	12,082.2	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta s$	0.1

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-08					
Ground Elevation =	17.6	ft			
Water Table Depth from ground surface	9.6	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
0.16	4.98	0.14	0.18	9,950.0	10,023.5	282.0	367.5	18.0	0.0	18.0	1.00	0.3	2.8	0.00	4.2	0.0	0.00E+00	12.0	0.1	142,350.9	0.00E+00	0.0000	0.0	0.0	0.0	10.8	0.0000	0.0000	0.000
0.33	4.98	0.08	0.18	9,950.0	10,022.4	166.0	362.2	36.1	0.0	36.1	1.00	0.6	1.7	1.02	4.2	3.0	3.06E+05	24.1	0.1	93,916.5	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
0.49	10.37	0.19	0.07	20,746.0	20,773.0	376.0	135.1	54.1	0.0	54.1	1.00	0.9	1.8	1.02	4.2	3.0	3.16E+05	36.1	0.1	73,635.5	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
0.66	17.67	0.26	0.13	35,330.0	35,381.9	518.0	259.5	72.2	0.0	72.2	1.00	1.2	1.5	1.03	4.2	3.0	3.26E+05	48.1	0.1	61,961.8	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
0.82	23.43	0.33	0.19	46,862.0	46,937.7	660.0	378.4	90.2	0.0	90.2	1.00	1.5	1.4	1.05	4.2	3.1	3.30E+05	60.1	0.1	54,197.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
0.98	31.88	0.39	0.23	63,764.0	63,854.8	786.0	454.0	108.3	0.0	108.3	1.00	1.8	1.2	1.06	0.0	3.1	3.32E+05	72.2	0.1	48,581.3	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
1.15	40.45	0.61	0.23	80,892.0	80,983.9	1,210.0	459.5	126.3	0.0	126.3	1.00	2.1	1.5	1.07	4.2	3.2	3.36E+05	84.2	0.1	44,289.6	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
1.31	37.48	0.91	0.23	74,958.0	75,048.8	1,822.0	454.0	144.4	0.0	144.4	1.00	2.4	2.4	1.05	4.2	3.1	3.43E+05	96.2	0.1	40,879.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
1.48	46.21	0.78	0.21	92,424.0	92,506.2	1,550.0	410.8	162.4	0.0	162.4	1.00	2.6	1.7	1.04	4.2	3.1	3.45E+05	108.3	0.1	38,090.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
1.64	37.68	0.76	0.16	75,354.0	75,418.9	1,520.0	324.3	180.4	0.0	180.4	1.00	2.9	2.0	1.03	4.2	3.1	3.39E+05	120.3	0.1	35,756.9	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
1.80	42.45	0.91	0.17	84,906.0	84,973.0	1,812.0	335.1	198.5	0.0	198.5	1.00	3.2	2.1	1.03	4.2	3.0	3.33E+05	132.3	0.1	33,769.5	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
1.97	52.94	1.21	0.20	105,878.0	105,959.1	2,414.0	405.4	216.5	0.0	216.5	1.00	3.5	2.3	1.02	4.2	3.0	3.34E+05	144.4	0.1	32,051.7	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
2.13	89.12	1.44	0.29	178,236.0	178,351.7	2,872.0	578.4	234.6	0.0	234.6	1.00	3.8	1.6	1.04	4.2	3.1	3.57E+05	156.4	0.1	30,548.8	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
2.30	91.29	1.70	0.25	182,588.0	182,687.4	3,392.0	497.2	252.6	0.0	252.6	0.99	4.1	1.9	0.99	4.2	2.9	3.88E+05	168.4	0.1	29,220.2	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
2.46	68.85	1.69	0.29	137,704.0	137,818.6	3,370.0	573.0	270.7	0.0	270.7	0.99	4.4	2.5	0.92	4.2	2.8	4.67E+05	180.4	0.1	28,035.3	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
2.62	93.24	1.29	0.26	186,488.0	186,594.0	2,582.0	529.8	288.7	0.0	288.7	0.99	4.7	1.4	0.87	4.2	2.6	5.38E+05	192.5	0.1	26,970.4	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
2.79	139.51	1.36	0.31	279,026.0	279,149.2	2,720.0	616.2	306.8	0.0	306.8	0.99	5.0	1.0	0.83	4.2	2.5	5.98E+05	204.5	0.1	26,007.0	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
2.95	275.86	1.11	0.26	551,722.0	551,828.0	2,224.0	529.8	324.8	0.0	324.8	0.99	5.3	0.4	0.83	4.2	2.5	6.20E+05	216.5	0.1	25,130.2	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
3.12	383.61	1.15	0.21	767,210.0	767,293.3	2,302.0	416.3	342.8	0.0	342.8	0.99	5.6	0.3	0.87	4.2	2.6	5.88E+05	228.6	0.1	24,328.1	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
3.28	412.77	0.90	0.24	825,546.0	825,641.1	1,796.0	475.6	360.9	0.0	360.9	0.99	5.9	0.2	0.91	4.2	2.7	5.55E+05	240.6	0.1	23,590.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
3.44	310.65	1.26	0.27	621,308.0	621,416.1	2,514.0	540.4	378.9	0.0	378.9	0.99	6.2	0.4	0.88	4.2	2.6	5.87E+05	252.6	0.1	22,910.2	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
3.61	190.87	1.91	0.22	381,738.0	381,827.7	3,822.0	448.6	397.0	0.0	397.0	0.99	6.4	1.0	0.85	4.2	2.6	6.13E+05	264.7	0.1	22,279.6	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
3.77	112.69	1.90	0.17	225,380.0	225,447.0	3,802.0	335.1	415.0	0.0	415.0	0.99	6.7	1.7	0.80	4.2	2.4	6.77E+05	276.7	0.1	21,693.2	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
3.94	68.77	1.54	0.29	137,534.0	137,649.7	3,082.0	578.4	433.1	0.0	433.1	0.99	7.0	2.2	0.76	4.2	2.3	7.48E+05	288.7	0.1	21,146.3	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
4.10	85.98	1.44	0.38	171,960.0	172,113.5	2,878.0	767.5	451.1	0.0	451.1	0.99	7.3	1.7	0.75	4.2	2.3	8.70E+05	300.7	0.1	20,634.6	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
4.27	83.55	1.12	0.31	167,100.0	167,224.3	2,230.0	621.6	469.2	0.0	469.2	0.99	7.6	1.3	0.69	4.2	2.1	9.37E+05	312.8	0.1	20,154.7	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
4.43	67.81	1.23	0.28	135,612.0	135,722.3	2,454.0	551.4	487.2	0.0	487.2	0.99	7.9	1.8	0.71	4.2	2.2	8.78E+05	324.8	0.1	19,703.4	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
4.59	62.66	0.66	0.24	125,324.0	125,419.1	1,310.0	475.6	505.2	0.0	505.2	0.99	8.2	1.0	0.76	4.2	2.3	7.95E+05	336.8	0.1	19,278.1	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
4.76	60.03	0.90	0.22	120,066.0	120,153.6	1,804.0	437.9	523.3	0.0	523.3	0.99	8.5	1.5	0.76	4.2	2.3	7.81E+05	348.9	0.1	18,876.5	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
4.92	40.67	0.91	0.10	81,344.0	81,384.0	1,824.0	200.0	541.3	0.0	541.3	0.99	8.8	2.3	0.80	4.2	2.4	8.50E+05	360.9	0.1	18,496.4	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
5.09	29.48	1.09	0.12	58,960.0	59,007.6	2,186.0	237.9	559.4	0.0	559.4	0.99	9.0	3.7	0.80	4.2	2.4	8.36E+05	372.9	0.1	18,136.1	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.001
5.25	43.10	0.35	0.14	86,206.0	86,263.3	708.0	286.6	577.4	0.0	577.4	0.99	9.3	0.8	0.83	4.2	2.5	7.98E+05	385.0	0.1	17,793.9	1.79E-05	0.0018	2.8	101.1	26.2	10.8	0.0013	0.0012	0.001
5.41	516.36	3.05	0.21	#########	#########	6,094.0	427.0	595.5	0.0	595.5	0.99	9.6	0.6	0.90	4.2	2.7	7.10E+05	397.0	0.1	17,468.3	2.02E-05	0.0021	4.0	106.4	30.4	10.8	0.0013	0.0011	0.001
5.58	273.35	4.62	0.16	546,690.0	546,756.0	9,234.0	329.8	613.5	0.0	613.5	0.99	9.9	1.7	0.90	4.2	2.7	6.93E+05	409.0	0.1	17,158.2	2.09E-05	0.0022	3.9	101.7	28.8	10.8	0.0014	0.0012	0.001
5.74	231.85	4.67	-0.17	463,706.0	463,636.8	9,336.0	-345.9	631.6	0.0	631.6	0.99	10.2	2.0	0.90	4.2	2.7	6.82E+05	421.0	0.1	16,862.4	2.14E-05	0.0022	3.9	99.2	28.1	10.8	0.0015	0.0013	0.001
5.91	228.94	2.71	0.02	457,884.0	457,893.7	5,422.0	48.7	649.6	0.0	649.6	0.99	10.5	1.2	0.83	4.2	2.5	7.34E+05	433.1	0.1	16,579.8	2.00E-05	0.0021	2.9	91.3	23.7	10.8	0.0017	0.0015	0.001
6.07	152.74	2.72	0.11	305,482.0	305,524.2	5,440.0	210.8	667.7	0.0	667.7	0.99	10.8	1.8	0.77	4.2	2.4	6.89E+05	445.1	0.1	16,309.4	2.15E-05	0.0022	2.2	75.0	18.2	10.8	0.0025	0.0022	0.002
6.23	109.64	2.46	0.04	219,276.0	219,293.3	4,914.0	86.5	685.7	0.0	685.7	0.99	11.1	2.2	0.76	4.2	2.3	7.08E+05	457.1	0.1	16,050.6	2.11E-05	0.0022	2.0	74.3	17.7	10.8	0.0025	0.0022	0.002
6.40	130.38	1.19	0.02	260,768.0	260,775.6	2,384.0	37.9	703.7	0.0	703.7	0.99	11.3	0.9	0.75	4.2	2.3	6.52E+05	469.2	0.1	15,802.3	2.31E-05	0.0024	2.0	67.1	15.8	10.8	0.0032	0.0028	0.002
6.56	119.19	1.48	0.02	238,382.0	238,391.7	2,954.0	48.7	721.8	0.0	721.8	0.98	11.6	1.2	0.76	4.2	2.3	6.18E+05	481.2	0.1	15,564.1	2.45E-05	0.0026	2.1	64.8	15.5	10.8	0.0035	0.0030	0.002
6.73	117.98	1.06	0.05	235,952.0	235,970.4	2,128.0	91.9	739.8	0.0	739.8	0.98	11.9	0.9	0.79	4.2	2.4	6.07E+05	493.2	0.1	15,335.2	2.52E-05	0.0026	2.3	66.0	16.2	10.8	0.0034	0.0029	0.002
6.89	142.65	1.93	0.06	285,302.0	285,327.9	3,854.0	129.7	757.9	0.0	757.9	0.98	12.2	1.4	0.85	4.2	2.6	6.30E+05	505.2	0.1	15,115.1	2.44E-05	0.0025	3.1	78.7	20.9	10.8	0.0024	0.0021	0.002
7.05	108.28	1.49	0.19	216,562.0	216,638.8	2,982.0	383.8	775.9	0.0	775.9	0.98	12.5	1.4	0.89	4.2	2.7	6.44E+05	517.3	0.1	14,903.2	2.41E-05	0.0025	3.7	86.5	24.1	10.8	0.0020	0.0017	0.001
7.22	205.85	1.72	0.31	411,700.0	411,822.2	3,434.0	610.8	794.0	0.0	794.0	0.98	12.8	0.8	0.90	4.2	2.7	7.39E+05	529.3	0.1	14,699.0	2.11E-05	0.0022	4.0	102.3	29.2	10.8	0.0014	0.0012	0.001
7.38	242.34	1.99	0.22	484,678.0	484,767.7	3,980.0	448.6	812.0	0.0	812.0	0.98	13.1	0.8	0.67	4.2	2.1	1.00E+06	541.3	0.1	14,502.2	1.57E-05	0.0016	1.4	89.7	19.4	10.8	0.0017	0.0014	0.001
7.55	162.46	2.11	0.25	324,928.0	325,029.6	4,210.0	508.2	830.1	0.0	830.1	0.98	13.3	1.3	0.56	4.2	1.8	1.14E+06	553.4	0.1	14,312.2	1.38E-05	0.0014	1.1	103.7	20.1	10.8	0.0014	0.0012	0.001
7.71	139.91	2.00	0.26	279,818.0	279,922.9	4,000.0	524.3	848.1	0.0	848.1	0.98	13.6	1.4	0.59	4.2	1.9	1.07E+06	565.4	0.1	14,128.7	1.49E-05	0.0015	1.2	94.8	18.9	10.8	0.0016	0.0014	0.001
7.87	149.80	2.07	0.26	299,602.0	299,706.9	4,138.0	524.3	866.1	0.0	866.1	0.98	13.9	1.4	0.63	4.2	2.0	1.00E+06	577.4	0.1	13,951.3	1.60E-05	0.0016	1.3	87.5	18.2	10.8	0.0018	0.0016	0.001
8.04	135.56	2.17	0.20	271,112.0	271,190.9	4,342.0	394.6	884.2	0.0	884.2	0.98	14.2	1.6	0.69	4.2	2.1	9.46E+05	589.5	0.1	13,779.8	1.71E-05	0.0018	1.5	84.2	18.5	10.8	0.0019	0.0017	0.001
8.20	112.32	1.44	0.07	224,646.0	224,674.1	2,882.0	140.5	902.2	0.0	902.2	0.98	14.5	1.3	0.72	4.2	2.2	9.04E+05	601.5	0.1	13,613.8	1.80E-05	0.0019	1.7	82.7	18.7	10.8	0.0020	0.0017	0.001
8.37	100.76	0.32	0.16	201,526.0	201,590.9	646.0	324.3	920.3	0.0	920.3	0.98	14.8	0.3	0.76	4.2	2.3	8.76E+05	613.5	0.1	13,453.0	1.87E-05	0.0019	2.0	84.7	20.1	10.8	0.0019	0.0017	0.001
8.53	89.63	0.36	0.06	179,252.0	179,275.8	718.0	118.9	938.3	0.0	938.3	0.98	15.0	0.4	0.79	4.2	2.4	8.87E+05	625.5	0.1	13,297.1	1.86E-05	0.0019	2.4	91.6	22.7	10.8	0.0017	0.0014	0.001
8.69	65.23	1.16	0.06	130,468.0	130,490.7	2,320.0	113.5	956.4	0.0	956.4	0.98	15.3	1.8	0.81	4.2	2.5	9.13E+05	637.6	0.1	13,146.0	1.82E-05	0.0019	2.6	97.2	24.5	10.8	0.0015	0.0013	0.001
8.86	41.13	0.51	0.03	82,250.0	82,261.9	1,018.0	59.5	974.4	0.0	974.4	0.98	15.6	1.3	0.80	4.2	2.4	8.69E+05	649.6	0.1	12,999.4	1.93E-05	0.0020	2.5	90.7	22.7	10.8	0.0017	0.0015	0.001
9.02	63.43	0.68	0.02	126,850.0	126,858.6	1,356.0	43.2	992.5	0.0	992.5	0.98	15.9	1.1	0.77	4.2	2.4	8.08E+05	661.6	0.1	12,857.1	2.09E-05	0.0022	2.1	78.6	18.9	10.8	0.0023	0.0020	0.002
9.19	32.67	0.62	0.04	65,348.0	65,364.2	1,244.0	81.1	1,010.5	0.0	1,010.5	0.98	16.2	1.9	0.74	4.2	2.3	8.15E+05	673.7	0.1	12,718.8	2.08E-05	0.0022	1.9	75.1	17.5	10.8	0.0025	0.0022	0.002
9.35	28.01	0.56	0.29	56,020.0	56,134.6	1,116.0	573.0	1,028.5	0.0	1,028.5	0.98	16.5	2.0	0.71	4.2	2.2	8.41E+05	685.7	0.1	12,584.5	2.03E-05	0.0021	1.6	73.3	16.4	10.8	0.0027	0.0023	0.002
9.51	24.05	0.35	0.17	48,106.0	48,175.2	698.0	345.9	1,046.6	0.0	1,046.6	0.98	16.7	1.5	0.70	4.2	2.2	8.37E+05	697.7	0.1	12,453.8	2.05E-05	0.0021	1.6	72.0	15.9	10.8	0.0028	0.0024	0.002
9.68	23.60	0.35	0.14	47,202.0	47,257.1	690.0	275.6	1,065.4	4.9	1,060.5	0.98	17.0	1.5	0.71	4.2	2.2	7.81E+05	710.3	0.1	12,321.3	2.22E-05	0.0023	1.7	67.9	15.3	10.8	0.0032	0.0027	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta$s	0.2

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-B-1

Ground Elevation =			19.8	ft			
Depth to Groumd Water Table =			11.8	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ =	0.925						
ϕ	35	degree					

Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	ΔS (in)
ft	*ft*	*m*	*ft*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*psf*	*tsf*		*psf*			*Y/N*		*%*	*%*	*%*	*in*
17.8	2	0.6	2.5	12	220.0	10.5	220.0	10.5	0.75	N	1.00	1.15	1	1.70	18	135.9	0.07	52.0	6.1E+05	1.00	1.9E-04	Y	0.00035	0.035	0.036	0.03	0.01
15.8	4	1.2	2.0	13	440.0	21.1	440.0	21.1	0.75	Y	1.13	1.15	1	1.70	22	271.8	0.14	55.6	9.2E+05	1.00	2.5E-04	Y	0.00036	0.036	0.030	0.03	0.01
13.8	6	1.8	2.5	13	660.0	31.6	660.0	31.6	0.80	N	1.00	1.15	1	1.70	20	407.6	0.20	54.6	1.1E+06	0.99	3.1E-04	Y	0.00500	0.500	0.500	0.46	0.14
10.3	9.5	2.9	2.5	31	1,045.0	50.1	1,045.0	50.1	0.85	N	1.00	1.15	1	1.28	39	645.4	0.32	67.8	1.7E+06	0.98	3.1E-04	Y	0.00250	0.250	0.075	0.07	0.02
																										Total	0.2
																										Multi-directional Shaking Total	0.4

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-B-2

Ground Elevation =			18.2	ft			
Depth to Groumd Water Table =			10.2	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ =	0.925						
ϕ	35	degree					

Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	Δs (in)
ft	*ft*	*m*	*ft*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*psf*	*tsf*		*psf*			*Y/N*		*%*	*%*	*%*	*in*
16.2	2	0.6	1.0	9	220.0	10.5	220.0	10.5	0.75	N	1.00	1.15	1	1.70	13	135.9	0.07	47.3	5.5E+05	1.00	2.1E-04	Y	0.010	1.0	1.3	1.2	0.14
14.2	4	1.2	5.0	23	440.0	21.1	440.0	21.1	0.75	Y	1.23	1.15	1	1.70	41	271.8	0.14	69.2	1.1E+06	1.00	2.0E-04	Y	0.002	0.2	0.1	0.1	0.03
12.2	6	1.8	1.0	6	660.0	31.6	660.0	31.6	0.80	N	1.00	1.15	1	1.70	9	407.6	0.20	42.2	8.5E+05	0.99	4.0E-04	Y	0.010	1.0	1.0	0.9	0.11
																										Total	0.3
																										Multi-directional Shaking Total	0.6

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-B-3

Ground Elevation =			19.2	ft			
Depth to Groumd Water Table =			11.2	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ =	0.925						
ϕ	35	degree					

Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	ΔS (in)
ft	*ft*	*m*	*ft*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*psf*	*tsf*		*psf*			*Y/N*		*%*	*%*	*%*	*in*
14.7	4.5	1.4	5.0	21	495.0	23.7	495.0	23.7	0.75	N	1.00	1.15	1	1.70	31	305.7	0.15	62.7	1.1E+06	0.99	2.4E-04	Y	0.003	0.3	0.2	0.1	0.09
																										Total	0.1
																										Multi-directional Shaking Total	0.2

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} = 0.81 g ASCE 7-16
Mw = 7.0

2020-B-01
Ground Elevation = 17.5 ft
Depth to Ground Water Table = 9.5 ft = EL 8 ft
γ = 110 pcf
γ_{sat} = 120 pcf
Boring Diameter = 4 inch = 101.6 mm
Rod Length Above Ground = 3 ft = 0.9 m
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ = 0.925
ϕ 35 degree
Energy Ratio = 84%

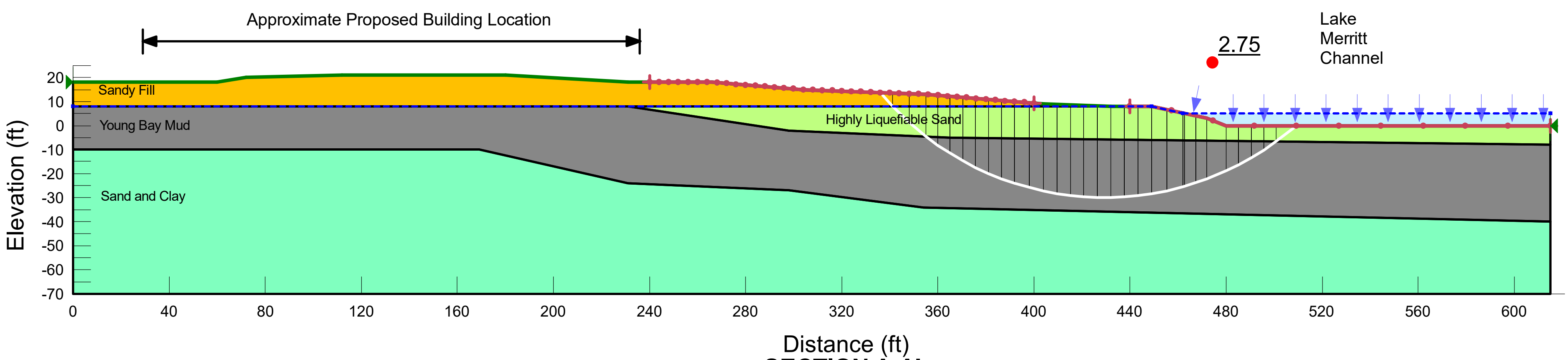
Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	Δs (in)
ft	ft	m	ft	blow/ft	psf	kPa	psf	kPa		Y/N						psf	tsf		psf			Y/N		%	%	%	in
13.5	4.0	1.2	6.5	19	440.0	21.1	440.0	21.1	0.75	N	1.00	1	1.4	1.70	34	271.8	0.14	64.7	1.1E+06	1.00	2.2E-04	Y	0.0025	0.250	0.100	0.09	0.07
9.5	8.0	2.4	3.0	4	880.0	42.2	880.0	42.2	0.80	Y	1.10	1	1.4	1.70	8	543.5	0.27	40.6	9.5E+05	0.98	4.8E-04	N	-	-	-	-	-
																										Total	0.1
																										Multi-directional Shaking Total	0.1

Appendix F

Slope Stability Analyses

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		

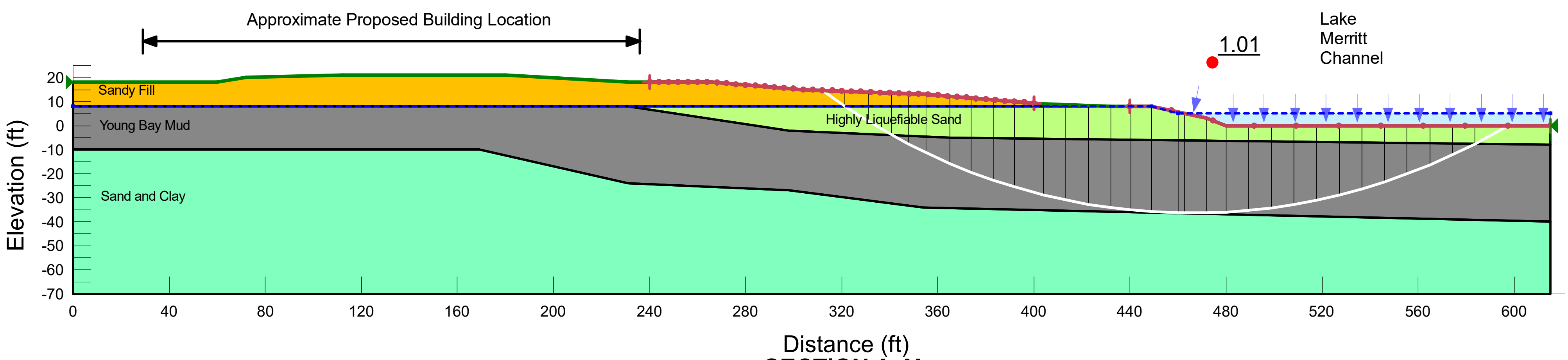


SECTION A-A'

PLATE F-1

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.12
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		

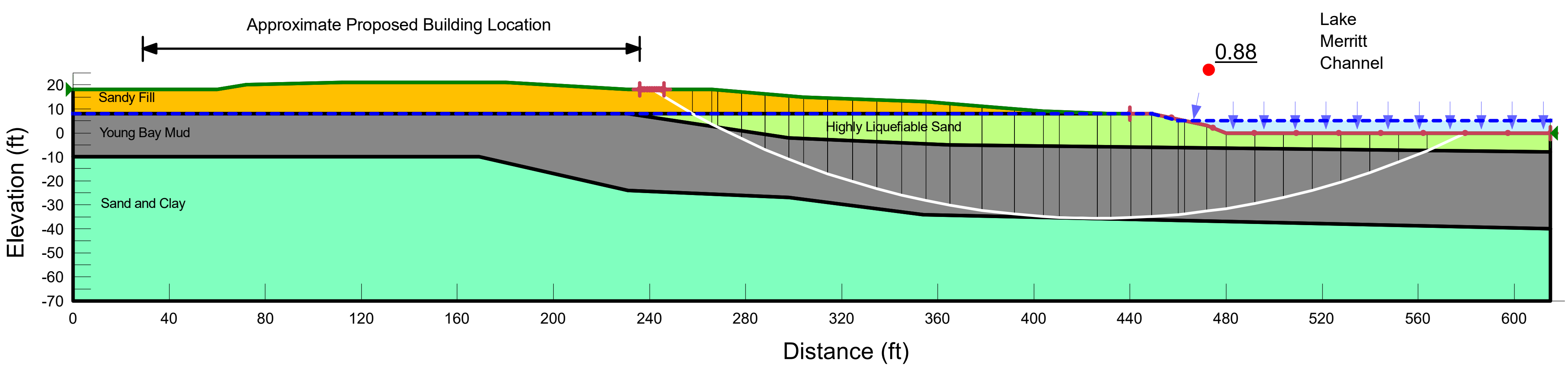


SECTION A-A'

PLATE F-2

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 3 - Pseudo-Static k = 0.15g; Fixed Slip Surface at Edge of Building
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		

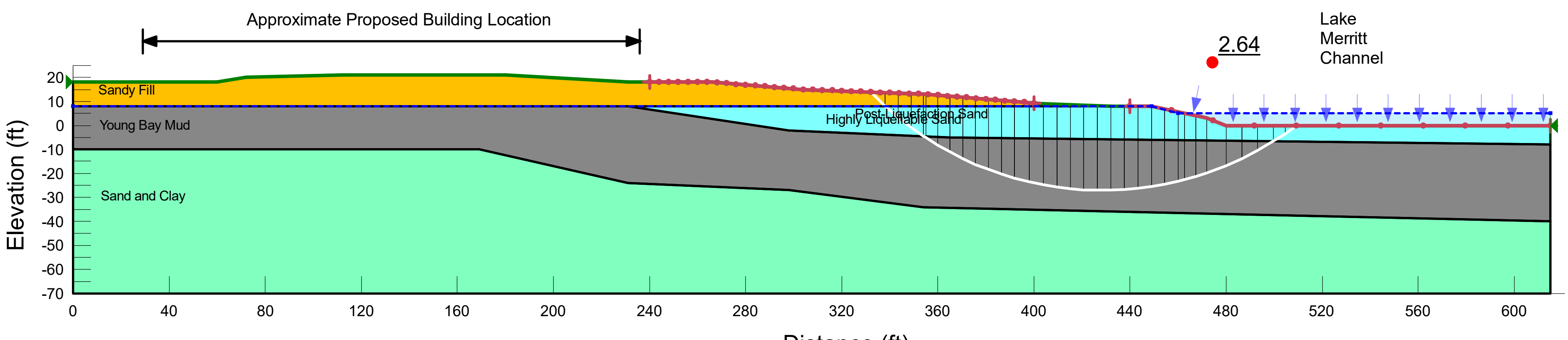


SECTION A-A'

PLATE F-3

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)
	Sandy Fill	Mohr-Coulomb	120	0	35	1						
	Young Bay Mud	S=f(overburden)	90			1	0.35	350				
	Sand and Clay	Mohr-Coulomb	130	0	40	1						
	Post-Liquefaction Sand	S=f(datum)	110			1			100	20	500	8

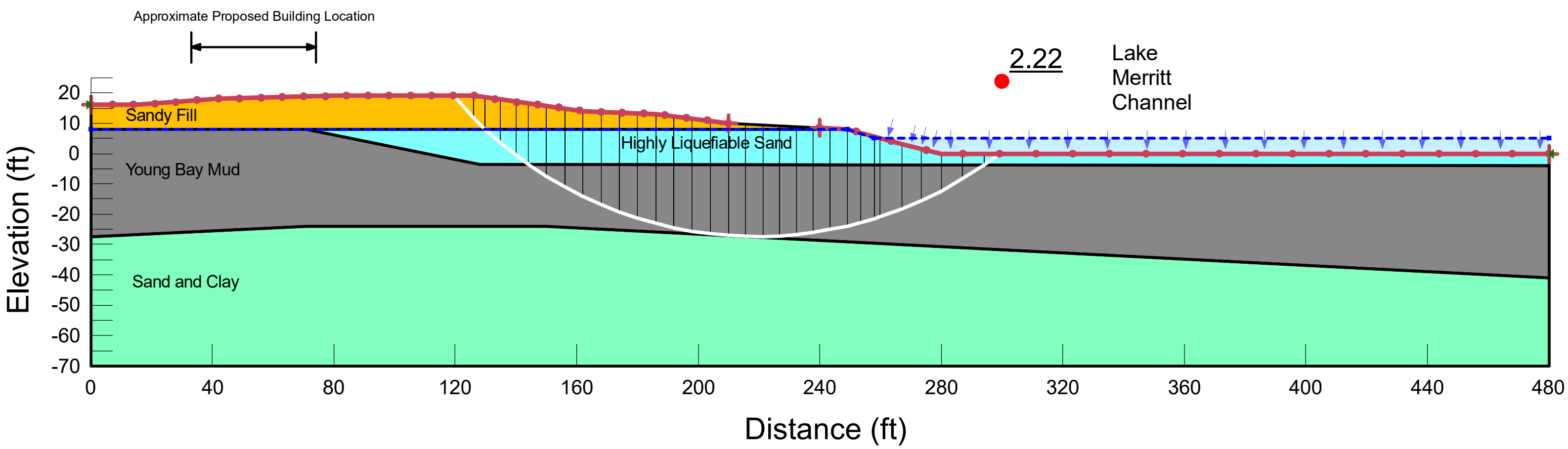


SECTION A-A'

PLATE F-4

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

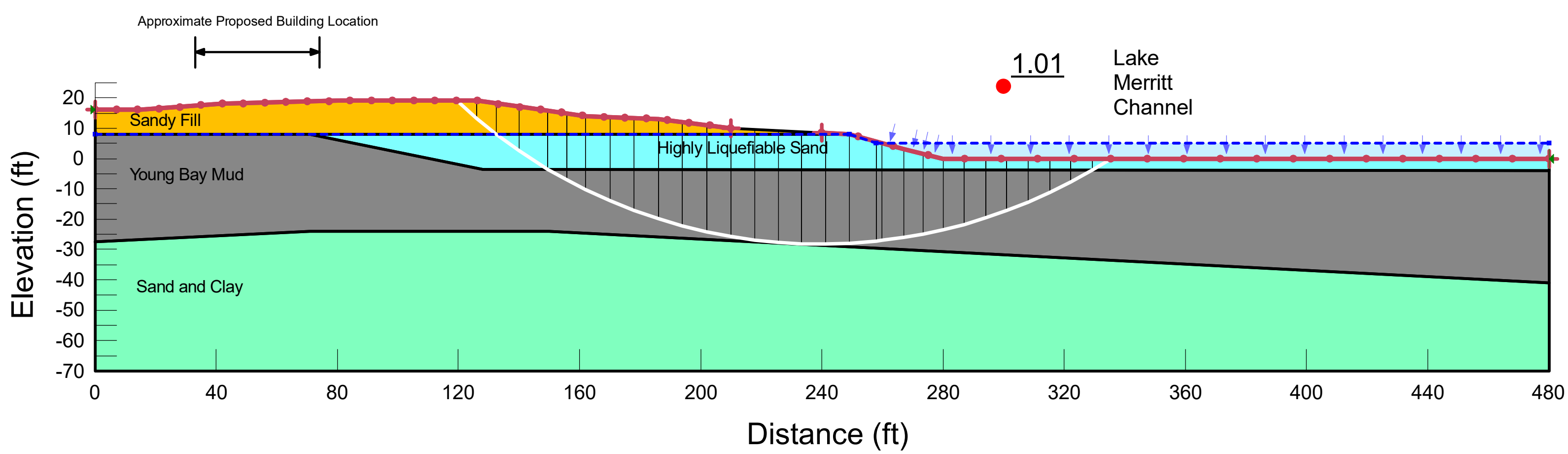


SECTION D-D'

PLATE F-5

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.12
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

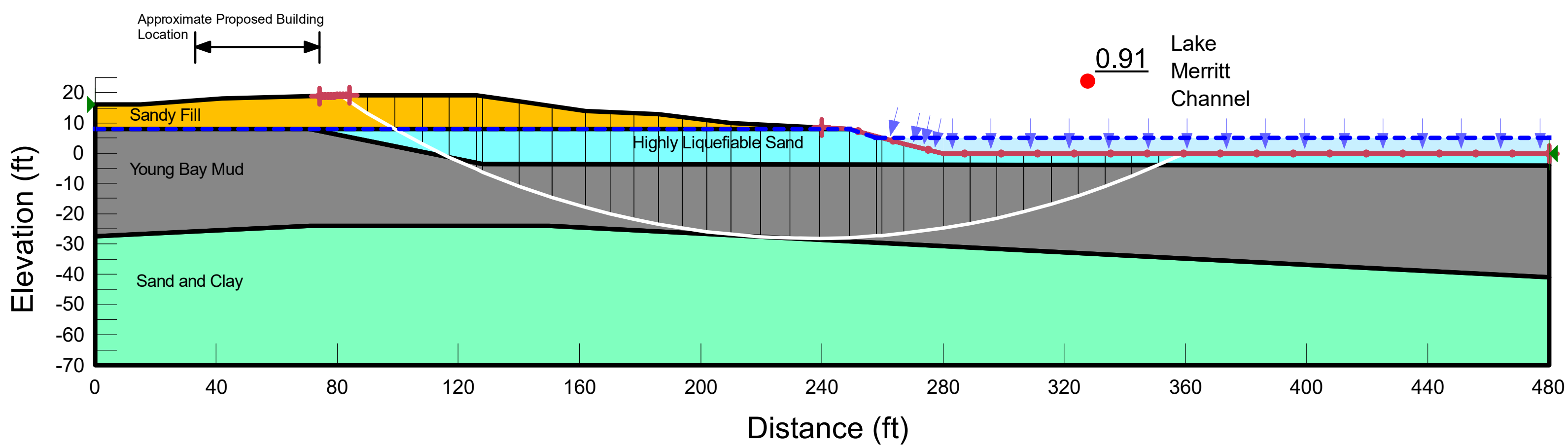


SECTION D-D'

PLATE F-6

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 3 - Pesudo-Static k = 0.15g; Fixed Slip Surface at Edge of Building
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

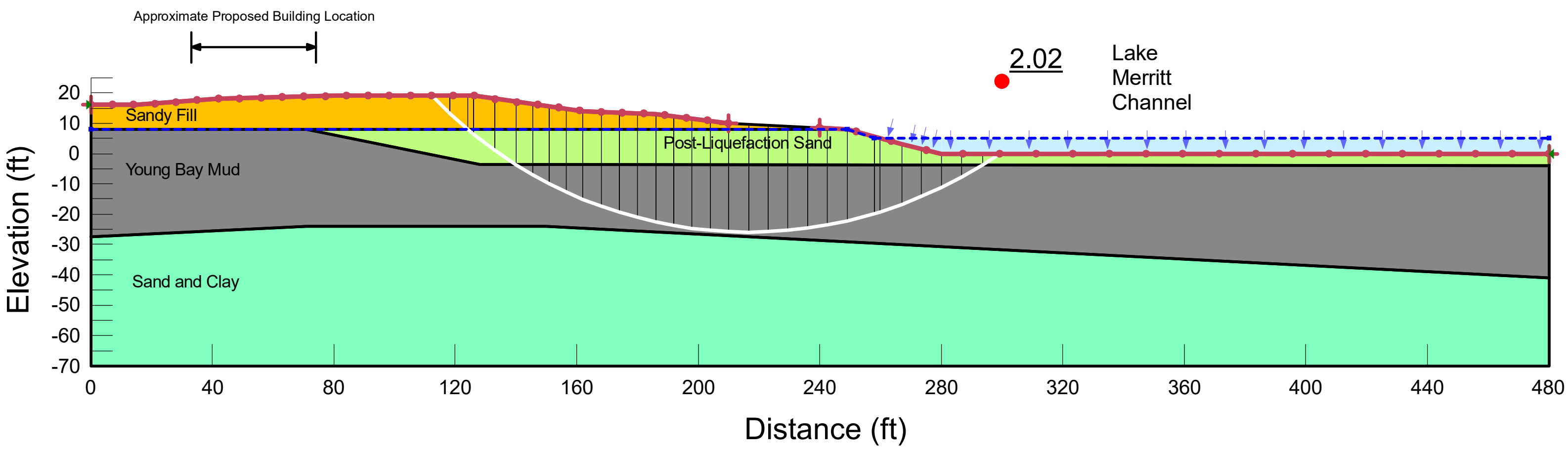


SECTION D-D'

PLATE F-7

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1						
	Post-Liquefaction Sand	S=f(datum)	110			1	100	20	500	8		
	Young Bay Mud	S=f(overburden)	90			1					0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1						

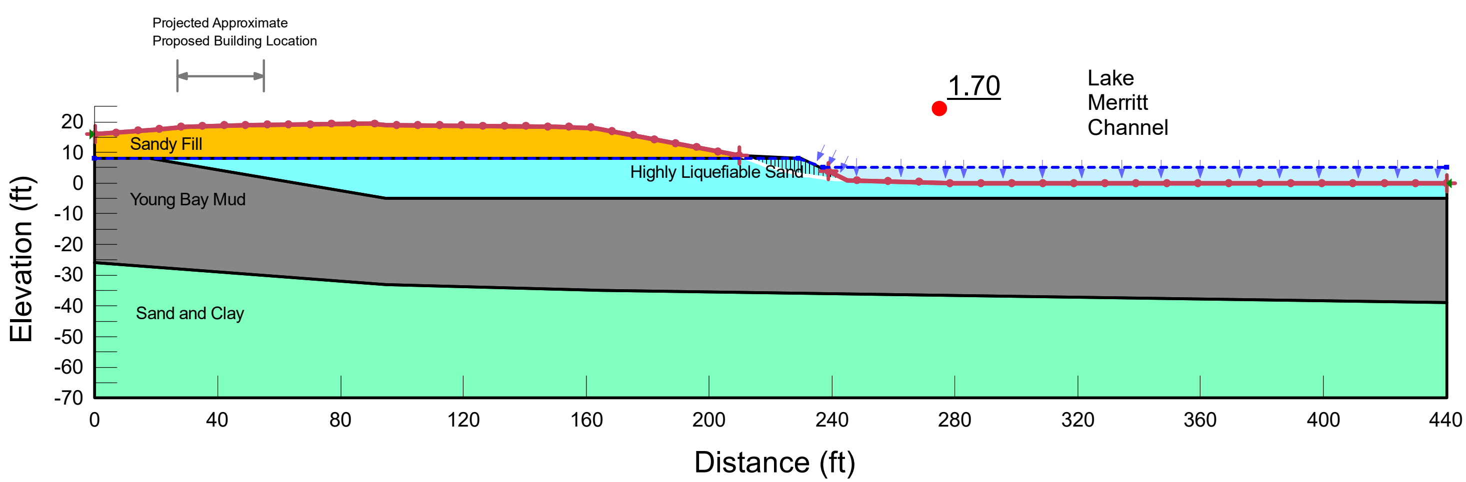


SECTION D-D'

PLATE F-8

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

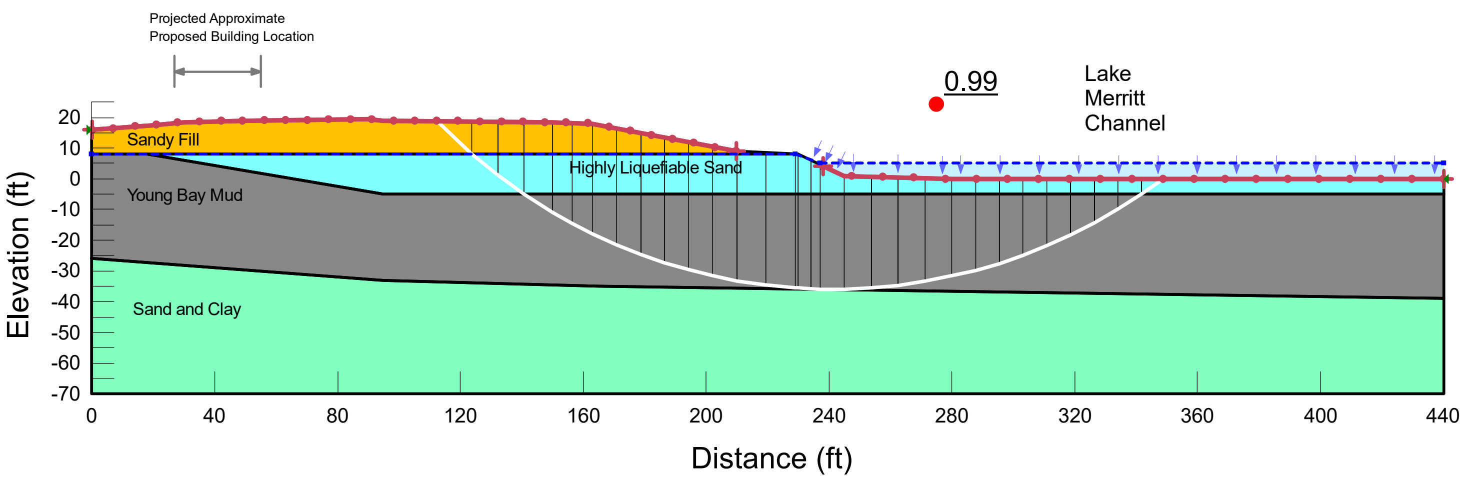


SECTION E-E'

PLATE F-9

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.11
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

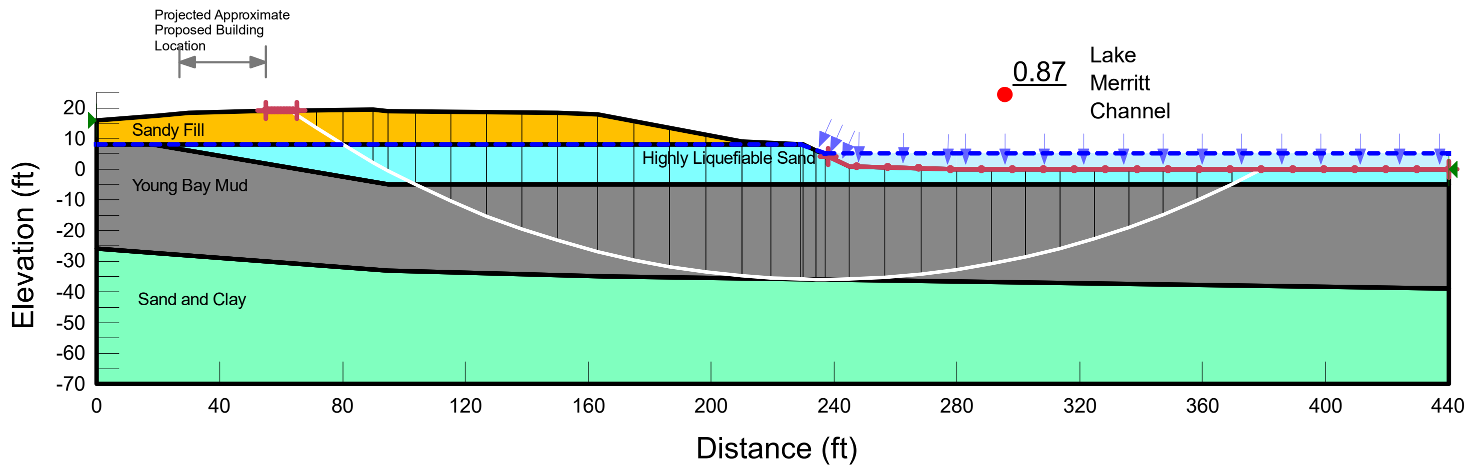


SECTION E-E'

PLATE F-10

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 3 - Pesudo-Static k = 0.15g; Fixed Slip Surface at Edge of Building
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

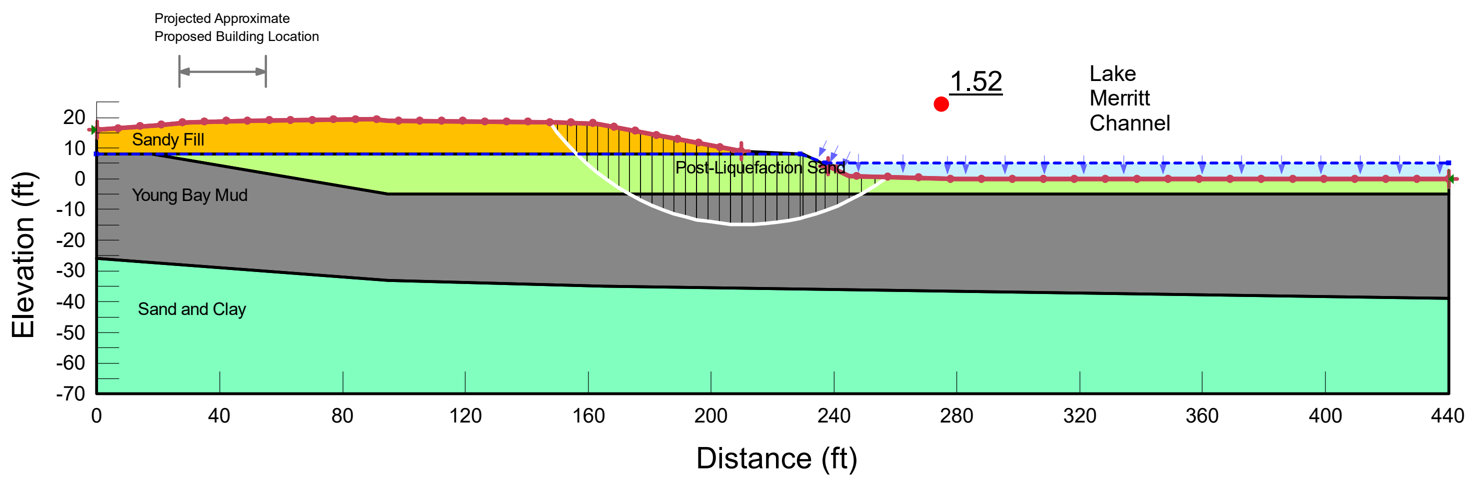


SECTION E-E'

PLATE F-11

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1						
	Post-Liquefaction Sand	S=f(datum)	110			1	100	20	500	8		
	Young Bay Mud	S=f(overburden)	90			1					0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1						



SECTION E-E'

PLATE F-12

Appendix G

Site-Specific Ground Motion Analyses

Contents

Tables in the Main Text

G.1 Introduction

This appendix summarizes a site-specific seismic hazard assessment and site response analyses conducted to estimate the severity of ground motions that may affect the project site for specific design levels of hazard. The seismic hazard assessment was conducted using the seismic source model adopted by the United States Geological Survey (USGS) to develop the 2014 National Seismic Hazard Map Project (NSHMP) (Petersen et al., 2014), and the NGA West 2 Ground Motion Models (Bozorgnia et al., 2014).

A liquefaction triggering hazard assessment indicated that the soils at the site are potentially liquefiable. Therefore, according to ASCE 7-16, the site is classified as Site Class F, and site response analyses are required to calculate the design ground motions at the ground surface. These site response analyses were performed using the commercial finite-difference program FLAC (Itasca, 2016) and evaluated the effect of nonlinear dynamic response of the soft and liquefiable soils at the site on the surface ground motions. The design ground motion parameters were calculated following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-16 (ASCE, 2016; 2018) as required by the 2019 California Building Code (CBC) (CBSC, 2019).

G.2 Subsurface Conditions for the Seismic Hazard Assessment

Subsurface conditions at the project site generally consist of approximately 10 feet (ft) of sandy fill overlaying approximately 20 to 30 ft of soft Young Bay Mud (YBM) overlaying denser sands and stiffer clays (e.g., see **Plates 7 and 9** of the main text). Liquefiable sand seams on the order of 5 ft in thickness exist within the YBM (these sands are referred to as YBM Sand herein). Bedrock at the project site is expected to exist at depths greater than approximately 500 ft (Rodgers and Figuers, 1991). Idealization of subsurface conditions for the seismic hazard assessment was based primarily on data from geotechnical borings (including standard penetration test [SPT] and laboratory test data) and cone penetration test (CPT) soundings performed at the project site. Locations of the project explorations and interpreted cross sections are shown on **Plate 3** of the main text.

Free-field site response analyses were performed for a one-dimensional soil column extending from the ground surface to the base of the YBM. The denser sands and stiffer clays underlying the YBM are considered competent (Site Class D), and consequently their effect on seismic wave propagation at the site is captured reasonably well by the ground motion models used in the seismic hazard assessment.

G.2.1 Shear Wave Velocity

The time-weighted average shear wave velocity (Vs) in the top 100 ft (30 meters [m]) (Vs30) is an important input parameter to include the local site conditions in the seismic hazard assessment.

Similarly, characterization of the small-strain stiffness, G (where $G = \rho V_s^2$ and ρ is density) is important for site response analysis. In-situ Vs measurements were conducted by Gregg Drilling and Testing for the seismic CPT-07 located between the building footprint and the Lake Merritt Channel (2020-CPT-07 on **Plate 3** of the main text; data presented in **Appendix A**). These measurements are shown on **Figure G.2-1** alongside Vs values calculated from empirical correlations between Vs and CPT data using the same CPT sounding. Two CPT-based shear wave velocity correlations are shown on **Figure G.2-1**; the Mayne and Rix (1995) correlation for clays is shown within the YBM and the Andrus et al. (2007) correlation is shown for all other strata. The correlations are consistent with the seismic measurements for this CPT sounding in the YBM and competent clays and sands underlying the YBM. Strata demarcations for CPT-07 consistent with the interpreted cross sections (e.g., **Plate 8** of the main text) are also shown on this figure. **Figure G.2-2** shows correlated Vs values for all project CPT soundings, where Mayne and Rix (1995) is shown for YBM and Andrus et al. (2007) is shown for all other strata. This range of data approximately represents the variability of Vs across the site. The relatively small range of correlated Vs values in YBM across all CPT soundings is similar to the range of measured values for CPT-07. Idealized shear wave velocities within the YBM and competent sands and clays underlying the YBM are shown on **Figure G.2-3**. Measured and correlated Vs values for CPT-07 are also shown on this figure. Extrapolation of shear wave velocities in the competent soils underlying the YBM was based on review of data from (1) local Fugro projects and (2) near the former Cypress Structure (Rogers and Figuers, 1991). A Vs30 from the base of the YBM of approximately 860 ft/s (260 m/s), corresponding to Site Class D per ASCE 7-16, was computed using the idealization shown on **Figure G.2-3** and was used for the seismic hazard assessment to develop input ground motions for the site response analyses. Vs30 from the ground surface was estimated to be approximately 560 ft/s (170 m/s), corresponding to Site Class E per ASCE 7-16; however, Site Class F was assigned because of the presence of potentially liquefiable YBM Sand seams. The Site Class F classification requires that a site response analysis in accordance with ASCE 7-16 Section 21.1 be performed.

G.2.2 Young Bay Mud Undrained Shear Strength

The undrained shear strength (s_u) of YBM was evaluated based on CPT and laboratory test data. YBM undrained shear strengths from (1) unconsolidated undrained (UU) triaxial compression tests, (2) unconfined compression (UC) tests, and (3) CPT measurements (i.e., $s_u = q_{t,net}/N_{kt}$ where $q_{t,net}$ is the net total cone resistance and the cone factor N_{kt} = 20) are shown on **Figure G.2-4**. The CPT data are shown as a hexagonally binned two-dimensional histogram (hexbin). The laboratory test data are biased low (i.e., they fall near the lower bound of the CPT data) likely because of sample disturbance effects. The idealized YBM undrained shear strength used for the site response analyses (i.e., for calibration of the modulus reduction and damping factor [MRDF] constitutive model as described in **Section G.6.1**) is also shown on **Figure G.2-4**.

Note that these are static strengths which were empirically adjusted for rate effects for the site response analyses as described in **Section G.6.1**.

G.2.3 Penetration Resistance for Sand-Like Soils (Fill and YBM Sand)

Penetration resistances in the fill and YBM Sand are summarized on **Figure G.2-5** which plots $(N_1)_{60cs}$ (i.e., equivalent clean sand blow counts corrected to 60% energy ratio and an effective overburden of one atmosphere) versus elevation. Hexbin profiles of correlated $(N_1)_{60cs}$ values from CPT data (per the procedures described by Boulanger and Idriss [2014]) are in good agreement with SPT measurements (shown with triangular markers on **Figure G.2-5**). Blow counts in the saturated YBM Sand are mostly between 9 and 16, whereas blow counts in the fill range from roughly 10 to greater than 30.

G.2.4 Idealized Profiles for One-Dimensional Site Response Analyses

Figure G.2-6 shows three idealized soil profiles used for the site response analyses. These profiles reasonably represent the expected stratigraphic variation beneath the building footprint (note that deeper YBM was encountered closer to the Lake Merritt Channel, outside of the building footprint, e.g., 2020-CPT-06 on **Plate 7**). The three idealized profiles are described below.

- **Profile P1** (deep YBM) consists of 10 ft of fill overlaying 31 ft of YBM.
- **Profile P2** (deep YBM with liquefiable sand) consists of 10 ft of fill overlaying 31 ft of YBM with a 5-foot-thick liquefiable YBM Sand layer within the YBM from depths of 25 to 30 ft.
- **Profile P3** (shallow YBM) consists 10 ft of fill overlaying 18 ft of YBM.

G.3 Probabilistic Seismic Hazard Analysis

A site-specific seismic hazard assessment was conducted for a Vs30 of 860 ft/s (260 m/s) corresponding to the base of the YBM, to calculate the input design ground motions for the site response analyses.

G.3.1 Project Location

A Probabilistic Seismic Hazard Analysis (PSHA) was conducted for one representative location of the project site. The geographical coordinates of the location used for the seismic hazard analyses are tabulated in **Table G.1**.

Table G.1: Representative Project Location Coordinates used in the PSHA

Latitude	Longitude
37.7948°N	122.2624°W

G.3.2 Methodology

PSHA Framework

The methodology for a PSHA includes the following components:

1. Seismic Source Model. This includes defining the location, style, and rates of earthquake occurrence in the model area. The characterization includes developing values for the following seismic source parameters:

   i. Source location and geometry. All major active faults and seismotectonic provinces are defined within the model area. This includes the geographical extent at the surface as well as the orientation and depth of the source zones.
   ii. Source type (e.g., shallow crustal area source zones, fault sources, subduction zones, etc.) and style of faulting (e.g., normal, strike-slip, reverse, etc.).
   iii. Magnitude potential (i.e., range of earthquake sizes possible on each source) and magnitude distribution (i.e., characterized using a magnitude probability density function).
   iv. Earthquake magnitude recurrence, which is a characterization of the annual rate at which earthquakes of a specified magnitude or greater occur in each source.

2. Ground Motion Model. Characterization of ground motion attenuation characteristics of each source are based on the geologic and tectonic environment. These characteristics are described by a series of ground motion models, or GMM (also known as "attenuation relationships," "attenuation models," or "ground motion prediction equations").
3. Probabilistic Seismic Hazard Analysis. A PSHA uses inputs from the seismic source model and GMMs selected for the specific environment, to estimate the ground motion hazard at the site. The hazard is expressed in terms of the annual frequency of exceeding a given spectral acceleration at the project site (i.e., annual hazard curves). This information also can be shown in the form of uniform hazard response spectra (UHRS), which correspond to spectral acceleration having the same probability of exceedance across all structural periods. The UHRS are typically used by different design codes to define the design response spectra.

PSHA Calculation

Computation of the seismic hazard involves the combination of uncertainties in earthquake size, location, frequency, and resulting ground motions. The estimated annual rate at which the ground motion, A, will exceed a particular value, a, is computed by (Cornell, 1968):

$$\lambda[A > a] = \sum_{i=1}^{Nsource} N(M_{\min}) \iint P[A > a | m, r]\; f_M(m)\; f_R(r)\, dm\, dr$$

Equation 1

where N_{source} is the total number of seismic sources; $N(M_{min})$ is the annual rate of earthquake with magnitude greater than or equal to M_{min}; $P[A > a|m,r]$ is the probability of the ground motion, A, exceeding the threshold value, a, given the earthquake magnitude and distance from the seismic source; and $f_M(m)$ and $f_R(r)$ are probability density functions describing magnitude and distance.

The computation of this integral is carried out numerically. By assuming that earthquake occurrence can be modeled as a Poisson process, the probability of exceedance in a specified exposure period (typically corresponding to the useful life of a project) may be estimated as follows:

$$P[A > a,t] = 1 - e^{-[\lambda(a)t]}$$

Equation 2

where $P[A > a, \mathrm{t}]$ is the conditional probability of the spectral acceleration (A) exceeding a specified acceleration (a) during a time interval (t) given that an earthquake will occur, and $\lambda(a)$ is the mean annual rate of exceedance of the specified acceleration level.

Seismic Source Model

The PSHA was conducted using the seismic source model adopted by the UGSG to develop the 2014 NSHMP (Petersen et al., 2014) for California which corresponds to the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3). The details of this seismic source model can be found in Field et al. (2013).

Empirical Ground Motion Models

The attenuation of seismic waves from a seismic source were modeled using empirical ground motion models (GMM's). These empirical GMM's should model the type of rupture mechanism as well as the regional geology to properly estimate site-specific strong ground motion parameters. Four of the Next Generation Attenuation (NGA) West 2 GMM's (Bozorgnia et al., 2014) were used. These four NGA West 2 GMM are: Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014). The four NGA West 2 GMM's were equally weighted, following the weighting scheme used in the development of the 2014 USGS NSHMP (Petersen et al., 2014).

Implementation

The PSHA was performed using the USGS computer code *nshmp-haz*, which has been used by the USGS to develop the US national seismic hazard maps.

G.3.3 Results from the PSHA

Figure G.3-1 shows the mean annual seismic hazard curves for selected spectral periods ranging from 0.01 to 10 seconds for a Vs30 of 260 m/sec. A spectral period of 0.01 seconds is used to represent the peak ground acceleration (PGA). These hazard curves represent the total mean hazard from combining all seismic sources and ground motion models. This figure also indicates the annual frequency of exceedance corresponding to a return period of 2,475 years.

Table G.2 tabulates the mean magnitude, distance, and epsilon calculated from the seismic hazard deaggregation for PGA and Sa (spectral acceleration) at 1 second for a return period of 2,475 years. Epsilon is the number of standard deviations that the estimated ground motion amplitude deviates from the estimated median ground motion amplitude. Thus, an epsilon of 1 indicates that the probabilistic value of the ground motion corresponds to a median plus one-standard-deviation value.

Table G.2: Mean Seismic Hazard Deaggregation for a Return Period of 2,475 years and Vs30 of 260 m/sec

	PGA	Sa at 1 sec.
Mean Magnitude (Mw)	7.00	7.27
Mean Distance (km)	9.2	10.0
Mean Epsilon	1.8	1.7

Figure G.3-2 presents the 5 percent-damped mean horizontal UHRS for a return period of 2,475 years and a Vs30 of 260 m/sec. **Table G.3** tabulates the mean horizontal UHRS for periods ranging from 0.01 (i.e., PGA) to 10 seconds for a return period of 2,475 years.

Table G.3: Mean Horizontal UHRS for Return Period of 2,475 Years and a Vs30 of 260 m/sec, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)
0.01 (PGA)	0.933
0.03	0.957
0.05	1.07
0.075	1.32
0.1	1.55
0.15	1.83
0.2	2.05
0.25	2.23
0.3	2.36
0.4	2.42
0.5	2.35
0.75	1.96
1	1.65
1.5	1.19
2	0.924
3	0.606
4	0.429
5	0.320
7.5	0.177
10	0.110

G.4 Design Response Spectra at Base of YBM

According to ASCE 7-16, for Site Class D sites with S1 (mapped 5% damped spectral response acceleration parameter at a period of 1 second) greater than or equal to 0.2 g, the design response spectrum and design acceleration parameters should be developed following the site-specific ground motion procedures defined in Section 21.2 of ASCE 7-16. The S1 for the project site was calculated as 0.660 g using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). Therefore, the design ground motions for the site should be calculated using the site-specific procedures from ASCE 7-16.

ASCE 7-16 defines a site-specific Risk-Targeted Maximum Considered Earthquake (MCE_R) as the lesser of probabilistic (MCE_R) and deterministic (MCE_R) ground motions. The probabilistic MCE_R ground motion is calculated as the ground motion in the direction of maximum horizontal response that is expected to achieve 1 percent probability of collapse within a 50-year period. The deterministic MCE_R ground motion is defined as the 84th percentile ground motion in the direction of maximum horizontal response of the largest acceleration from deterministic seismic hazard analysis (DSHA) of the characteristic earthquakes on all known active faults within the project region. Additionally, ASCE 7-16 specifies a lower limit to the deterministic MCE_R ground motion. The site-specific MCE_R should not be less than 150 percent of the site-specific design response spectrum. The site-specific design response spectrum is calculated as 2/3 of the site-specific MCE_R. The site-specific design response spectrum should be greater than or equal to 80 percent of the spectral acceleration as determined by using the general response spectrum of Section 11.4.6 of ASCE 7-16, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16.

The PSHA results described in the previous section were used to calculate the probabilistic MCE_R spectrum. As specified in ASCE 7-16, to obtain ground motions with a uniform 1 percent probability of collapse within a 50-year period, the UHRS for a return period of 2,475 was scaled by a risk coefficient, C_R. The C_R values were calculated using Method 1 described in Chapter 21 of ASCE 7-16. The mapped risk coefficients at spectral periods of 0.2 and 1.0 sec, C_{RS} and C_{R1}, respectively, were determined using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). The value of these risk coefficients C_{RS} and C_{R1} are 0.921 and 0.906, respectively. The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-16. **Figure G.4-1** shows the UHRS for a return period of 2,475 years along with the probabilistic MCE_R response spectrum.

The deterministic MCE_R spectrum was calculated by performing a DSHA in EZ-FRISK™ (Fugro, 2019) using the same seismic sources and GMM's used in the PSHA. The UCERF3 source model includes magnitude frequency distributions (MFD's) which relate frequency of occurrence to earthquake magnitude; however, these MFD's include multi-fault ruptures scenarios with large magnitudes but with low probability of occurrence. Therefore, following the current USGS approach to calculate deterministic ground motions from the UCERF3 source model, to estimate the characteristic magnitude for the seismic sources, we used the empirical relationships proposed by Wells and Coppersmith (1994) that relates rupture geometry to earthquake magnitude. The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-16. **Figure G.4-1** illustrates the calculation of the deterministic MCE_R response spectrum. The deterministic MCE_R response spectrum was calculated as the maximum of the 84th DSHA response spectrum and the lower limit specified by ASCE 7-16 Supplement 1 calculated for a Site Class D.

Figure G.4-2 presents the development of the site-specific MCE_R and design response spectra for the base of the YBM. In this case, the deterministic MCE_R spectrum is lower than the probabilistic MCE_R spectrum for all spectral periods. The site-specific MCE_R spectrum is the maximum of: 1) the minimum of the probabilistic and deterministic MCE_R, and 2) 150 percent of the design response spectrum. Following ASCE 7-16, the design response spectrum was calculated as the maximum of 2/3 of the site-specific MCE_R and the lower limit specified by ASCE 7-16 (80 percent of the general spectrum for Site Class D, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16). The transition period from constant velocity to constant displacement, T_L, required to calculate the lower limit, was estimated as 8 seconds using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html).

Table G.4 tabulates the spectral ordinates of the recommended site-specific MCE_R and design response spectra per ASCE 7-16 for the base of the YBM.

Table G.4: MCE$_R$ and Design Response Spectra per ASCE 7-16 for a Vs30 of 260 m/sec (base of YBM), 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)									
	UHRS for Return Period of 2,475 Years	Risk Coefficients	Max. Direction Scaling Factors	Probabilistic MCE$_R$	84th Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE$_R$	Site-Specific MCE$_R$	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.933	0.921	1.10	0.945	0.711	0.555	0.782	0.782	0.400	0.521
0.03	0.957	0.921	1.10	0.970	0.717	0.559	0.789	0.789	0.459	0.526
0.05	1.07	0.921	1.10	1.08	0.783	0.611	0.861	0.861	0.518	0.574
0.075	1.32	0.921	1.10	1.34	0.928	0.724	1.02	1.02	0.591	0.680
0.1	1.55	0.921	1.10	1.57	1.07	0.831	1.17	1.17	0.664	0.781
0.15	1.83	0.921	1.10	1.86	1.29	1.01	1.42	1.42	0.811	0.946
0.190	2.01	0.921	1.10	2.04	1.42	1.11	1.56	1.56	0.927	1.04
0.2	2.05	0.921	1.10	2.08	1.45	1.13	1.60	1.60	0.927	1.06
0.25	2.23	0.919	1.13	2.32	1.57	1.26	1.77	1.77	0.927	1.18
0.3	2.36	0.917	1.15	2.49	1.66	1.36	1.91	1.91	0.927	1.28
0.4	2.42	0.915	1.19	2.62	1.75	1.48	2.08	2.08	0.927	1.39
0.5	2.35	0.912	1.21	2.61	1.74	1.50	2.11	2.11	0.927	1.41
0.75	1.96	0.909	1.26	2.25	1.50	1.34	1.89	1.89	0.927	1.26
0.949	1.70	0.906	1.29	2.00	1.33	1.22	1.73	1.73	0.927	1.15
1	1.65	0.906	1.30	1.95	1.30	1.20	1.69	1.69	0.880	1.13
1.5	1.19	0.906	1.35	1.46	0.983	0.942	1.33	1.33	0.587	0.885
2	0.924	0.906	1.39	1.16	0.783	0.770	1.09	1.09	0.440	0.724
3	0.606	0.906	1.44	0.789	0.538	0.548	0.773	0.773	0.293	0.515
4	0.429	0.906	1.47	0.572	0.383	0.400	0.564	0.564	0.220	0.376
5	0.320	0.906	1.50	0.435	0.283	0.301	0.425	0.425	0.176	0.283
7.5	0.177	0.906	1.50	0.240	0.140	0.149	0.210	0.210	0.117	0.140
8	0.159	0.906	1.50	0.216	0.124	0.131	0.185	0.185	0.110	0.124
10	0.110	0.906	1.50	0.150	0.0801	0.0852	0.120	0.120	0.0704	0.0801

G.5 Ground Motion Acceleration Time Histories for Input to Site Response Analyses

G.5.1 Selection of Seed Ground Motions

Following Section 21.1.1 of ASCE 7-16, five pairs of orthogonal recorded horizontal seed ground motion (GM's) acceleration time histories were selected and scaled to comply with the site-specific MCE_R response spectrum at the base of the YBM developed in the previous section. During the selection of seed GM's, we considered the following criteria:

- The selected GM's were recorded from seismic events that are comparable with events that control the MCE_R scenario from the seismic deaggregation.
- The shape of the GM's acceleration response spectra.
- The lowest usable frequency of the selected GM's.
- Other criteria including strong motion duration, Arias Intensity, faulting mechanism, and shear wave velocity at the site where the GM's were recorded.

Table G.5 lists the properties of the selected seed GM's.

Table G.5: Selected Seed Ground Motions

No.	Record Sequence Number (RSN)	Earthquake Name	Recording Station	Moment Magnitude (Mw)	Faulting Mechanism	Vs30 of Recording Station (m/s)	Rupture/ Closest Distance (km)	Minimum Usable Frequency (Hz)	Average Scaling Factor
1	729	1987 Superstition Hills-02	Imperial Valley Wildlife Liquefaction Array	6.54	Strike slip	179	24	0.1	4.1
2	1545	199 Chi-Chi_ Taiwan	TCU120	7.62	Reverse Oblique	459	7.4	0.0375	4.1
3	6952	2010 Darfield_ New Zealand	Papanui High School	7	Strike slip	263	19	0.0625	4.0
4	806	1989 Loma Prieta	Sunnyvale - Colton Ave.	6.93	Reverse Oblique	268	24	0.1	4.4
5	1176	1999 Kocaeli_ Turkey	Yarimca	7.51	Strike slip	297	5	0.0875	3.3

G.5.2 Scaling of Seed Ground Motions

Figure G.5-1 shows a comparison between the response spectra of the two components (H1, H2) for each of the linearly scaled ground motions (thin colored lines), the mean response spectra of the five scaled motions (thick red line) and the target MCE_R at the base of the YBM (thick black line). On average, the mean of the scaled acceleration response spectra shows good agreement with the target response spectrum.

The scale factor for each of the seed ground motions was selected such that the average of their spectral accelerations within the period range from 0.05 seconds to 5 seconds matches, on average, the spectral accelerations of the target MCE_R response spectrum within the same period range. The average scaling factor for the response spectra of the two components of the seed ground motions is listed in **Table G.5** above.

G.6 One-Dimensional Site Response Analyses

According to ASCE 7-16, for sites classified as Site Class F, the design response spectrum and design acceleration parameters should be developed following the site-specific ground motion procedures defined in Chapter 21 of ASCE 716. Specifically, site response analyses shall be performed in accordance with ASCE 7-16 Section 21.1. The approach, analyses, and results for one-dimensional free-field site response analyses are presented herein.

G.6.1 Approach

One-Dimensional Site Response Modelling in FLAC

One-dimensional site response analyses were performed using the commercial finite difference program FLAC (Fast Analysis of Continua) (Itasca, 2016). One-dimensional site response was modeled with a single column of 2.5-foot square zones. Analyses were performed for the three idealized profiles shown on **Figure G.2-6**. The water table was modeled at the base of the fill for all profiles. Analyses were performed using the user defined constitutive models MRDF (modulus reduction and damping factor hysteretic model, Hashash et al., 2010) and PM4Sand (Boulanger and Ziotopoulou, 2017). MRDF was used to model the fill and YBM, and PM4Sand was used to model the liquefiable, saturated YBM Sand in profile P2. Analyses were performed for each of the 10 scaled ground motion time histories (5 ground motion records, 2 components) developed in the previous section.

For dynamic simulation, a quiet (absorbing) boundary was used at the base of the model and the lateral boundaries were attached (i.e., at a given elevation the left and right nodes displace together). A single elastic zone was included at the base of the model with properties representative of the competent soils underlaying the YBM (i.e., Vs30 of 860 ft/s). Outcrop ground motions were input at the base of the model (at the quiet boundary) as shear stress time histories. Shear stress time histories were computed from outcrop acceleration time histories by integrating to obtain velocity and multiplying by twice the competent soil density times the competent soil Vs per the compliant base procedure proposed by Mejia and Dawson (2006).

Constitutive Calibration and Input Parameters

The bases for constitutive model calibration and input parameters are summarized in **Table G.6**. YBM shear wave velocity was modeled using the idealization shown on **Figure G.2-3**. Shear wave velocity in the fill and YBM Sand was modeled based on correlation to SPT blow count. Representative $(N_1)_{60cs}$ values of 17 and 12 were used to model the fill and YBM Sand, respectively. These $(N_1)_{60cs}$ values correspond to V_{s1} = 586 ft/s in the fill (i.e., V_s ranges from about 300 to 500 ft/s in the fil) and V_{s1} = 544 ft/s in the YBM Sand (i.e., V_s of about 550 ft/s in the YBM Sand).

Target empirical shear modulus reduction (G/G_{max}) and material damping relationships are summarized in **Table G.6**. In general, the degree to which the target relationships are represented by the calibrated models depends on the model (i.e., MRDF vs. PM4Sand) and the calibration procedure. For MRDF, fitting parameters can be selected to produce near exact matches with target shear modulus reduction and damping curves, however, such calibrations may underpredict or overpredict shear strength depending on the small-strain stiffness (G). For site response analyses, the relative importance of matching these behaviors (i.e., empirical G/G_{max} and shear strength) depends on the strain-level of interest and is problem dependent. Soft clays at the project site are expected to develop large shear strains for the MCE_R level of shaking, hence MRDF was calibrated to honor the idealized undrained shear strength profile shown on **Figure G.2-4;** a dynamic multiplier of 1.4 was applied to these idealized strengths to account for strain-rate effects. This was done following the procedure described by Hashash et al. (2010) where G/G_{max} values for shear strains greater than 0.1% are adjusted to achieve the desired shear strength. For PM4Sand primary input parameters were correlated to $(N_1)_{60cs}$ as described by Boulanger and Ziotopoulou (2017); all secondary input parameters used default values. Boulanger and Ziotopoulou (2017) demonstrate reasonable consistency with the EPRI (1993) modulus reduction and damping curves for a range of $(N_1)_{60cs}$ and effective overburden pressures.

Lastly, the PM4Sand contraction rate parameter was calibrated based on $(N_1)_{60cs}$ and the Idriss and Boulanger (2008) SPT-based liquefaction triggering correlation.

Table G.6: Constitutive Model Calibration Basis

Strata	Constitutive Model	Shear wave velocity, Vs	Basis for MRDF Strength	G/G_{max} and Damping Ratio Curve Source(s)
Fill	MRDF	$V_{s1} = 85[(N_1)_{60} + 2.5]^{0.25}$ m/s (Boulanger and Ziotopoulou, 2017)	Bolton (1986) strength-dilatancy relationship for plane strain ($\varphi'_{cv} = 33°$)	EPRI (1993)
YBM	MRDF	V_s = 310 ft/s at 10 ft depth Increasing at 5 ft/s/ft (**Figure G.2-3**)	**Figure G.2-4** with 1.4 dynamic multiplier	Fugro (2007, 2020)
YBM Sand	PM4Sand	$V_{s1} = 85[(N_1)_{60} + 2.5]^{0.25}$ m/s (Boulanger and Ziotopoulou, 2017)	N/A	EPRI (1993)

Verification of Modelling Approach

To verify the FLAC modeling approach (i.e., the numerical platform, application of earthquake loading, MRDF constitutive model implementation, etc.), a subset of analyses was performed using both FLAC and DEEPSOIL (Hashash et al., 2017). Comparisons between FLAC and DEEPSOIL were made for profile P1 for two levels of shaking (the MCE_R and a smaller level of shaking with PGA ≈ 0.45 g). Comparisons of results obtained from the two analysis platforms showed near identical surface response spectra, stress-strain responses, and profiles of maximum shear strain, PGA, and maximum shear stress. The FLAC modelling approach was adopted for all other analyses (including modelling of liquefiable YBM Sand in profile P2), as described in the preceding sections.

G.6.2 Results

Baseline Analyses

Results for one-dimensional site response analyses for profile P1, P2, and P3 are shown on **Figure G.6-1** and **Figure G.6-2**. Profiles of absolute maximum shear strain and PGA are shown on **Figure G.6-1**. The thin lines are for individual ground motions and the thick lines are mean responses per profile. Overall, large shear strains develop in the YBM at the MCE_R level of shaking. Surface response spectra and amplification ratios are shown on **Figure G.6-2**. The amplification ratios were calculated as the ratio of the response spectra at the surface to the input response spectrum. The thin lines show responses for each ground motion time history and the thick lines show mean responses per idealized profile. Overall, there is little variation in the mean surface spectra for the three profiles analyzed. The shorter period (higher frequency) mean responses exhibit significant deamplification, whereas periods greater than approximately three seconds exhibit amplified responses. Yielding in the YBM deamplifies higher frequencies and effectively base isolates the soil column, hence there is little difference in the surface response spectra for the three idealized profiles. For smaller levels of shaking, clear differences in the response of the three profiles is expected.

Figure G.6-3 shows the idealized amplification ratios developed based on the average amplification ratios from the site response analyses. The idealized amplification ratios consider variability on the soil stratigraphy and variability on ground motion time histories. However, sensitivity analyses conducted showed similar amplification ratios by considering variability in soil properties (YBM shear wave velocity and undrained shear strength).

Parametric Analyses

Parametric analyses were performed for profile P1 to evaluate the effect of lower bound YBM shear wave velocities and a range of YBM undrained shear strength idealizations on the site response. Overall, these parameter variations had little effect on the surface spectrum (for the same reasons discussed above). An upper bound undrained shear strength profile caused the most significant change to the surface spectrum, slightly increasing the amplification for periods between about 1.5 to 4 seconds while decreasing the amplification for periods greater than approximately 4 seconds. Even with an upper bound undrained shear strength, large shear strains developed throughout the YBM (mean absolute maximum shear strains were on the order of 10 to 20 percent).

G.7 Design Response Spectra at the Ground Surface

Figure G.7-1 presents the development of the site-specific MCE_R and design response spectra for the ground surface. The MCE_R response spectrum from the site response analyses is calculated as the site-specific MCE_R at the base of the YBM (input to the site response analyses) multiplied by the idealized amplification ratios presented on **Figure G.6-3**. The site-specific MCE_R spectrum is the maximum of: 1) MCE_R response spectrum from the site response analyses, and 2) 150 percent of the design response spectrum. Following ASCE 7-16, the design response spectrum was calculated as the maximum of 2/3 of the site-specific MCE_R and the lower limit specified by ASCE 7-16 (80 percent of the general spectrum for Site Class E, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16). The transition period from constant velocity to constant displacement, T_L, required to calculate the lower limit, was estimated as 8 seconds using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html).

Table G.7 tabulates the spectral ordinates of the recommended site-specific MCE_R and design response spectra per ASCE 7-16 for the ground surface. The corresponding design acceleration parameters S_{MS}, S_{M1}, S_{DS}, and S_{D1} are tabulated in **Table G.8**.

Table G.7: MCE_R and Design Response Spectra per ASCE 7-16 at the Ground Surface, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)		
	Site-Specific MCE_R	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.584	0.389	0.389
0.03	0.639	0.426	0.426
0.05	0.694	0.463	0.463
0.075	0.763	0.508	0.508
0.1	0.831	0.554	0.554
0.15	0.969	0.646	0.646
0.2	1.11	0.738	0.738
0.25	1.24	0.829	0.829
0.3	1.38	0.921	0.921
0.304	1.39	0.927	0.927
0.4	1.39	0.927	0.927
0.5	1.39	0.927	0.927
0.75	1.39	0.927	0.927
1	1.39	0.927	0.927
1.5	1.39	0.927	0.927
1.52	1.39	0.927	0.927
2	1.06	0.704	0.704
3	0.827	0.469	0.551
4	0.733	0.352	0.489
5	0.561	0.282	0.374
7.5	0.282	0.188	0.188
8	0.264	0.176	0.176
10	0.169	0.113	0.113

Table G.8: Design Acceleration Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.39 g
S_{M1}	2.93 g
S_{DS}	0.927 g
S_{D1}	1.96 g

G.8 References

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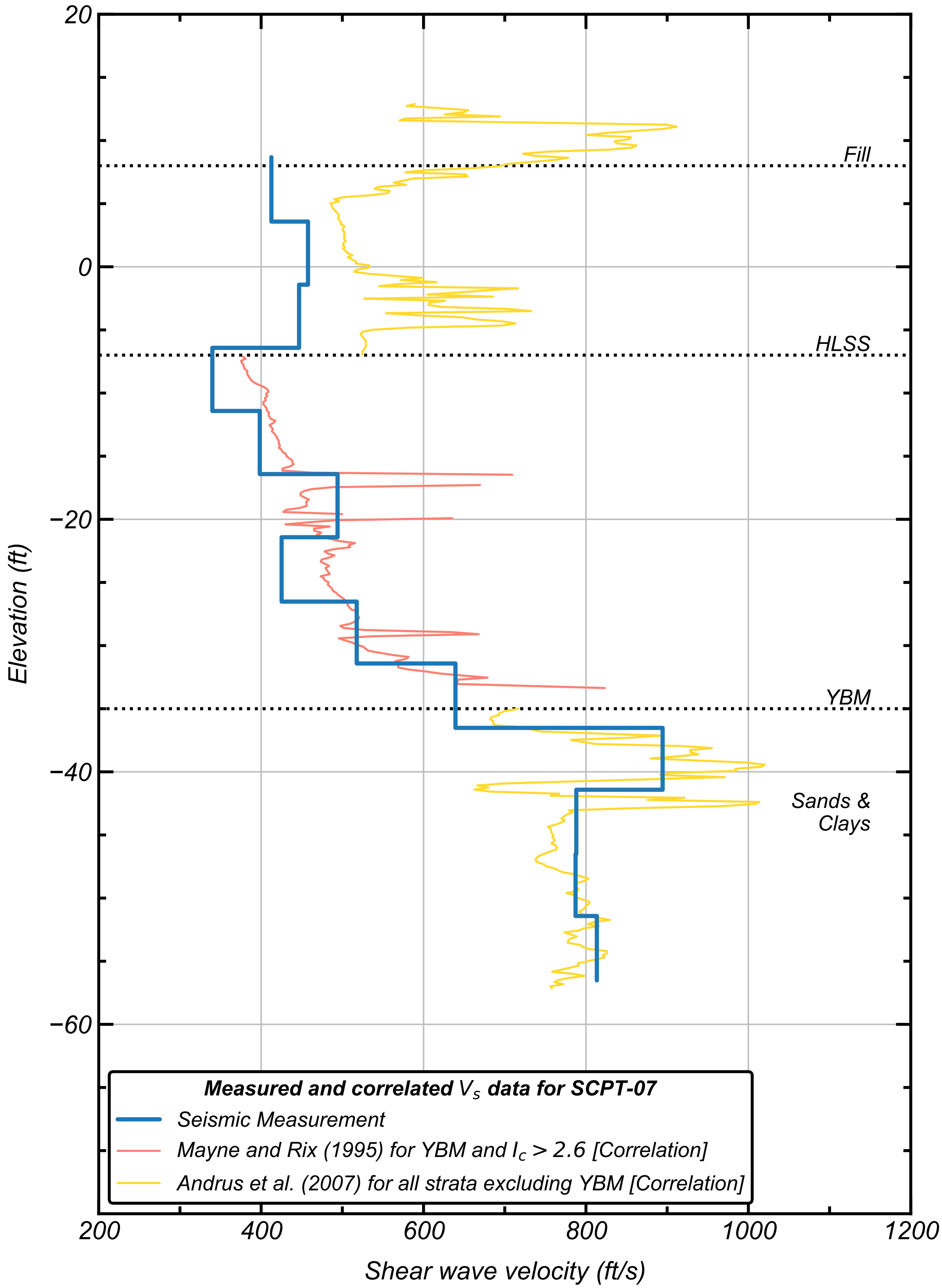

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Figure G.2-1: Measured and Correlated Vs Data for SCPT-07

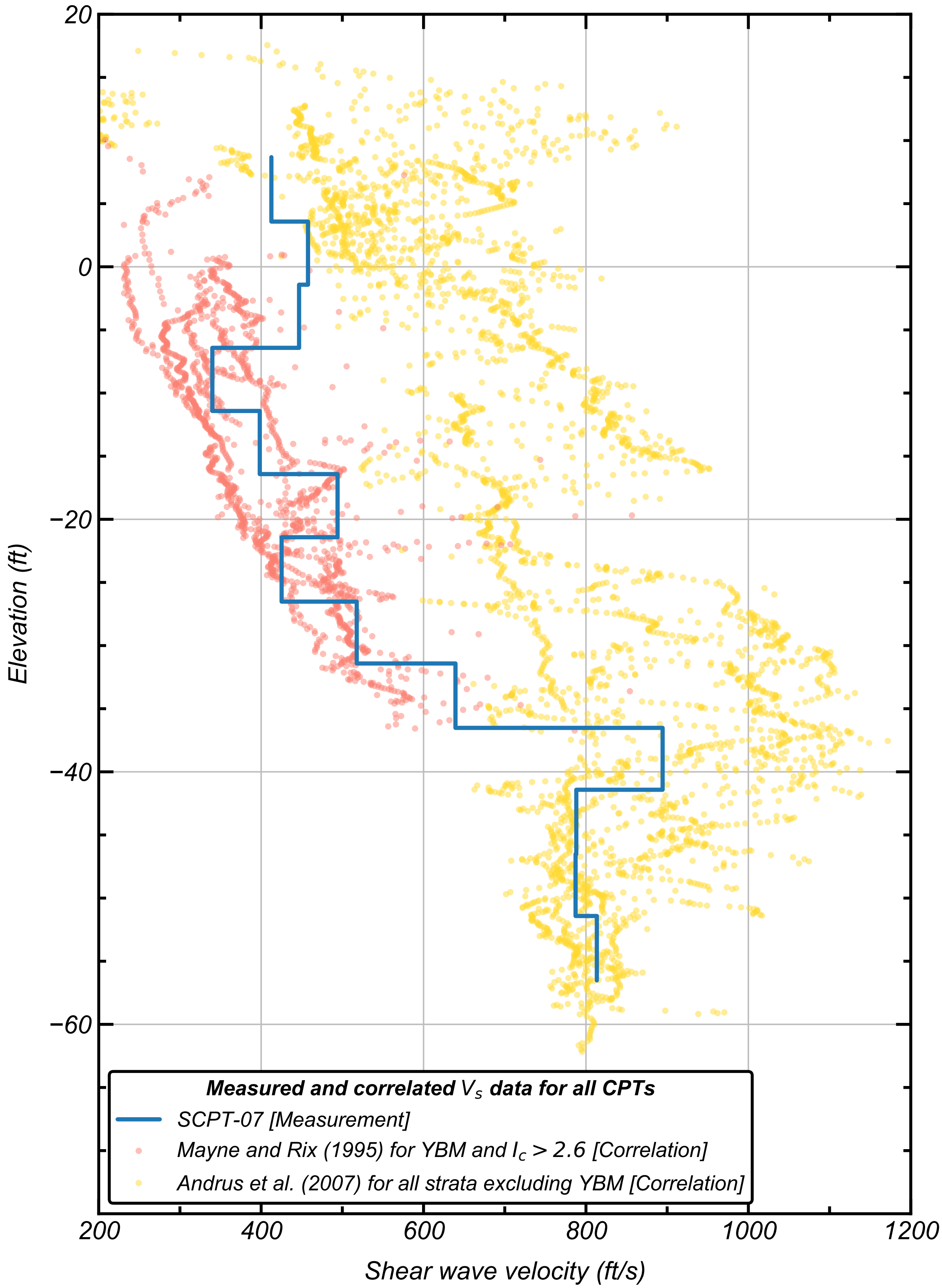


Figure G.2-2: Measured and Correlated Vs Data for All CPTs

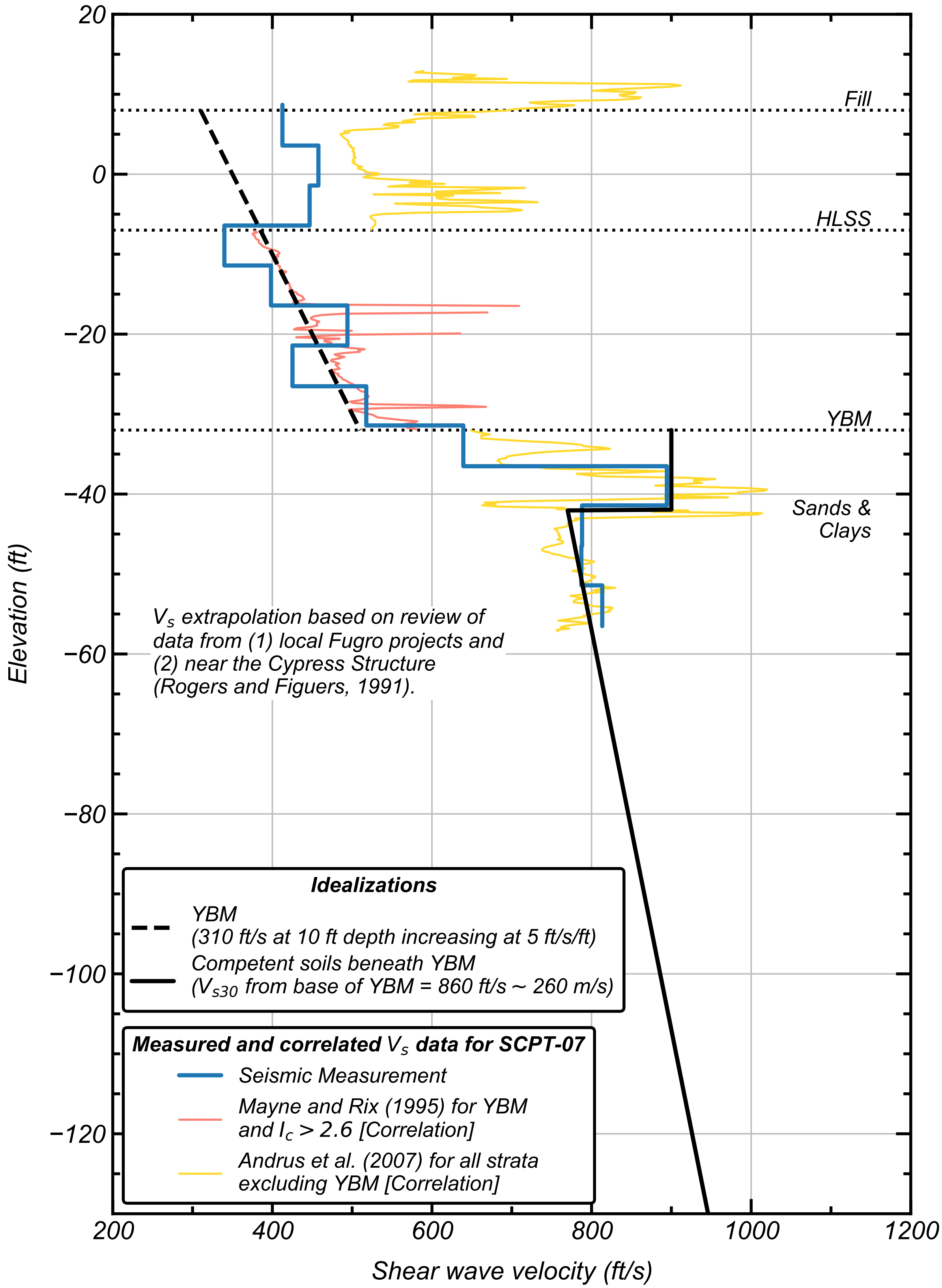


Figure G.2-3: Shear Wave Velocity Idealizations

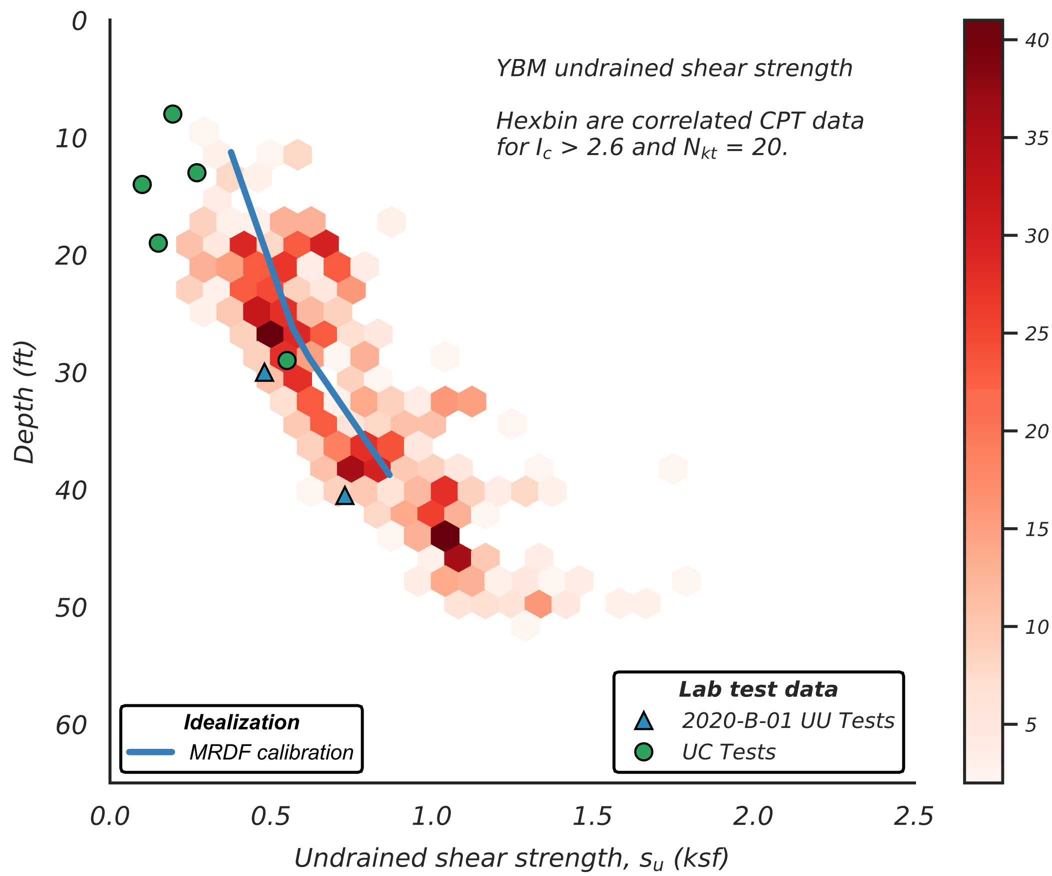


Figure G.2-4: YBM Undrained Shear Strength

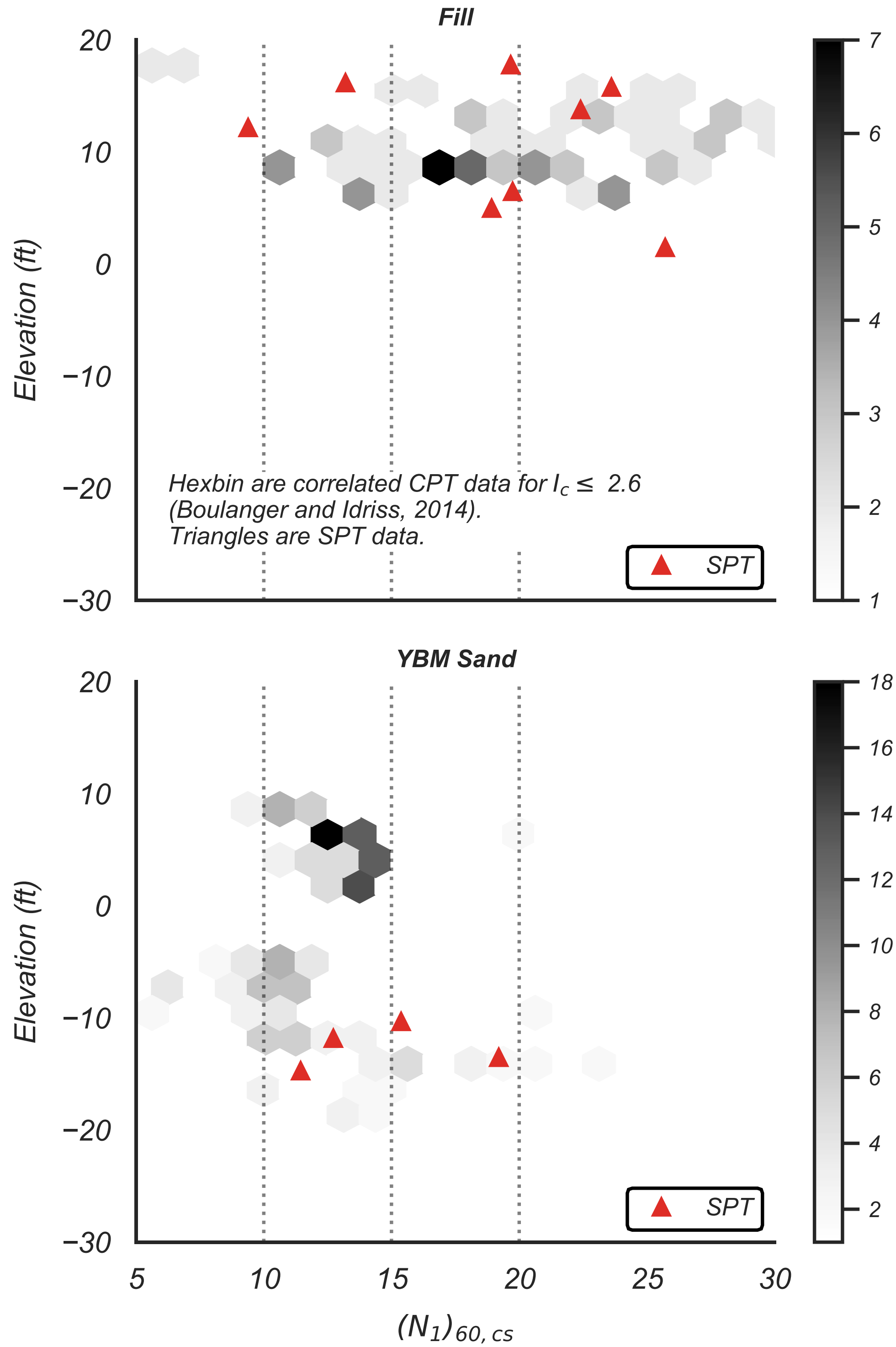


Figure G.2-5: Penetration Resistance (N160cs) vs. Elevation for Fill and YBM Sand

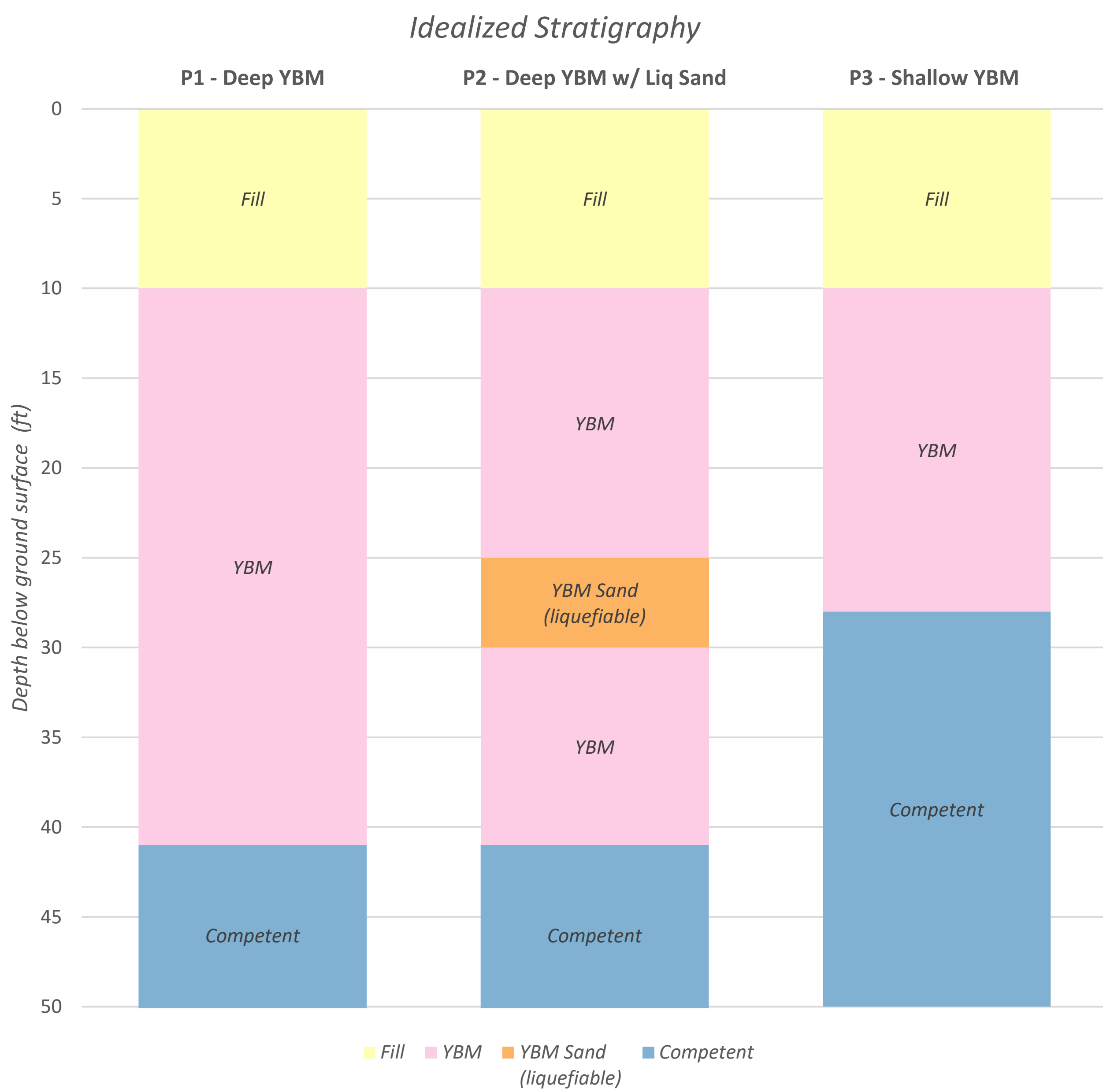


Figure G.2-6: Idealized Stratigraphy for 1D Site Response Analyses

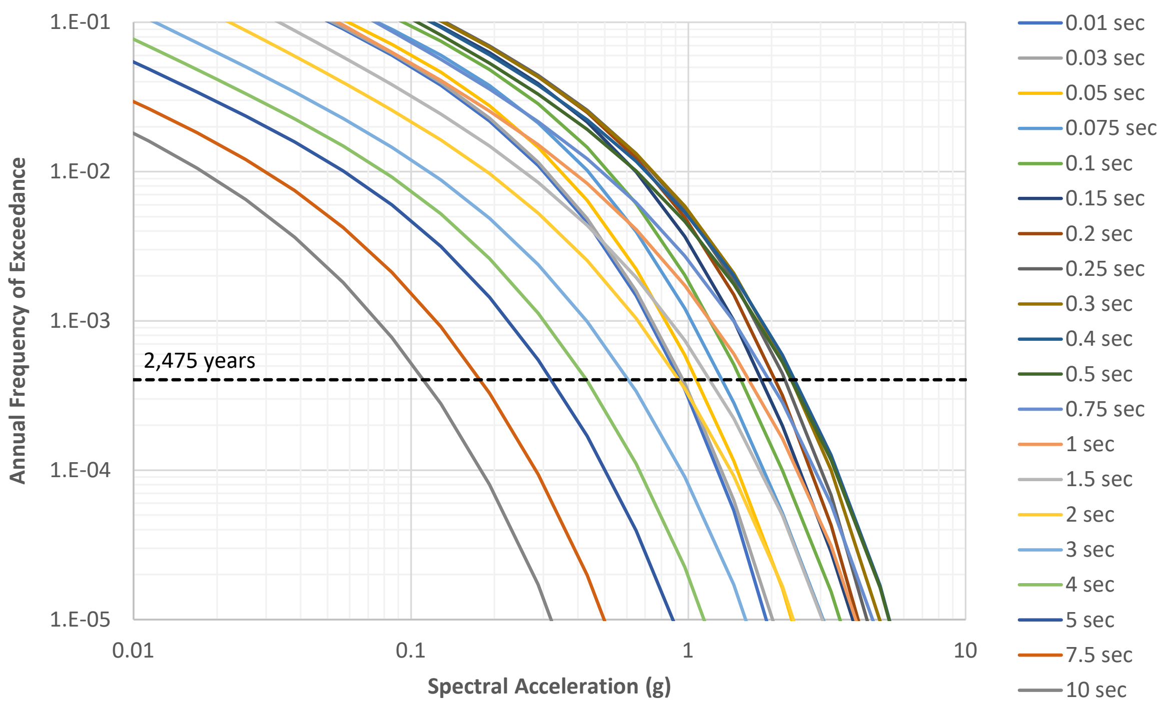


Figure G.3-1: Mean Annual Seismic Hazard Curves for Vs30 of 260 m/s (Base of YBM)

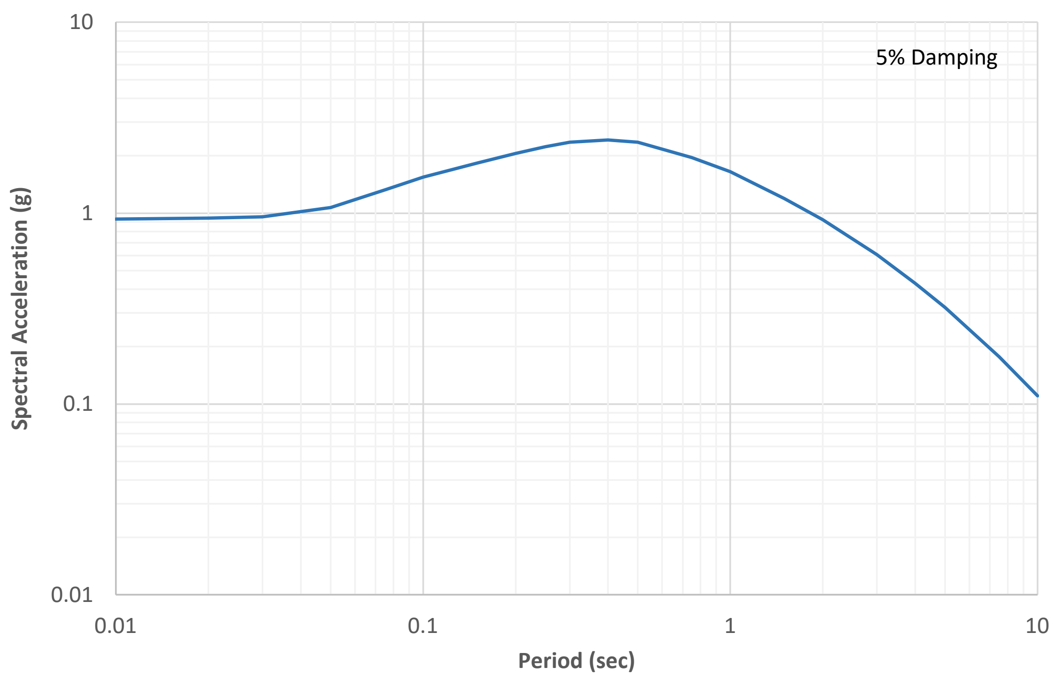
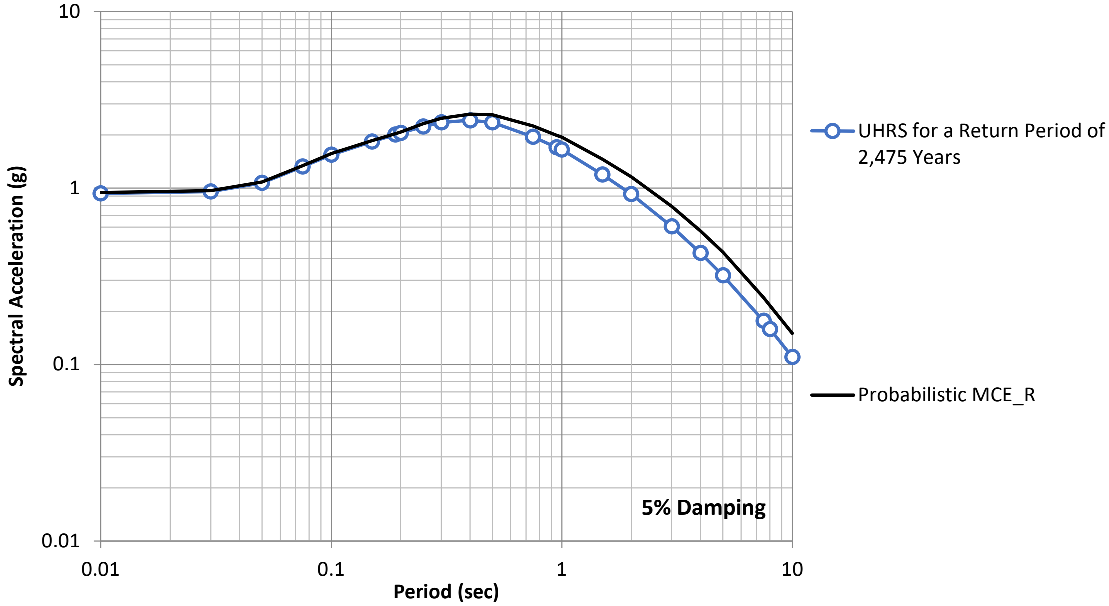


Figure G.3-2: Mean Horizontal Uniform Hazard Response Spectrum for a Return Period of 2,475 Years and Vs30 of 260 m/s (Base of YBM)



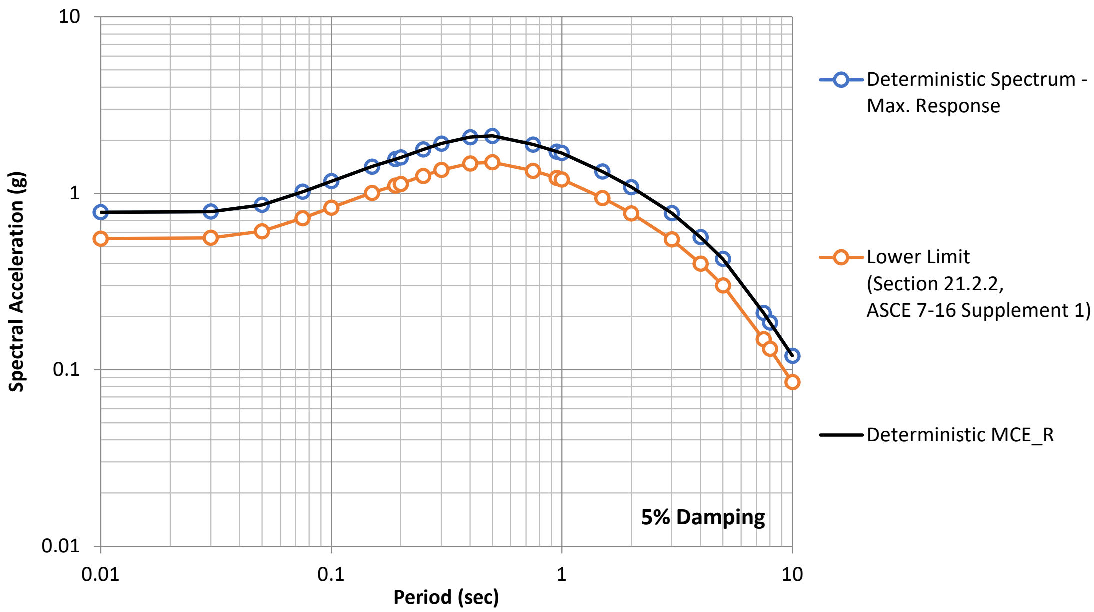
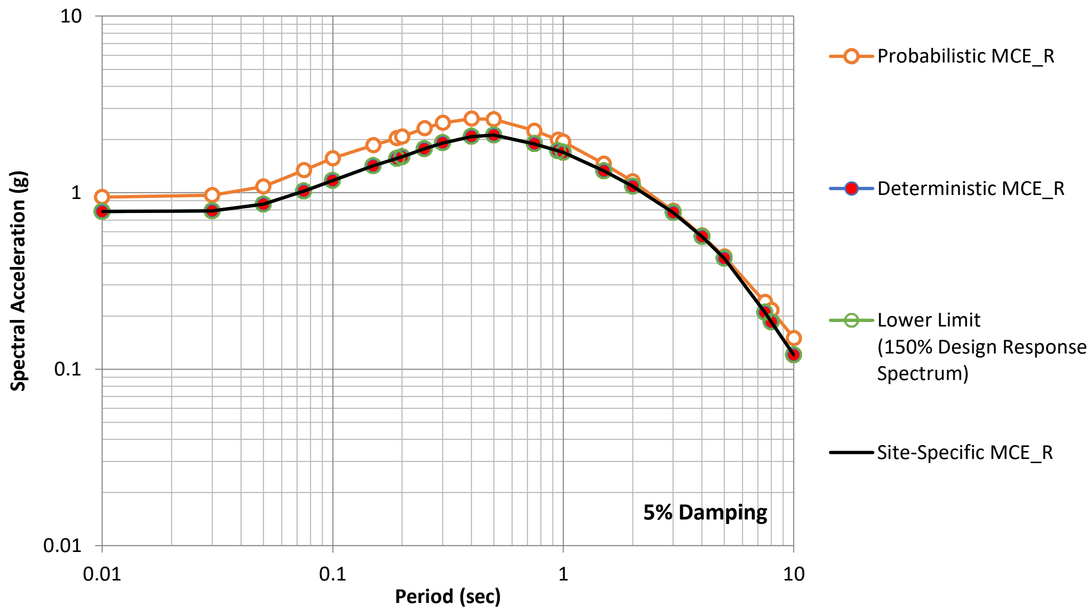


Figure G.4-1: Calculation of the Probabilistic and Deterministic Horizontal MCE_R Response Spectra per ASCE 7-16 for Vs30 of 260 m/s (Base of YBM)



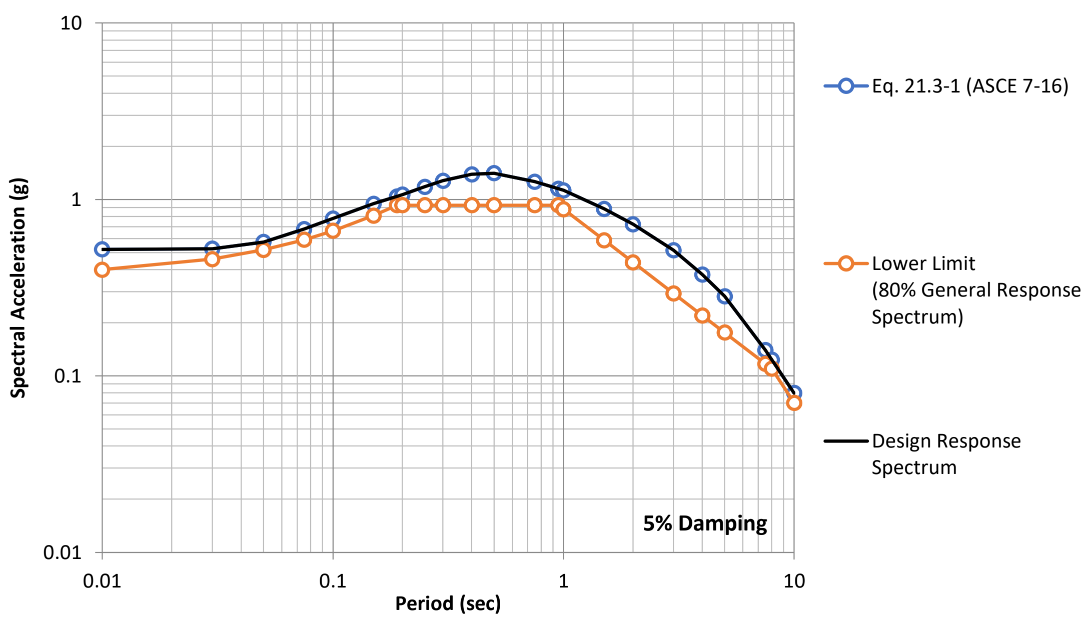


Figure G.4-2: Calculation of the Site-Specific Horizontal MCE_R and Design Response Spectra per ASCE 7-16 for Vs30 of 260 m/s (Base of YBM)

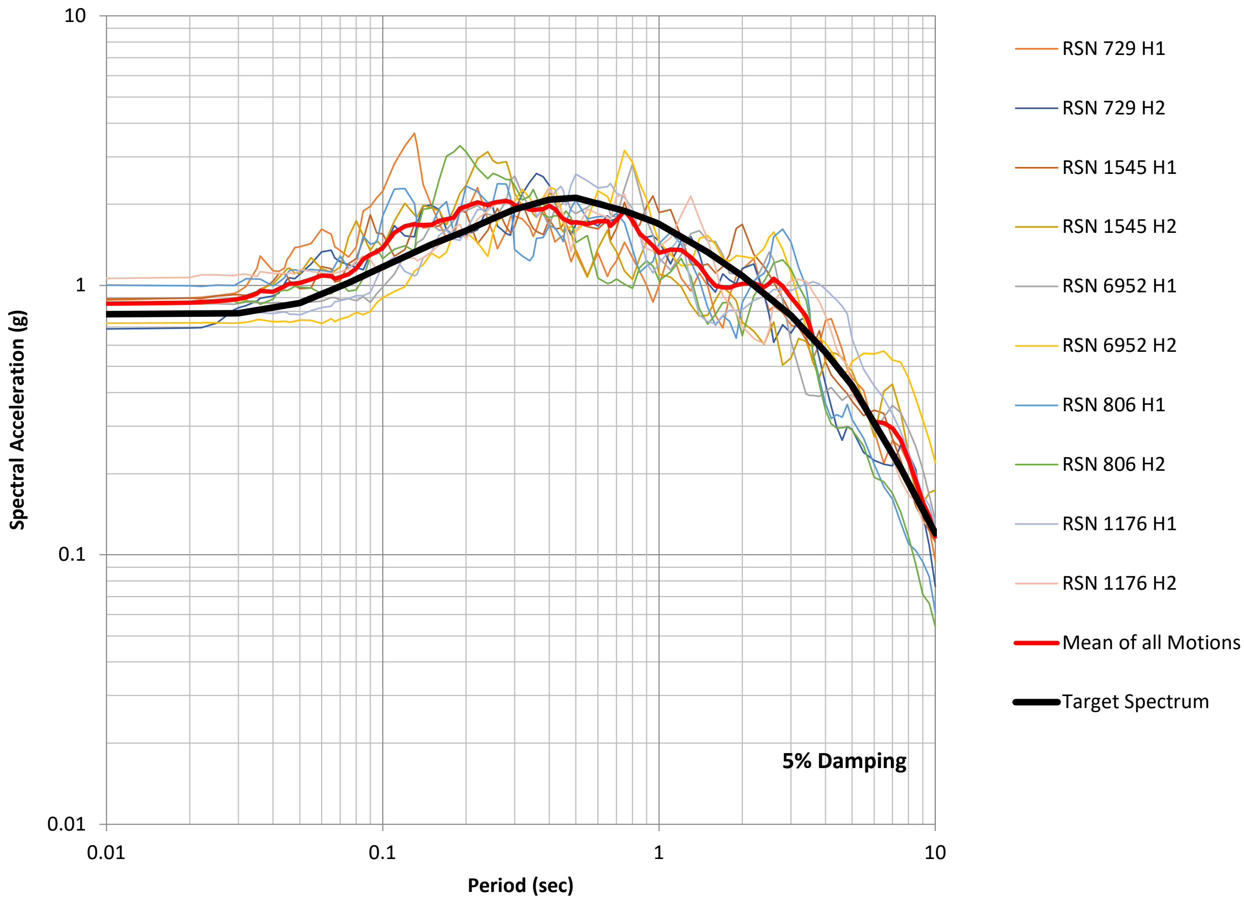
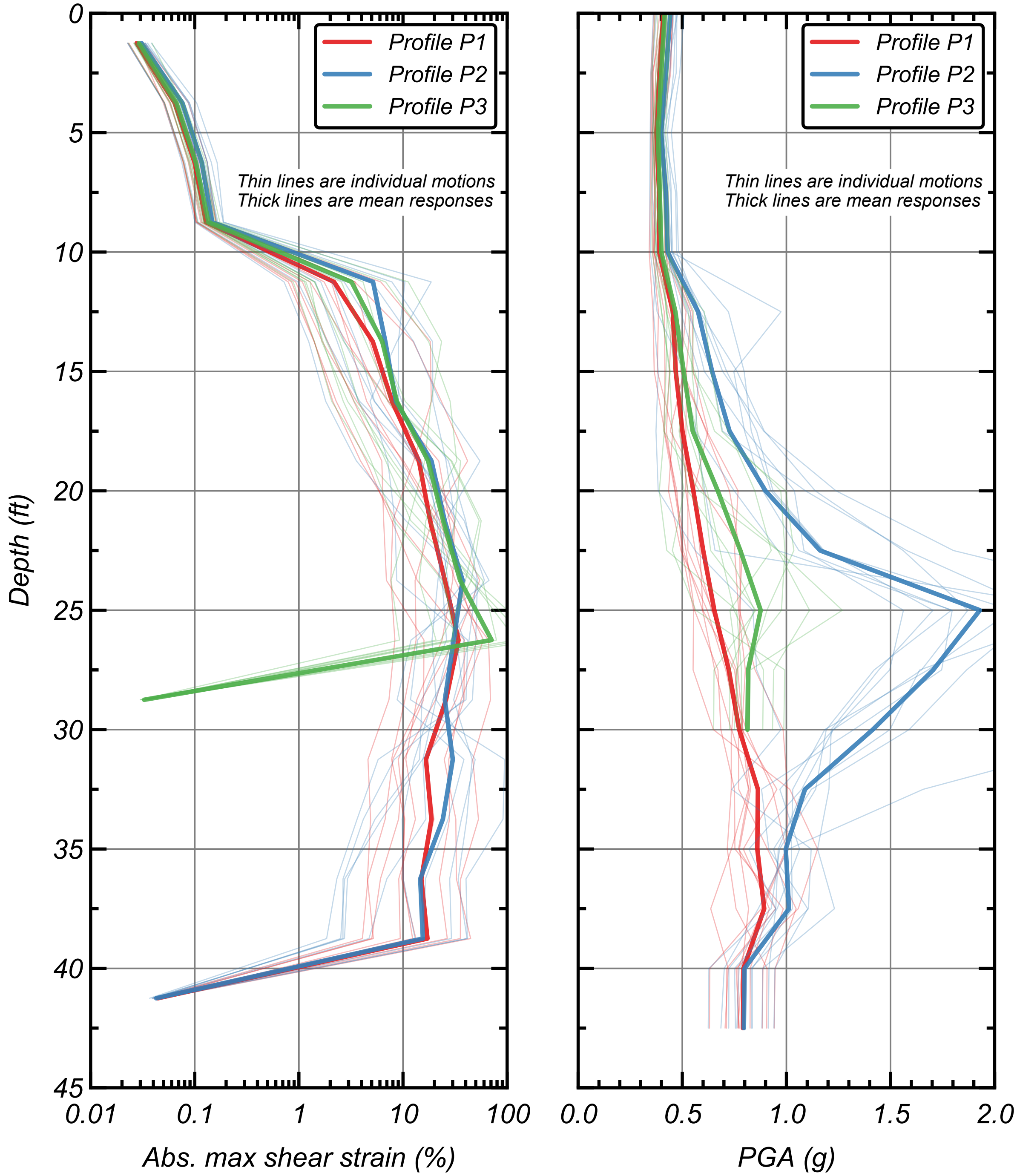


Figure G.5-1: Comparison of Target Response Spectrum (MCE_R), Mean of Scaled Response Spectra and Individual Scaled Ground Motions Response Spectra



//WLCPFUSAFS01/Data/Projects/Location-72/2019/04.72190021 Laney College Library Learning Resource Center/05_Analyses/12_Numerical_Analyses/01_Site_Response_Analyses/01_FLAC/simviz/plots/rev12/flac_1dsr.pdf, 02/28/2020 at 12:57:34

Figure G.6-1: Profiles of Absolute Maximum Shear Strain and PGA from 1D Site Respone Analyses

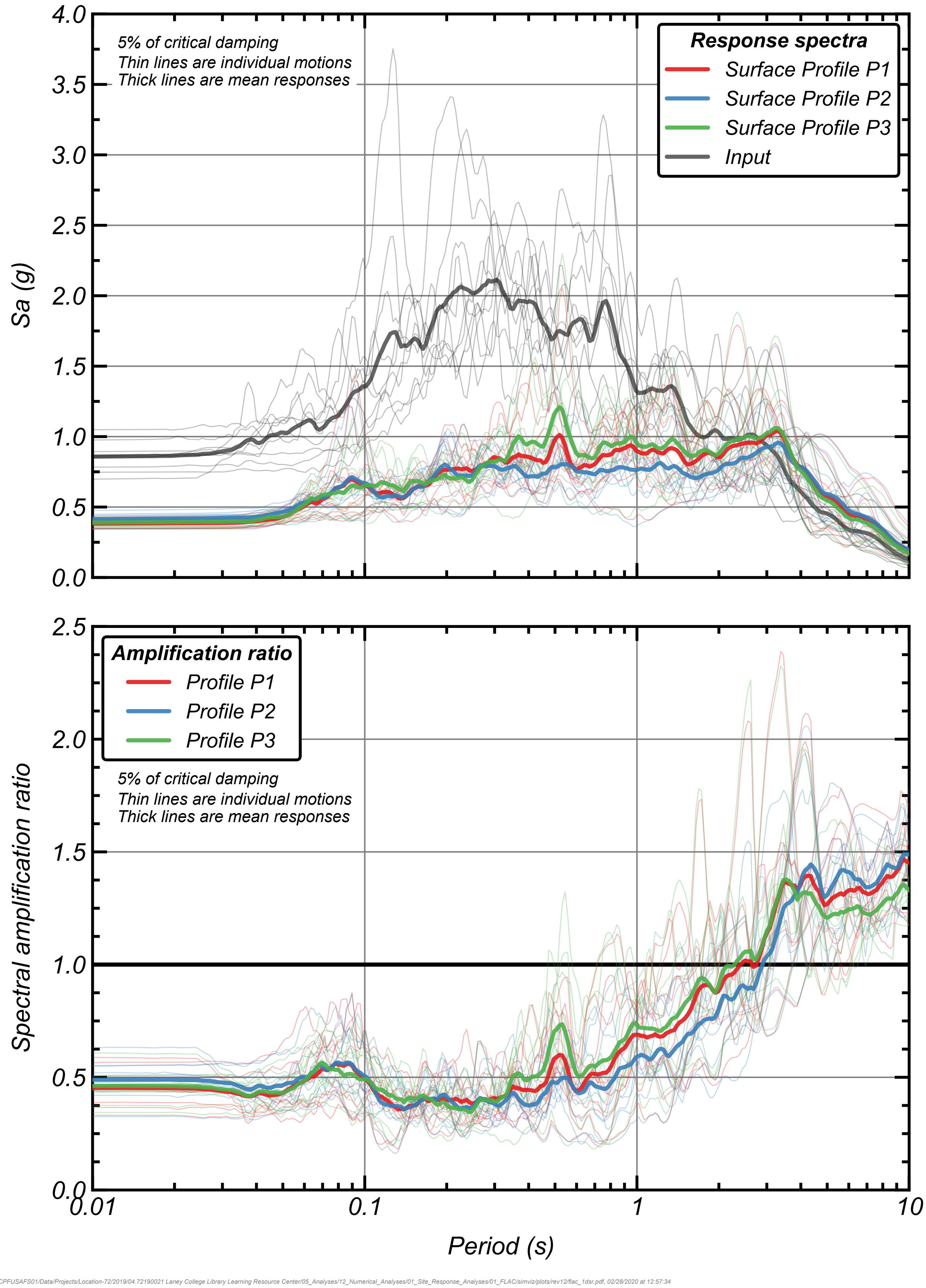


Figure G.6-2: Surface Response Spectra and Amplification Ratios from 1D Site Response Analyses

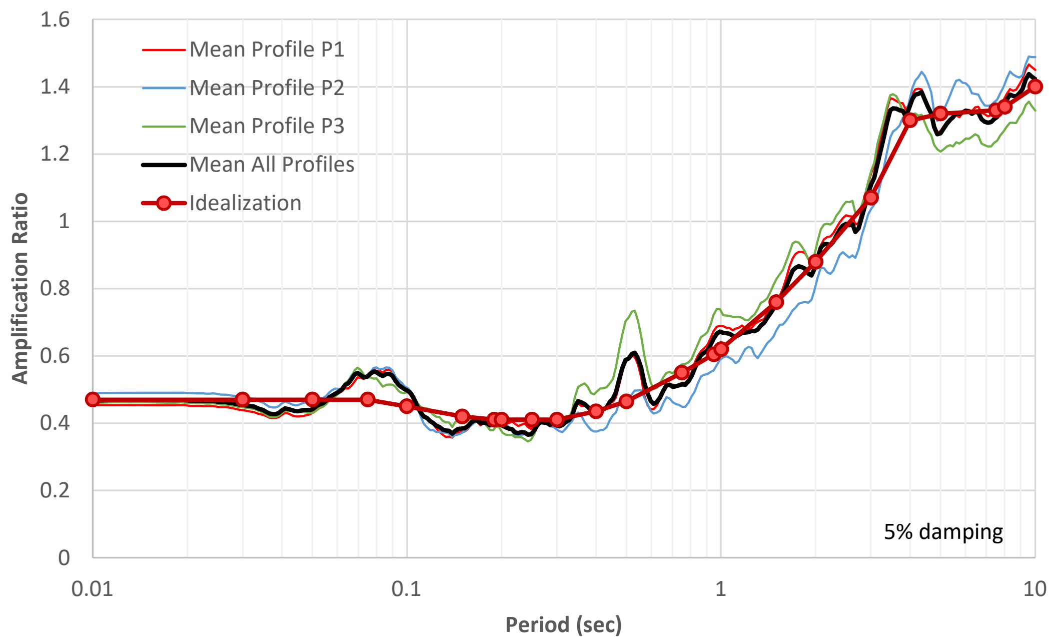
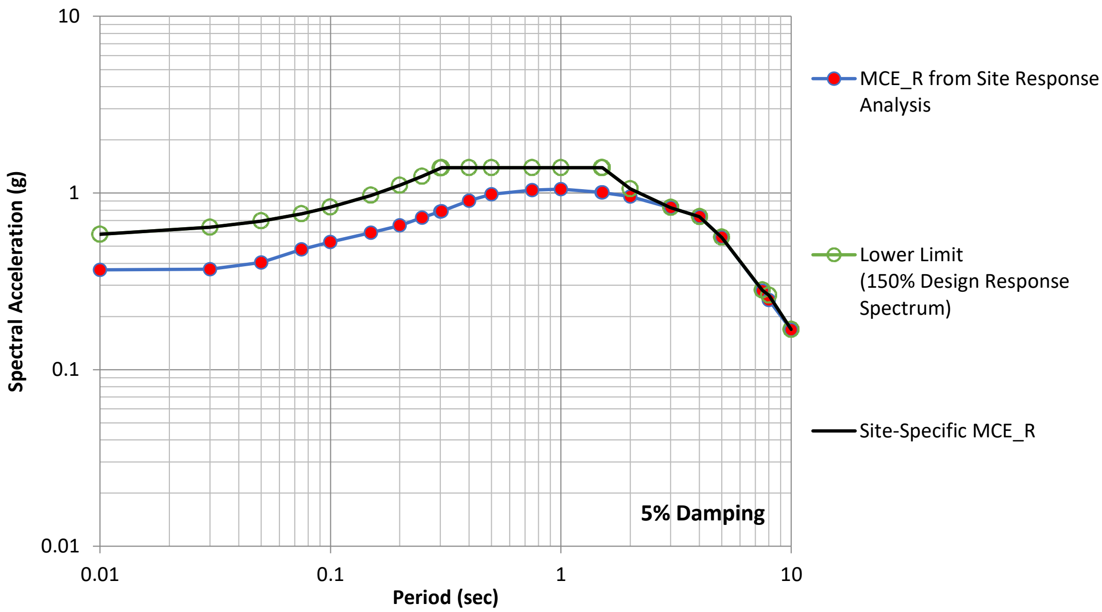


Figure G.6-3: Idealized Amplification Ratios



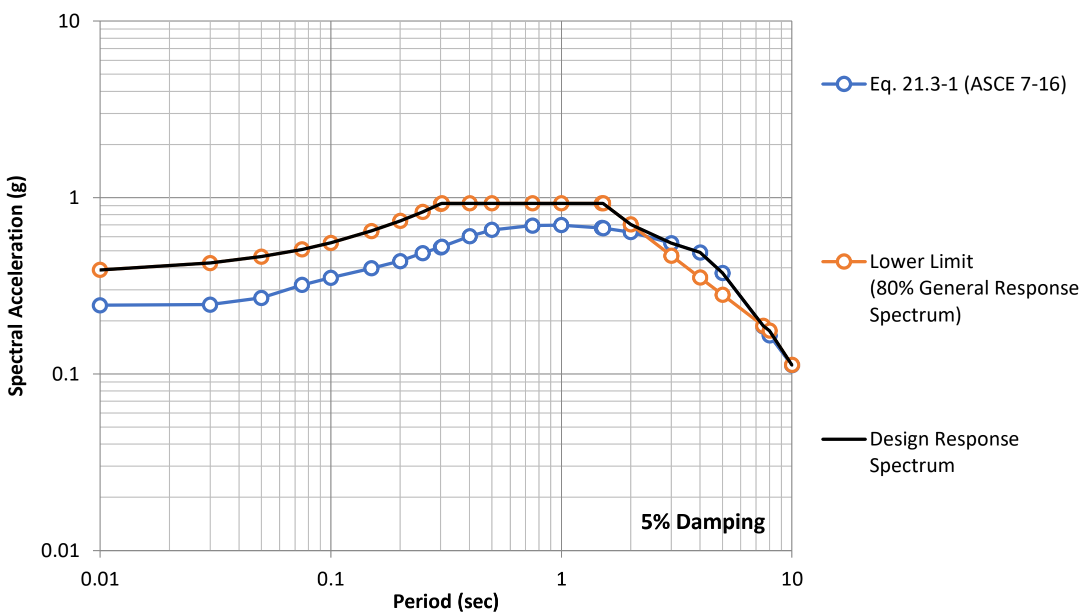


Figure G.7-1: Calculation of the Site-Specific Horizontal MCE_R and Design Response Spectra per ASCE 7-16 for the Ground Surface

Appendix H

LPILE Analyses

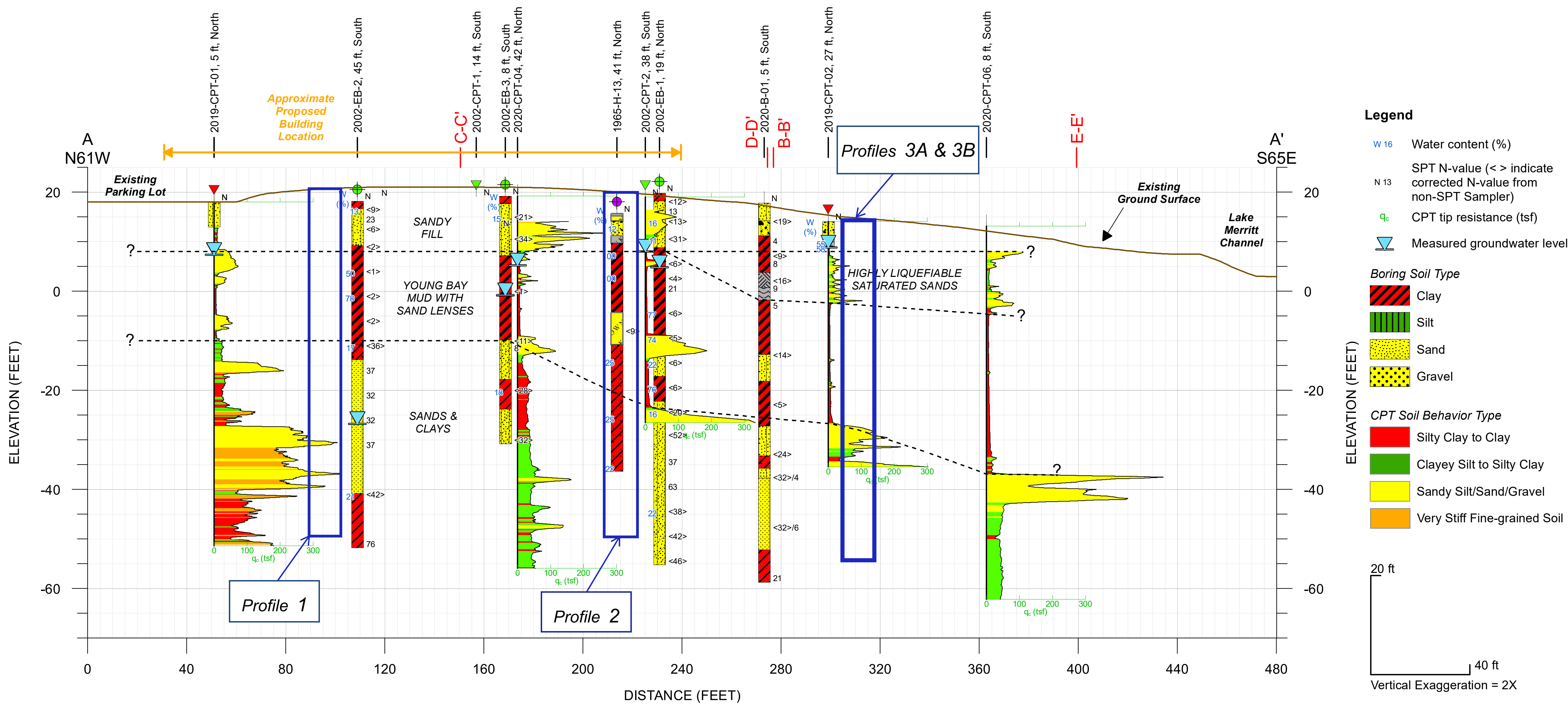


PLATE G-1: Profiles Along Cross Section A-A′

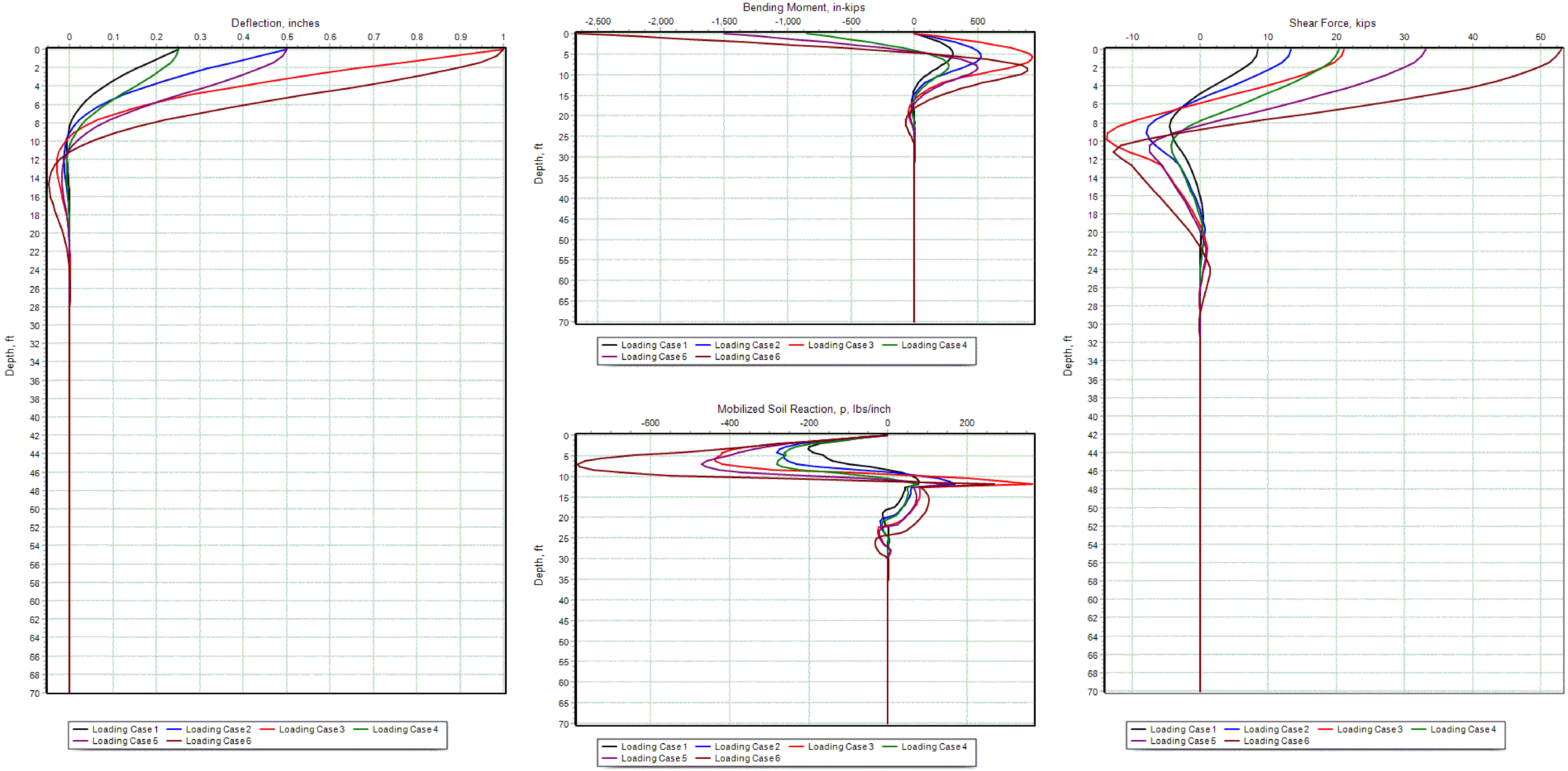


PLATE G-2: LPILE Results for Profile 1

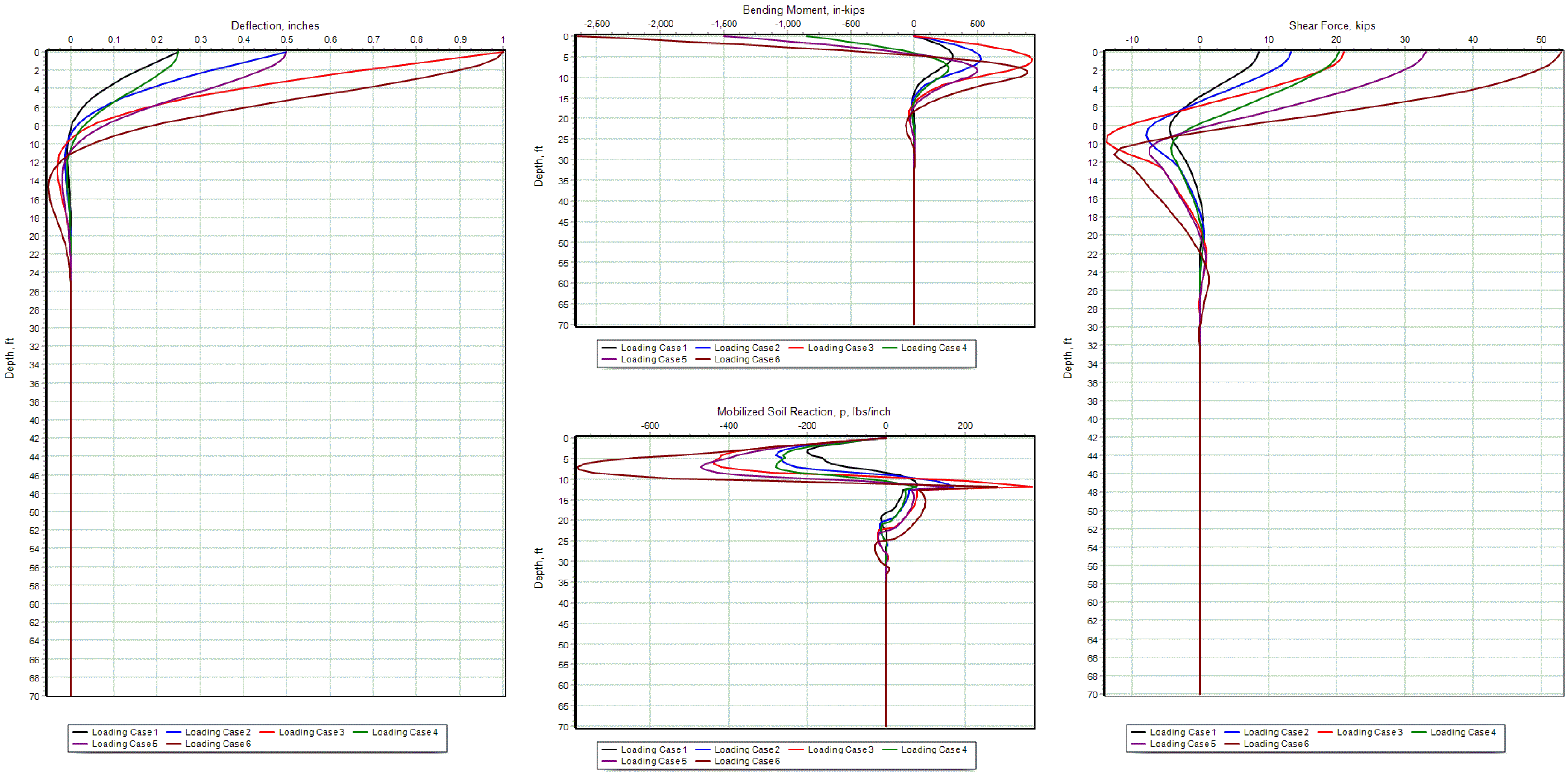


PLATE G-3: LPILE Results for Profile 2

FUGRO

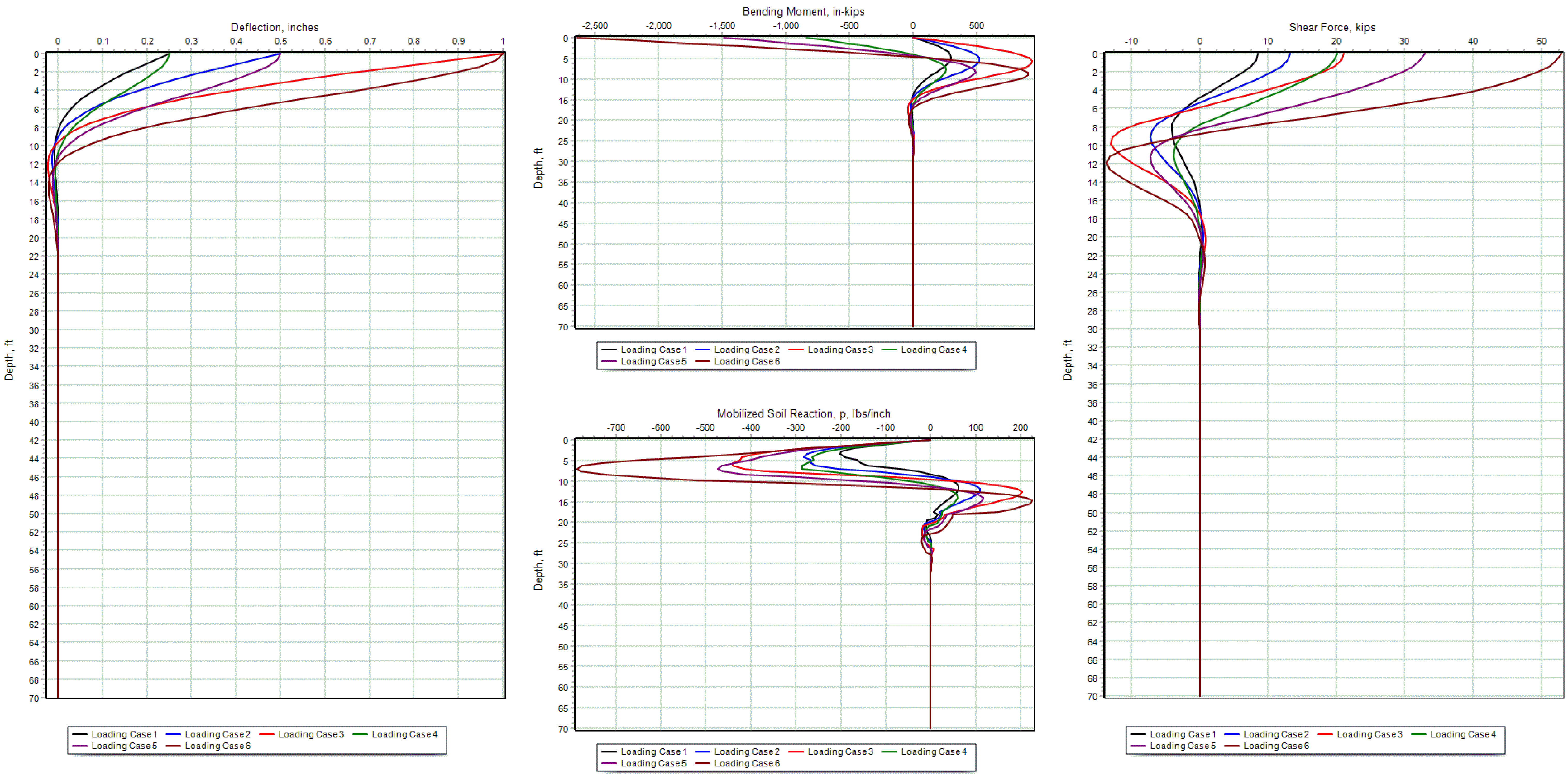


PLATE G-4: LPILE Results for Profile 3A

FUGRO

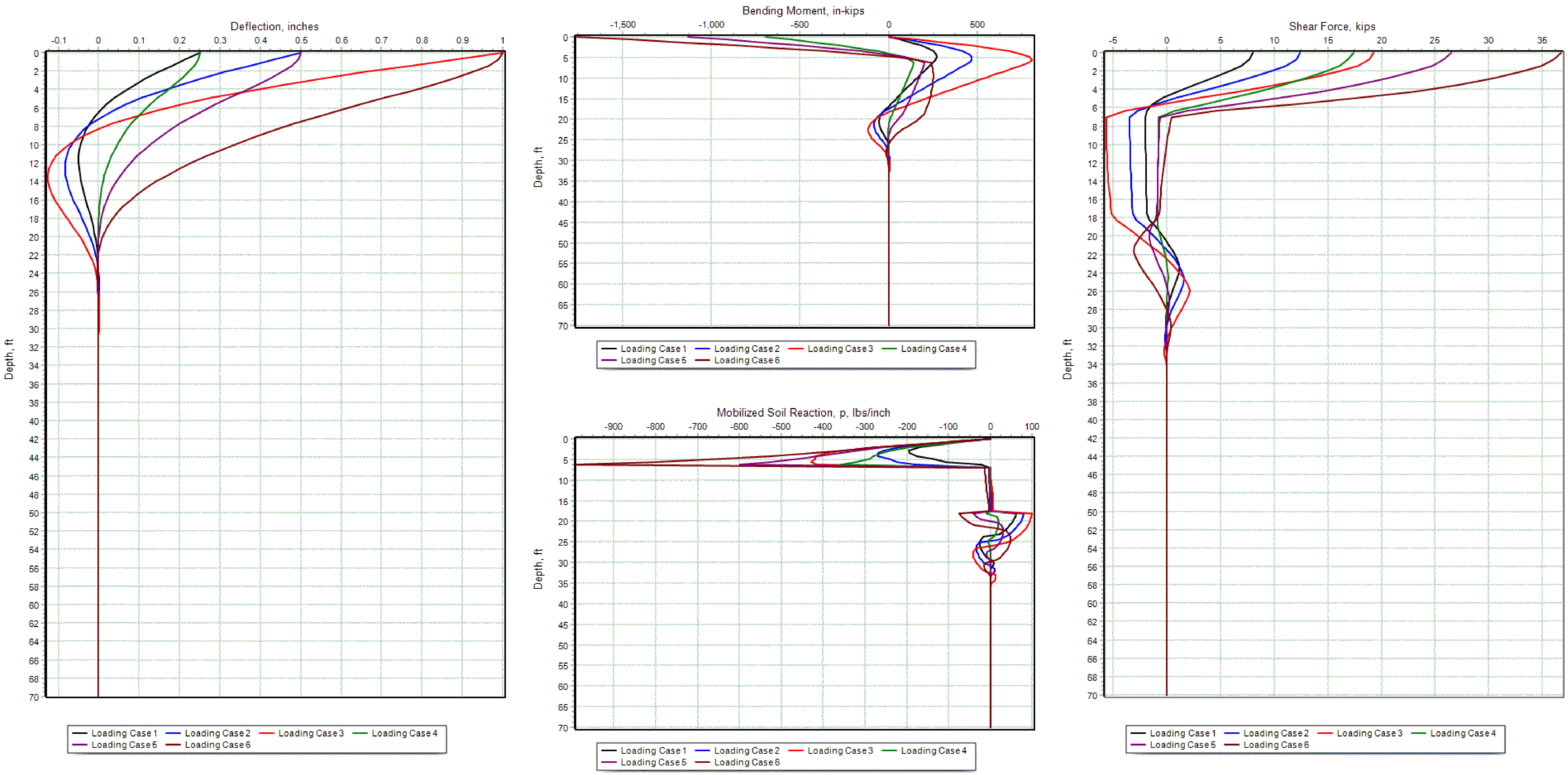


PLATE G-5: LPILE Results for Profile 3B

Appendix I

Response to CGS Comments

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December 22, 2021

Dear Ms. Smith,

Fugro is pleased to provide the following responses to the California Geological Survey (CGS) review comments on our Geotechnical Investigation and Geologic Hazard Evaluation report for the Laney College Library Learning Resource Center, Oakland, California, dated February 28, 2020.

The CGS review comments letter is titled "Engineering Geology and Seismology Review for Laney College & Learning Resource Center, 900 Fallon Street, Oakland, CA, CGS Application No. 01-CGS4416", dated December 1, 2020.

Based on CGS comments, reasonable geotechnical assessment of engineering geology and seismology issues for the project was provided by Fugro. However, CGS has requested more details and clarifications on a few aspects of the engineering geology and seismology issues at the site. Below, we have provided some clarifications on those comments of concern. A copy of CGS review letter is presented in Appendix A.

Response to CGS Review Comments

The review comments by CGS are indicated below in italics, which are followed by Fugro's responses and clarifications.

CGS Comment 14

> **CGS**: *General Procedure Ground Motion Analysis:* ***Additional information is requested.*** *The consultant's report the site is classified as Site Class F and site response analyses are required to calculate the design ground motions at the ground surface. However,* ***the consultants should report the mapped S_S and S_1 parameters.***
>
> **Fugro**: The mapped S_s and S_1 values are 1.74g and 0.66g, respectively. This will be added to the report.

CGS Comment 15

CGS: *Site-Specific Ground Motion Hazard Analysis:* ***Additional information is requested****. The consultants provide a site response analysis, as required for Site Class F.*

Development of Base Target Spectrum: The consultants provide a site-specific ground motion analysis to develop input ground motions for site response analyses. They use V_{s30} = 260 m/s to represent soil condition at the base of the site response model, which appears reasonable based on-site data presented.

The consultants' site-specific ground motion analyses follow most of the provisions of ASCE 7-16, and their probabilistic MCE spectrum appear reasonable based on comparison with results from the National Seismic Hazard Model (Petersen and others, 2014). However, they do not provide adequate documentation for CGS review. Also, there are minor inconsistencies with code procedure. Specially, we request the consultants address the following issues:

- *Provide pertinent parameters for all controlling faults in deterministic analysis, including fault name, characteristic magnitude, and site-to-fault distance. Present $Z_{1.0}$ and $Z_{2.5}$ values or report if GMM default values are used.*

  Fugro: Table 1 presents the characteristic magnitude and source-to-site distance of the seismic sources used to calculate the deterministic ground motions. This table will be added to the revised report.

  Table 1: Seismic Sources Used in the DSHA

Seismic Source	Characteristic Magnitude (Mw)	Source-to-Site Distance (km)
Hayward (No) 2011 CFM	6.9	5.6
Hayward (So) 2011 CFM	7.0	7.5
Hayward (No+So)	7.3	5.6
Hayward (No+So+SoExt)	7.4	5.6
San Andreas (Peninsula) 2011 CFM	7.4	24.5
San Andreas (Peninsula+North Coast)	7.7	24.5
San Andreas (Peninsula+North Coast+Santa Cruz)	7.9	24.5

  The Z1.0 (depth to Vs of 1 km/s) and Z2.5 (depth to Vs of 2.5 km/s) values used in the seismic hazard analyses correspond to the default values implemented the Ground Motion Models (GMM's). This text will be added to the revised report.

- *The discussion regarding design response spectrum for base of the YBM in Section G.4 and design response spectrum presented in Table G.4 and Figure G.4-2 are irrelevant and can lead to confusion.*

Only site-specific MCER spectrum at the base of YBM is needed for site response analysis. Site-specific design response spectrum at the surface is derived from site response analyses.

Fugro: The design response spectrum for the base of the YBM is required to calculate the site-specific MCER according to the requirements of ASCE 7-16 Supplement 1 (i.e., the site-specific MCER should not be less than 150 percent of the site-specific design response spectrum). For this project, the MCER is larger than the 150 percent of the design response spectrum; however, for completeness, we propose to leave the discussion of the development of the design response spectrum.

- *Add spectral-dependent maximum direction scaling factors and risk coefficients to Table G.4 to facility CGS review.*
- **Fugro**: Table 2 is a revised Table G.4 including the maximum direction scaling factors and risk coefficients. This table will be added to the revised report.

Table 2: MCER and Design Response Spectra per ASCE 7-16 for a Vs30 of 260 m/sec (base of YBM), 5% Damping

	Horizontal Spectral Acceleration (g)									
Period (sec)	UHRS for Return Period of 2,475 Years	Risk Coefficients	Max. Direction Scaling Factors	Probabilistic MCE_R	84th Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE_R	Site-Specific MCE_R	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.933	0.921	1.10	0.945	0.711	0.555	0.782	0.782	0.400	0.521
0.03	0.957	0.921	1.10	0.970	0.717	0.559	0.789	0.789	0.459	0.526
0.05	1.07	0.921	1.10	1.08	0.783	0.611	0.861	0.861	0.518	0.574
0.075	1.32	0.921	1.10	1.34	0.928	0.724	1.02	1.02	0.591	0.680
0.1	1.55	0.921	1.10	1.57	1.07	0.831	1.17	1.17	0.664	0.781
0.15	1.83	0.921	1.10	1.86	1.29	1.01	1.42	1.42	0.811	0.946
0.190	2.01	0.921	1.10	2.04	1.42	1.11	1.56	1.56	0.927	1.04
0.2	2.05	0.921	1.10	2.08	1.45	1.13	1.60	1.60	0.927	1.06
0.25	2.23	0.919	1.13	2.32	1.57	1.26	1.77	1.77	0.927	1.18
0.3	2.36	0.917	1.15	2.49	1.66	1.36	1.91	1.91	0.927	1.28
0.4	2.42	0.915	1.19	2.62	1.75	1.48	2.08	2.08	0.927	1.39
0.5	2.35	0.912	1.21	2.61	1.74	1.50	2.11	2.11	0.927	1.41
0.75	1.96	0.909	1.26	2.25	1.50	1.34	1.89	1.89	0.927	1.26
0.949	1.70	0.906	1.29	2.00	1.33	1.22	1.73	1.73	0.927	1.15
1	1.65	0.906	1.30	1.95	1.30	1.20	1.69	1.69	0.880	1.13
1.5	1.19	0.906	1.35	1.46	0.983	0.942	1.33	1.33	0.587	0.885
2	0.924	0.906	1.39	1.16	0.783	0.770	1.09	1.09	0.440	0.724
3	0.606	0.906	1.44	0.789	0.538	0.548	0.773	0.773	0.293	0.515
4	0.429	0.906	1.47	0.572	0.383	0.400	0.564	0.564	0.220	0.376
5	0.320	0.906	1.50	0.435	0.283	0.301	0.425	0.425	0.176	0.283
7.5	0.177	0.906	1.50	0.240	0.140	0.149	0.210	0.210	0.117	0.140
8	0.159	0.906	1.50	0.216	0.124	0.131	0.185	0.185	0.110	0.124
10	0.110	0.906	1.50	0.150	0.0801	0.0852	0.120	0.120	0.0704	0.0801

- *The "deterministic lower limit" appears to be outdated but does not affect the final spectra. The consultants are reminded that the "deterministic lower limit" has been superseded, see ASCE 7-16, Supplemental No.1, §21.2.2.*

  Fugro: The deterministic lower limit was calculated using ASCE 7-16 Supplement 1, as indicated in the last full paragraph of page 8 of Appendix G of the report, and on Figure G.4-1. We agree that ASCE 7-16 does not refer to this lower limit as "deterministic lower limit" anymore (as it was referred to in ASCE 7-10). However, the limit provided in ASCE 7-16 Supplement 1 is effectively a lower limit to the deterministic spectrum because the site-specific deterministic spectrum cannot be lower than this specified limit.

Time History Selection and Scaling: The consultants select five pairs of recorded acceleration time histories and scale them linearly to the approximate level of the site- specific MCER spectrum at base of YBM. The selection and scaling appear appropriate. However, potential near-source effects should be reflected in time history selection for a near-fault site (defined in Section 11.4.1, ASCE 7-16). The consultants are requested to modify their time history selection ***to include records with velocity pulses*** *or present pulse periods in Table G.5 if any of their ten (10) time histories contain velocity pulses. They also are requested to present scaled input time histories (including acceleration, velocity, and displacement) and, at a minimum, one set of example time histories at the ground surface.*

Fugro: Only ground motion RSN1176 include a velocity pulse with a period of 4.9 sec. Using a mean earthquake magnitude of 7.27 for spectral acceleration at 1 sec (see Table G.2) and the model proposed by Shahi and Baker (2014), the expected period of the velocity pulse at the site is 5.1 sec. Given the fundamental period of the soil column of approximately 0.4 sec, and the fundamental period of the structure of approximately 0.33 sec (Y-direction) and 0.45 sec (X-direction), ground motions with velocity pulses with a period of 5.1 sec are expected to have negligible effect on the analyses. The as-recorded seed input ground motion time histories are provided on **Plate 1**. Examples of acceleration and displacement time histories recorded at the ground surface from the site response analyses conducted for profile P1 are provided on **Plate 2**.

Site Condition Modelling and Site Response Analyses: The consultants' site response analyses, for the most part, follow the procedure in Section 21.1, ASCE 7-16. However, their documentation is insufficient for CGS review. In addition, some modelling aspects can be improved. The consultants are requested to address the following issues:

- *The idealized shear wave velocity profile (Figure G.2-3) appears appropriate for YBM and the underlying sand and clay layers. However, for fill layers above the YBM, it does not reflect either the in-situ Vs measurements from CPT-07 or the correlation results from Andrus et al. (2007) and is much lower than both these datasets. The consultants are requested to justify their selection or modify Vs to be consistent with site data.*

Fugro: The dashed line in Figure G.2-3 of the geotechnical report is the depth-dependent shear wave velocity idealization used for YBM only. The depth and thickness of YBM varies at the site. Highly Liquefiable Saturated Sands (HLSS) were encountered east of the proposed building location and at the location of the in-situ seismic measurements (CPT-07, Plate 3 in the main text). Idealized profiles for site response analyses (Figure G.2-6) were selected to reasonably represent conditions within the proposed building footprint, and hence the top of YBM was idealized at 10 feet below the ground surface. Note that Figure G.2-3 does not show idealized shear wave velocities for the sandy fill above the YBM (because there were no seismic measurements in fill at this location east of the proposed building). Idealized shear wave velocities for the sandy fill and the liquefiable YBM Sand (idealized stratigraphy is shown on Figure G.2-6) were based on correlation with N160 (as described in section G.6.1.1 and Table G.6). Per section G.6.1.2, representative clean sand N1,60 values of 17 and 12 were used to model the fill and YBM Sand, respectively. These N1,60 values correspond to Vs1 = 586 ft/s in the fill (i.e., Vs ranges from about 300 to 500 ft/s in the fill) and Vs1 = 544 ft/s in the YBM Sand (i.e., Vs of about 550 ft/s in the YBM Sand). These shear wave velocities are reasonably consistent with the available range of values from CPT-based correlations.

- *Indicate detailed layering used in FLAC modeling for site responses for each of the three idealized stratigraphy shown in Figure G.2-6. Present and justify Vs and unit weight for each layer. The consultants are further requested to: i) justify why it is sufficient to include one single liquefiable layer in only one of the three profiles modeled; ii) clarify whether FLAC analyses is total stress or effective stress analyses, and iii) discuss the effect of liquefaction process in the liquefiable layer on surface ground motion.*

Fugro: Idealized stratigraphy for site response analyses are shown on Figure G.2-6 and model discretization is described in section G.6.1.1 (i.e., 2.5-foot square zones were used in the FLAC models).

Justification for the idealized shear wave velocity in the YBM is shown on Figures G.2-2 and G.2-3 and described in section G.2.1. Basis for the idealized N160 values for the fill and YBM Sand is shown on Figure G.2-5 (CPT and SPT data). Idealized shear wave velocities for these sandy soils were correlated with idealized N160 values (section G.6.1.2 and Table G.6, and as described in the response to the previous comment) and the resulting range of Vs values is in reasonable agreement with the range of corresponding CPT data. Idealized unit weights where 120, 105, and 115 pcf were used for the fill, YBM, and YBM-Sand, respectively; these representative unit weights were selected based on review of available laboratory measurements.

Three idealized profiles for site response analyses were selected to reasonably capture the range of conditions at the proposed building location. A relatively continuous and approximately 5-foot-thick liquefiable YBM Sand layer was encountered near the southeast side of the proposed building location. This layer was modeled in P2 (figure G.2-6). Additional sand lenses exist within the YBM but were not included in the 1D analyses because they are generally thin and discontinuous. In any case, for the MCE_R level of shaking, the modeled liquefiable sand within the YBM had a negligible effect on the surface spectra as described in section G.6.2.1.

The FLAC analyses were effective stress analyses.

Results from the baseline analyses are described in section G.6.2.1. Yielding in the YBM due to the MCE_R ground motions deamplifies higher frequencies and effectively base isolates the soil columns, hence there is little difference in the surface response spectra for the three idealized profiles (one of which includes liquefiable YBM Sand).

- *It is important for CGS to evaluate whether uncertainty in shear wave velocity is adequately incorporated in site response analyses. The consultants are requested to clearly present their upper bound and lower bound Vs profiles for each of the three idealized stratigraphic models and demonstrate that these profiles cover the uncertainty range adequately. They are further requested to present the surface spectra from all site response sensitivity cases (in terms of stratigraphy and shear wave velocity) and to demonstrate that the final recommended surface MCER spectrum sufficiently envelops surface spectra from all analysis cases. We note it is appropriate to use mean surface-to-base spectral ratios from all time histories as transfer function for each soil profile case. However, it is inadequate to take the mean spectrum from all soil profile cases as the recommended surface spectrum. Instead, the recommended surface MCER spectrum should sufficiently envelop surface spectra from all sensitivity profile cases.*

  CGS notes it appears the consultants use modified Fa (1.0) and Fv (4.0) values associated with Site Class E in their input site-specific ground motion analysis for the site response analysis, which appears to be in accordance with ASCE 7, §21.3.

  Fugro: Parametric analyses were performed for YBM stiffness and strength as described in section G.6.2.2. Lower bound (0.87 x baseline idealization) and upper bound (1.2 x baseline idealization) YBM Vs profiles were developed to capture the range of YBM shear wave velocities estimated from the Mayne and Rix (1995) CPT correlation. For the MCE_R level of shaking the YBM yields and develops large shear strains. This behavior was considered in the selection of meaningful parametric analyses (i.e., for lower bound Vs and upper bound Su). Analyses with the upper bound YBM undrained shear strength Su had the greatest effect on surface spectra. For smaller levels of shaking greater sensitivity of surface spectra to reasonable YBM Vs variations is expected. These sensitivity analyses are summarized in the figures below.

  As discussed in our response above, yielding in the YBM due to the MCER ground motions deamplifies higher frequencies and effectively base isolates the soil columns, hence there is little difference in the surface response spectra for the three idealized profiles. Additionally, Figure G.7-1 indicates that the surface MCER and design response spectra are controlled by the lower limit specified by ASCE 7-16 up to periods of about 2 seconds. Therefore, given the fundamental period of the structure of approximately 0.33 sec (Y-direction) and 0.45 sec (X-direction), averaging or enveloping the results from the three idealized profiles has a negligible effect.

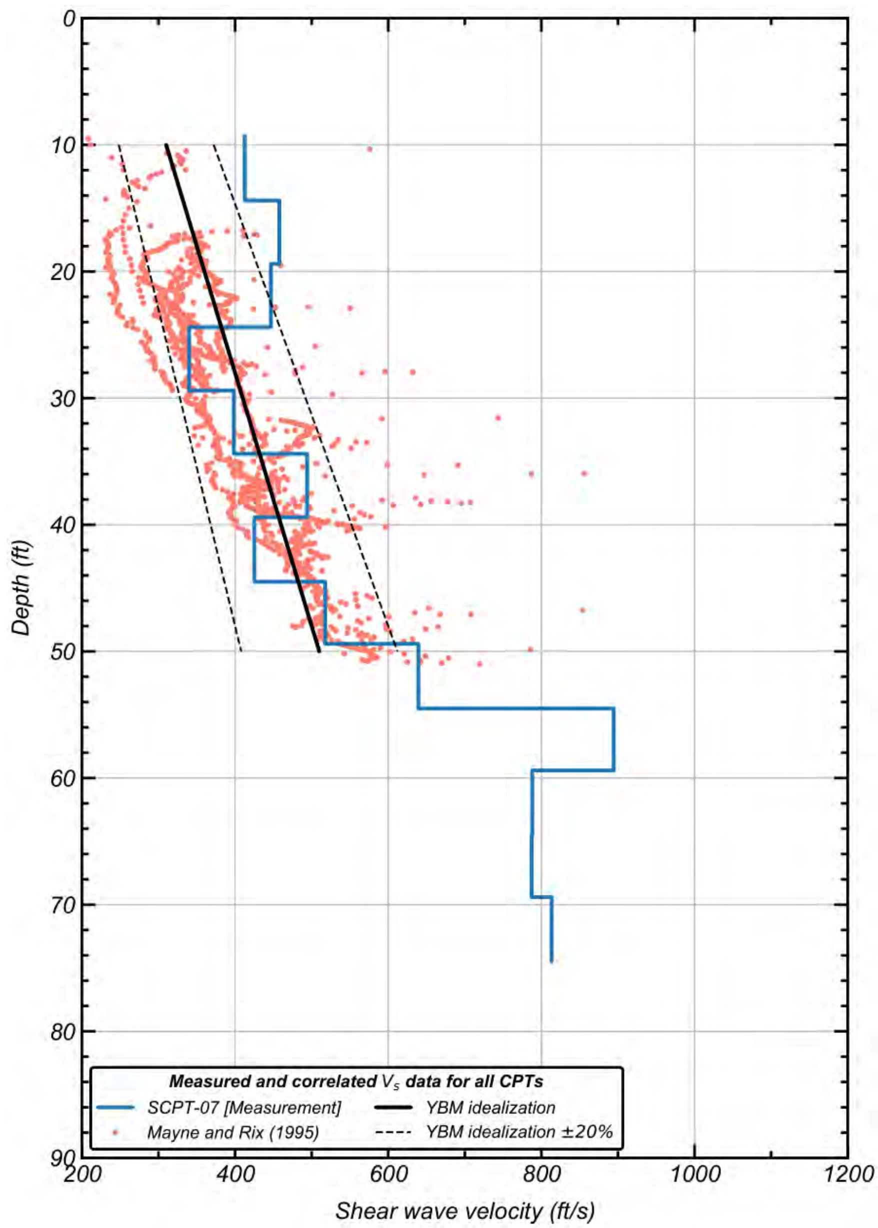

Figure 1: Measured and Correlated Vs Data and Idealized YBM Vs Profiles

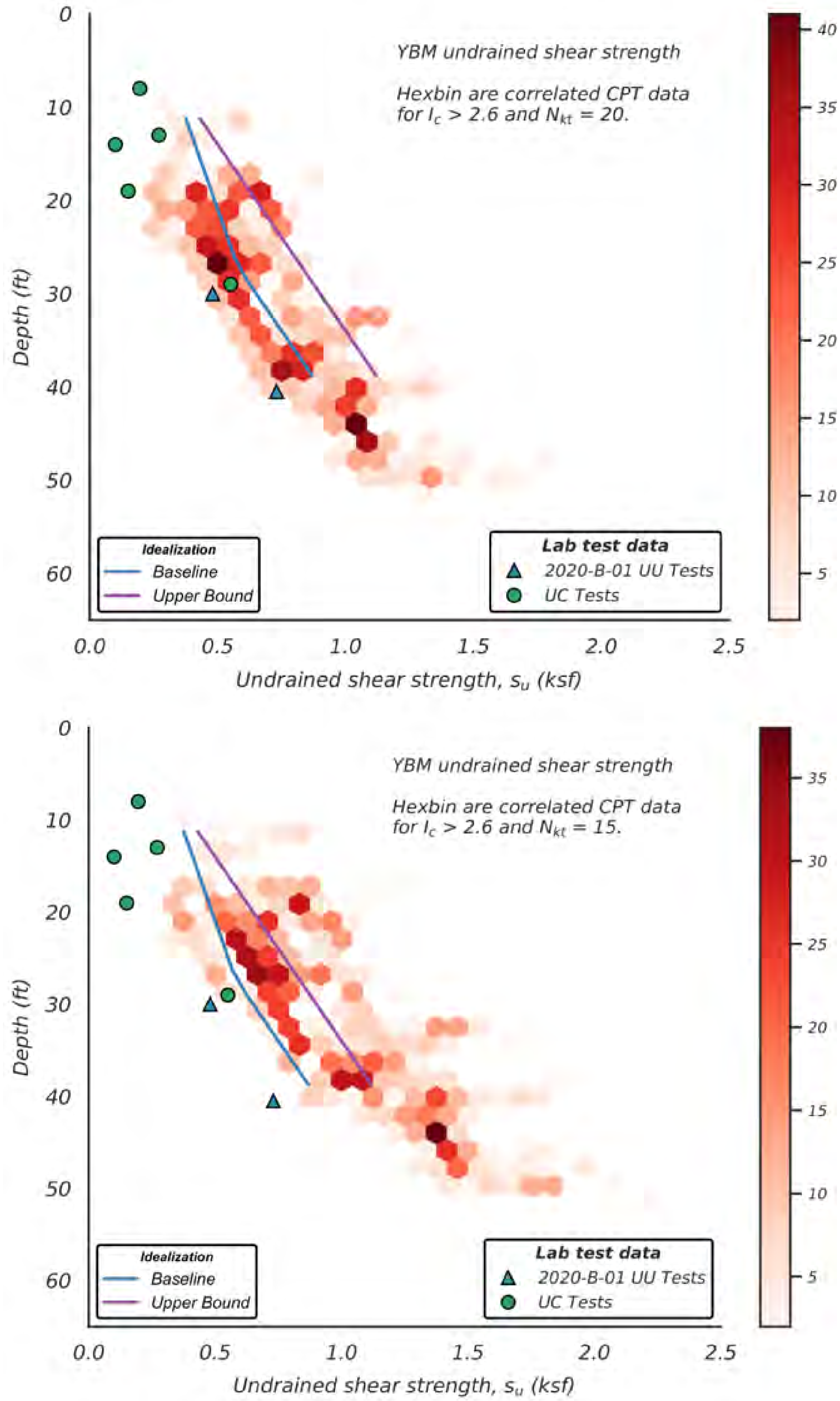

Figure 2: YBM Undrained Shear Strength: (top) Nkt = 20; (bottom) Nkt = 15

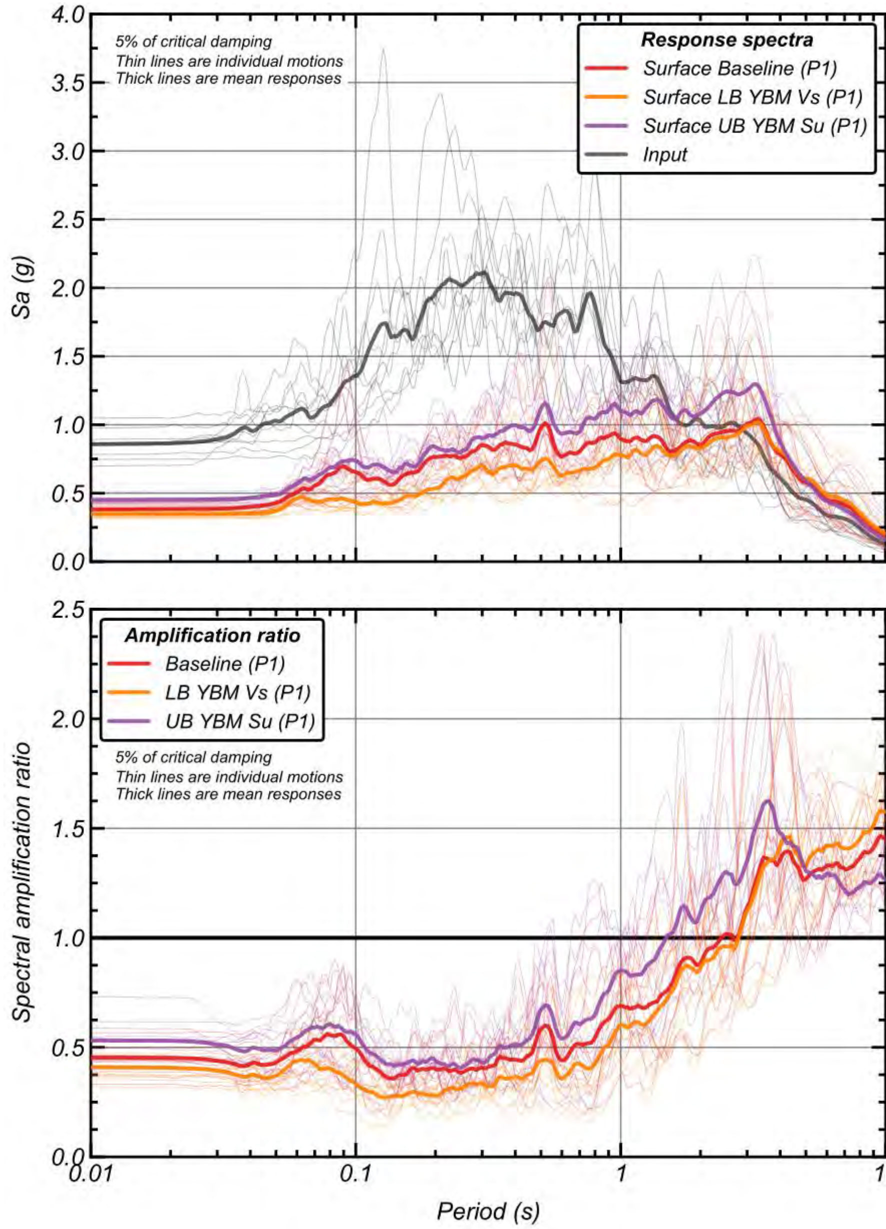

Figure 3.

Figure 3: Surface Response Spectra and Amplification Ratios from 1D Site Response Analyses for Profile P1 Sensitivity Analyses

CGS Comment 17

CGS: *Time Histories of Earthquake Ground Motion: See item 15.*

Fugro: Explained in item 15 and **Plate 2**.

CGS Comment 20

CGS: *Seismic Settlement Calculations:* ***Additional information is requested****. The consultant's report they used a PGA_M of 0.810g and set groundwater at an elevation of +8 feet in their liquefaction analysis of the site, which appear to be reasonable. However, the consultants should utilize the* ***maximum considered earthquake magnitude associated with the Hayward Fault*** *in accordance with 2019 CBC §1803A.5.12, Subsection 2 in their analysis of liquefaction and seismic settlement for the proposed improvements.*

Fugro: We performed a sensitivity analysis with changing earthquake Magnitude to 7.6. Sensitivity analyses show the estimated settlements are not sensitive to earthquake Magnitude as the volumetric strains models saturate at the already low Factors of Safety estimated in this study.

CGS Comment 21

CGS: *Other Liquefaction Effects:* ***Additional information is requested****. The consultant's report the loose to medium dense, near-surface sand layers encountered by CPTs and borings in the area adjacent to the Lake Merritt Channel are liquefiable and have a* ***high potential to undergo lateral spreading*** *during soil liquefaction resulting from an MCE event. This general conclusion appears reasonable based on the information provided. The consultants have estimated the areal extent and potential horizontal displacements from lateral spreading using the same input parameters of earthquake magnitude and peak ground acceleration as for their analysis of liquefaction triggering. However, as noted above in Item 20, the consultants* ***should utilize the maximum considered earthquake magnitude associated with the Hayward Fault in their analysis of liquefaction and associated effects, including lateral spreading.*** *The consultants are therefore requested to update their analysis of potential lateral spreading and to review and revise their conclusions regarding the likely areal extent and estimated horizontal displacements of lateral spreading at the site, if warranted. The consultants are further requested to review their recommendations for mitigation of lateral spreading and associated instability of the gently sloping ground adjacent to the Lake Merritt Channel, if warranted, based on the results of their updated analysis.*

Fugro: We used empirical correlations developed by Youd et al. (2002) to estimate the possible ground displacement at the location of each CPT and boring for the earthquake scenario with Magnitude of 7.6 and distance to fault of 6.8 km. Estimated lateral displacements are shown in table below. Lateral displacement is sensitive to the earthquake magnitude. Increase in magnitude by 0.6 will result in a maximum lateral displacement of about 5 feet.

Table 3: CPT- and SPT-Based Lateral Displacement (feet)

Location	Magnitude=7.6; Distance=6.8 km
2019-CPT-01	3.0
2019-CPT-02	4.2
2019-CPT-03	3.1
2020-CPT-04	2.8
2020-CPT-05	4.1
2020-CPT-06	4.5
2020-CPT-07	2.9
2020-CPT-08	4.2
2002-CPT-2	4.9
2020-B-01	4.5
2002-EB-1	5.2
2002-EB-2	4.6
2002-EB-3	3.5

To protect the building against lateral spreading impact, we recommended permanent shoring to be installed at the east side of the building, as shown on **Plate 3**. It is not planned to improve the sloping ground beyond the shoring wall, and it is not part of the scope of our geotechnical investigation. However, considerations need to be given to the 36-in diameter pipeline that is planned to be placed in between the LRC building and the Lake Merritt Channel.

Due to the conflict between the LRC building footprint and the pipeline, East Bay Municipal Utility District (EBMUD) is planning to relocate the pipeline further east towards the Lake Merritt Channel and replace it with flexible earthquake resistance iron pipe that is capable of resisting large lateral displacements during earthquake. Location of the relocated pipeline is shown on **Plate 3**.

CGS Comment 22

CGS: *Mitigation Options for Liquefaction/Seismic Settlement:* ***Additional information is requested.*** *The consultants reasonably recommend the building be supported by pile foundations bearing in competent soils at depths well below liquefiable soils and designed to resist downdrag due to liquefaction-induced settlement. They also recommend using flexible connections for pipes/utilities to mitigate potential damage resulting from liquefaction-induced settlement of soils outside the building relative to the pile-supported structure.*

Based on their assessment of the limits and estimated horizontal displacements due to lateral spreading, the consultants recommend the southeast side of the new building foundation should include a "permanent shoring system" to mitigate the detrimental effects from the potential lateral spreading and *slope instability from the areas immediately adjacent to the Lake Merritt Channel. While this general recommendation appears, reasonable,* ***the consultants are requested to review the results of updated lateral spreading analysis requested in Item 21 and to update their mitigation recommendations if warranted.*** *Further discussion and request for additional information regarding the recommended "permanent shoring system" is provided in Item 28 of this checklist.*

Fugro: Estimated lateral displacements based on those borings and CPTs located on the slope show up to about 7.5 feet of displacement. The new pipeline should be designed to tolerate up to 7.5 feet of displacement. Refer to the response to **Comment No. 26** for further discussion on analysis for lateral spreading.

CGS Comment 24

CGS*: Determination of Static and Dynamic Strength Parameters:* ***Additional information is requested****. The consultant's report soil engineering properties were developed based on the field exploration and laboratory testing results by Fugro and others, and typical engineering correlations. They provide unit weights and static and seismic shear strength parameters for the sandy fill, highly liquefiable sand, and sand and clay soil layers which appear to be reasonable based on the data provided. The consultants also provide soil properties for YBM, which include a total unit weight, a minimum undrained shear strength, and an undrained shear strength ratio used in both static and seismic analyses. However,* ***the undrained shear strength ratio appears to be high*** *based on review of the reported laboratory test data (TXUU) and does not appear to consider the potential for cyclic softening and loss of strength during seismic loading (refer also to Item 31I). The consultants should provide further discussion and justification for the higher than anticipated undrained shear strength ratio for the YBM and consider* ***potential for reduced strength in YBM due to cyclic softening****.*

Fugro: Figure 2 (refer to response to **Comment No. 15**) shows the Undrained Shear Strength Ratio interpreted from CPT data. An average Strength Ratio of 0.28 is selected for the updated stability analysis. Strength measurements from UU tests fall on the lower end of the range of the CPT data (when interpreted using a likely conservative Nkt value of 20) as shown in **Figure 2** in the response to Comment No.15. This is likely due to sample disturbance (e.g., Ladd & DeGroot, 2003).

An undrained shear strength ratio of 0.22 (i.e., 0.8 x 0.28) is used in our updated pseudo-static slope stability analysis. An undrained shear strength reduction factor of 0.8 was selected to approximate YBM cyclic softening behaviours for pseudo-static analyses representing dynamic loading conditions based on the cyclic strength (i.e., cyclic resistance ratio, CRR) for M=7.5 suggested by Idriss and Boulanger (2008) for plastic silts and clays, and Fugro's past experience characterizing and modelling YBM.

For post-earthquake stability analysis for static loading, a multiplier of 0.6 was used to account for strength degradation from cyclic softening (i.e., undrained shear strength ratio= 0.6 x 0.28 = 0.17). For post-earthquake analyses a strength reduction factor of 0.6 was used based on post cyclic strength data in the literature (e.g., Thiers & Seed, 1968) and Fugro's past experience characterizing and modelling YBM (e.g., Price et al., 2021). The 0.6 factor is for static loading rates.

CGS Comment 25

CGS: *Determination of Pseudo-Static Coefficient (Keq):* ***Additional information is requested****. The consultant's report a pseudo-static coefficient (k) of 0.15g. They report calculating slope displacement based on maximum horizontal acceleration of 0.81g from a modal magnitude 7.5 causative earthquake located 6.8 km from the site, which appears to be reasonable. However, they should* ***provide supporting calculations utilizing these input values for our review****.*

Fugro: Seismic coefficient was updated based on the recommended procedure by DMG Special Publication 117, Guidelines for Analysing and Mitigating Landslide Hazards in California. We assumed a magnitude 7.6 earthquake located 6.8 km from the site. The seismic coefficient keq, is obtained by:

$$K_{eq} = f_{eq} \, x \, MHA_r$$

Where MHA_r is the maximum horizontal acceleration at the site for soft rock condition and f_{eq} is a factor that accounts for the effects of site seismicity. f_{eq} can be obtained by:

$$f_{eq} = \frac{NRF}{3.477}\left[1.87 - \ log_{10}\left(\frac{u}{(MHA_r \, g \ \times NRF \ \times \ D_{5-95}}\right)\right]$$

Where NRF is a factor that accounts for the nonlinear response of the material above the slide plane, u is displacement; and $D_{5\text{-}95}$ is the duration of strong shaking that is a function of earthquake magnitude and distance[1].

California Geological Survey (CGS) – Note 48 checklist recommends "using design-level ground motion based on geometric mean and without risk coefficient (i.e., PGAM/1.5)" to be used for the determination of pseudo-static coefficient. Therefore, following input values was used in our calculations:

MHA_r = 0.81g/1.5 = 0.54g; Earthquake Magnitude= 7.6, Distance (u) = 6.8 km; and Slope displacement (u)= 76 cm (2.5 feet). Calculations are presented in **Plate 4**.

CGS Comment 26

CGS: *Identify Critical Slip Surfaces for Static and Dynamic Analyses:* ***Additional information is requested****. The consultants present analyses of slope failure for three cross sections (A-A', D-D', and E-E') extending from/through the proposed library building to the adjacent Lake Merritt Channel. They analyze four cases for each section: long-term (static), pseudo-static (yield acceleration), pseudo-static (fixed surface and pseudo-static coefficient), and post liquefaction (static) scenarios. The consultant's report by fixing the slip surface daylight location at the edge of the proposed building location, factors of*

[1] Stewart, J. P et. al (2003). A screen Analysis Procedure for Seismic Slope Stability. Earthquake Spectra, 19 (3).

safety against slope failures for the pseudo-static analyses are below 1 for each of the three sections. They conclude the analyses fail to meet the commonly accepted minimum factor of safety value of 1.15 for seismic performance and that seismic slope stability of the site is most likely governed by the extent of possible lateral spreading. While CGS considers these general conclusions reasonable, we ***request the consultants discuss and consider if non-circular failure surfaces would better represent potential lateral spreading masses and may result in lower factors-of-safety and/or greater potential hazard to the proposed building than the analyzed circular surfaces.*** *CGS also requests the consultants consider revised strength parameters for the YBM soils in their additional analyses of circular and tabular or wedge failure surfaces that may better reflect likely deformation mechanisms.*

Fugro: The table below summarizes the updated soil properties used in our analyses:

Table 4: Soil Engineering Properties Used in Site Slope Stability Analyses

Material	Unit Weight (pcf)	Material Shear Strength	
		Cohesion c′ (psf)	Friction Angle Φ′ (degree)
Sandy Fill	120	0	35
Young Bay Mud with Sand Lenses	90	(a) x Effective Overburden Stress (psf)	0
Interbedded Clays and Sands	130	0	40
Highly Liquefiable Sands	110	0	33
Post-Liquefaction Sands (Residual Strength)	110	100 + 20 x Depth (ft)	-
* a=0.28 for static, a=0.22 for pseudo-static condition, and a=0.17 for post-liquefaction condition			

The results of our slope stability analyses are presented in the table below. The factors of safety for circular slip surface are lower than wedge failure case (values in parenthesis). Our interpreted cross-section stratigraphic profiles, soil engineering properties used in the analyses, and the detailed results of the analyses are presented on the computer program printouts in the attached **Plates 5-1** through **5-24.**

Table 5: Slope Stability Analysis Results for Circular Slip Surface and (Wedge Failure)

Cross-Section	Case 1 Long Term	Case 2 Seismic Event Yield Acceleration	Case 3 Seismic Event k = 0.15g; Fixed Slip Surface at Edge of Building	Case 4 Post-Liquefaction
	Factor of Safety	Ky	Factor of Safety	Factor of Safety
A-A′	2.4 (2.5)	0.1 (0.1)	0.6 (0.6)	1.5 (1.6)
D-D′	1.9 (2.0)	0.09 (0.1)	0.6 (0.7)	1.3 (1.4)
E-E′	1.6 (1.7)	0.08 (0.09)	0.6 (0.6)	0.9 (1.0)

The results of our slope stability analyses generally indicate that the factors of safety against slope failures for the Case 1 (Long Term, Static) are 2.4, 1.9, and 1.6, respectively, for Sections A-A′, D-D′ and E-E′, which exceed the generally accepted value of 1.5 for long term conditions.

For the Case 2 (Seismic Event Yield Acceleration, Pseudo-static), the yield accelerations (ky) are determined to be 0.1g, 0.09g, and 0.08g for Sections A-A′, D-D′, and E-E′, respectively. Using the Bray (1998) procedure as recommended by SCEC publication (2002), we calculated slope displacements on the order of about 19 to 34 inches (48 to 86 centimetres) may occur during an MCE event (with a maximum horizontal acceleration of 0.81g from a mode magnitude 7.6 causative earthquake located at 6.8 kilometres from the site). These calculated displacements exceed the threshold of 6 inches (15 cm) defined by the SCEC publication (2002), which likely distinguishes conditions in which small to moderate displacements are likely from conditions in which large displacements are likely. As indicated on the result printouts, the most critical slip surfaces along these cross-sections daylight about 25 feet behind the edge of the proposed building.

In addition, factors of safety against slope failures for the Case 3 (Seismic Event k = 0.15g, Pseudo-Static) are all 0.6 for Sections A-A′, D-D′ and E-E′, which also fail to meet the commonly accepted minimum value of 1.15 for seismic performance (Seed, 1979)[2].

In Case 4 (Post-Liquefaction, Static), post-liquefaction residual shear strength was used for the highly liquefiable sands. The factors of safety against slope failures are 1.5, 1.3, and 0.9 for Sections A-A′, D-D′ and E-E′, respectively. The factor of safety for Section E-E′ is lower than generally accepted minimum value of 1.3 for short term conditions after major liquefaction events.

Seismic slope stability analysis suggests that for Sections A-A′ and D-D′, the slip surface starts at the edge of the building. If we were to let the slip surface to go beyond the building edge the slip circle will start from about 25 feet behind the edge of the building (see **Plates 6-1**, and **6-2**). It should be noted that the project proposed building location is located about 130 to 160 feet away from the

[2] Seed, 1979. *Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams*, Geotechnique, V. 29 (3), p. 215-263.

edge of the west bank. Our evaluations are only meant to assess the global stability of the proposed development and the potential lateral extents of ground failures caused by the possible lateral spreading of the channel bank during an MCE event (if it ever occurs). Detailed stability evaluation of the existing channel west bank is beyond our scope of work.

Due to the high degree of uncertainties on site subsurface conditions, seismic characteristics of the triggering earthquake, and analysis methodology, the results of our seismic slope stability and lateral spreading analyses should be considered as an index of site performance during major earthquake events. It is our opinion that the potential for slope instability during an MCE event and/or after major liquefaction event to impact the proposed building location is low to moderate. However, extensive slope failures may occur for the areas immediately adjacent to the Lake Merritt Channel if soil liquefaction and ground lateral spreading do occur at the site region during major earthquake events. Our estimated updated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3**.

CGS Comment 27

CGS: *Dynamic Site Conditions:* ***Additional information is requested****. CGS notes that seismic slope stability should consider site response analysis in determining the psuedo-static coefficient (see Items 15 and 25).*

Fugro: Conservatively, we used PGA=0.81 in our calculations based on ASCE 7-16 for site class E. Note that based on the site-specific analysis per ASCE-7-16 section 21.5, site specific PGA_M is 0.59. Also, per California Geological Survey (CGS) – Note 48 checklist recommends "using design-level ground motion based on geometric mean and without risk coefficient (i.e., PGAM/1.5)" to be used for the determination of pseudo-static coefficient (=0.39g).

CGS Comment 28

CGS: *Mitigation Options/Other Slope Failure:* ***Additional information is requested****. As noted previously (refer to Items 10, 11B, and 22), the consultants recommend the southeast side of the building foundation should include a "permanent shoring system" to mitigate the detrimental effects from the potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. They further recommend that the shoring system be designed to retain from 12 to 18 feet of soils, and they provide recommended lateral pressures for the shoring system design on a pair of Plates that indicate the system is "to be designed by others." Given our requests for the consultants to provide additional information and updated analyses of lateral spreading and slope stability that inform the consultants' recommendations for the shoring system, CGS requests the consultants* ***address the potential for greater areal extent/depth of lateral spreading/slope instability to be mitigated by the shoring system****. CGS further requests the consultants* ***provide discussion and justification for the lateral pressures recommended for design and address the potential for load transfer/surcharge from the building grade beams and/or pile foundations to the shoring system that should be incorporated in the design of the system****. When available, the design and plans for the shoring system should also be provided to CGS for review.*

Fugro: As Explained in the response to **Comment No. 26**, the areal extent of lateral spreading shifts about 25 feet behind the edge of the building. Our results generally indicate the loose to medium dense sand layers encountered by CPTs and borings in the area adjacent to the Lake Merritt Channel at Elevations between +8 and -5 feet have a high potential to trigger ground surface lateral spreading during soil liquefaction from an MCE event. The other onsite liquefiable sand layers are considered as having low potential to trigger ground lateral spreading due to their presence in isolated thin pockets and/or being located in deeper depths in relation to the bottom of the Lake Merritt Channel. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3**.

In **Section 7.4.1** of the Geotechnical report, we recommended any undrained unrestrained walls, which are free to deflect or rotate, be designed to resist an equivalent fluid pressure of 85 pounds per cubic foot (pcf). Undrained restrained walls should be designed to resist an equivalent fluid pressure of 100 pcf. This assumes walls with level backfills. Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 1/3 the anticipated surcharge load for unrestrained walls, and 1/2 the anticipated surcharge load for restrained walls.

If back-drainage is provided behind the walls, we recommend that drained unrestrained walls be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot (pcf). Drained restrained walls should be designed to resist an equivalent fluid pressure of 75 pcf. These recommended drained lateral pressures assume walls are fully-back drained to prevent the build-up of hydrostatic pressures.

The coefficient of active (unrestrained condition) and at-rest (restrained condition) earth pressure of 0.4 and 0.6, respectively were used in the static lateral earth pressure calculations. For seismic earth pressure increment, we used the AASHTO A11.3.3 General Limit Equilibrium Method (GLE). In this method a lateral force is applied at 1/3H of wall (one-third of the height of wall). This force is increased until the static factor of safety is equal to 1.0. Then the horizontal seismic coefficient is applied as well as a lateral force 1/2H. This force is increased until the factor of safety equals 1.0. This force is equivalent to the lateral seismic earth load increment which can be distributed evenly long the height of wall. We used the horizontal coefficient of 0.54g (i.e., 2/3 x 0.81g) in our calculations. The calculation of seismic earth pressure increment is shown in **Plate 7**.

We recommended the permanent shoring system along the east side of the proposed building (estimated lateral spreading/slope instability lateral extent) be designed to retain a 12-foot high of soils (type A), assuming the loss of adjacent ground support due to slope failure. The small portion of the shoring system located to further east of the estimated lateral spreading/slope instability lateral extent (southeast of the building) should be designed to retain a 18-feet high of soils (type B).

The current foundation includes 14-inch square driven piles that are designed to restrict the building lateral movement to ½ inch. The shoring wall is a tangent wall with 24-inch diameter cast in place piles that is structurally connected to the pile caps to limit the wall movement at the top. The portion of the wall that retains 12 feet of soil is 104 linear foot while the portion of the wall at the southeast of the building that retains 18 foot of soil is 46 liner foot.

The lateral static earth pressures as shown on **Plate 8** are obtained based on the State of California Department of Transportation Trenching and Shoring Manual, Chapter 7 for restrained shoring systems. We used Lpile software to calculate the effect of restraining the shoring wall on the pile foundation system.

To investigate the impact of the shoring wall, the earth pressures shown on **Plate 8** are applied on a 24-inch diameter single pile with free head condition. It is assumed that the pile head is free to move laterally. In the next step, a lateral load was applied at the pile head against the direction of movement, and it was increased until the pile movement was ½ inch. This load will be carried by building pile foundations.

Table 6: Impact of the Shoring Wall on Building Foundations

Wall Type (Wall Linear Length- ft)	Number of Piles per Wall	Load to Restrict the Wall Movement to ½ inch (kips)	Total Load (kips)
A (104)	52	10.9	566
B (46)	23	26.7	614

In summary the total load of 1180 kips (562 + 614 kips) will be carried by the building foundation in addition to budling lateral load during earthquake event. Because all piles are connected with pile caps, grade beams, and structural slab, the load will be carried by all foundation elements. We recommend dividing the total load evenly between all building foundation piles.

The deflection, shear force, and moment diagrams of piles for wall type A and B are presented in **Plate 9**. For the Lpile analysis, concrete f'c=5000 (psi) and pile is elastic section with 50%EI.

The calculations package and drawings of the pile foundation and shoring wall will be provided for your review.

CGS Comment 31

Conditional Geologic Assessment: Selected geologic hazards addressed by the consultant are listed below:

I. *Clays and cyclic softening:* ***Additional information is requested****.* ***The consultants should evaluate and provide explicit discussion of the potential for cyclic softening and loss of strength in the very soft to soft and weak YBM soils at the site due to seismic shaking****. If the consultants determine that significant potential for cyclic softening and loss of strength exists at the* *site, the associated hazards of lateral spreading/instability of slopes and impact upon the project design and the proposed improvements should be further addressed as noted previously in Items 11A, 24, 26, and 28.*

Fugro: We have revised our slope stability analysis to reflect the potential for strength degradation from cyclic softening in Bay Mud (refer to the response to **Comment No.26**).

Closing

The conclusions and recommendations provided in this letter are meant to supplement the relevant foundation recommendations included in our March 14, 2016, report. Our conclusions and recommendations are solely professional opinions and were made in accordance with generally accepted local and current geotechnical engineering principles and practices. We make no warranty, either express or implied. Should you have any questions or require additional information, please contact us.

Sincerely,

Reza Rahimnejad, PhD
Project Engineer

Ronald L. Bajuniemi, PE, GE
Principal Consultant

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JU	Jose Ugalde	Principal Engineer

List of Plates

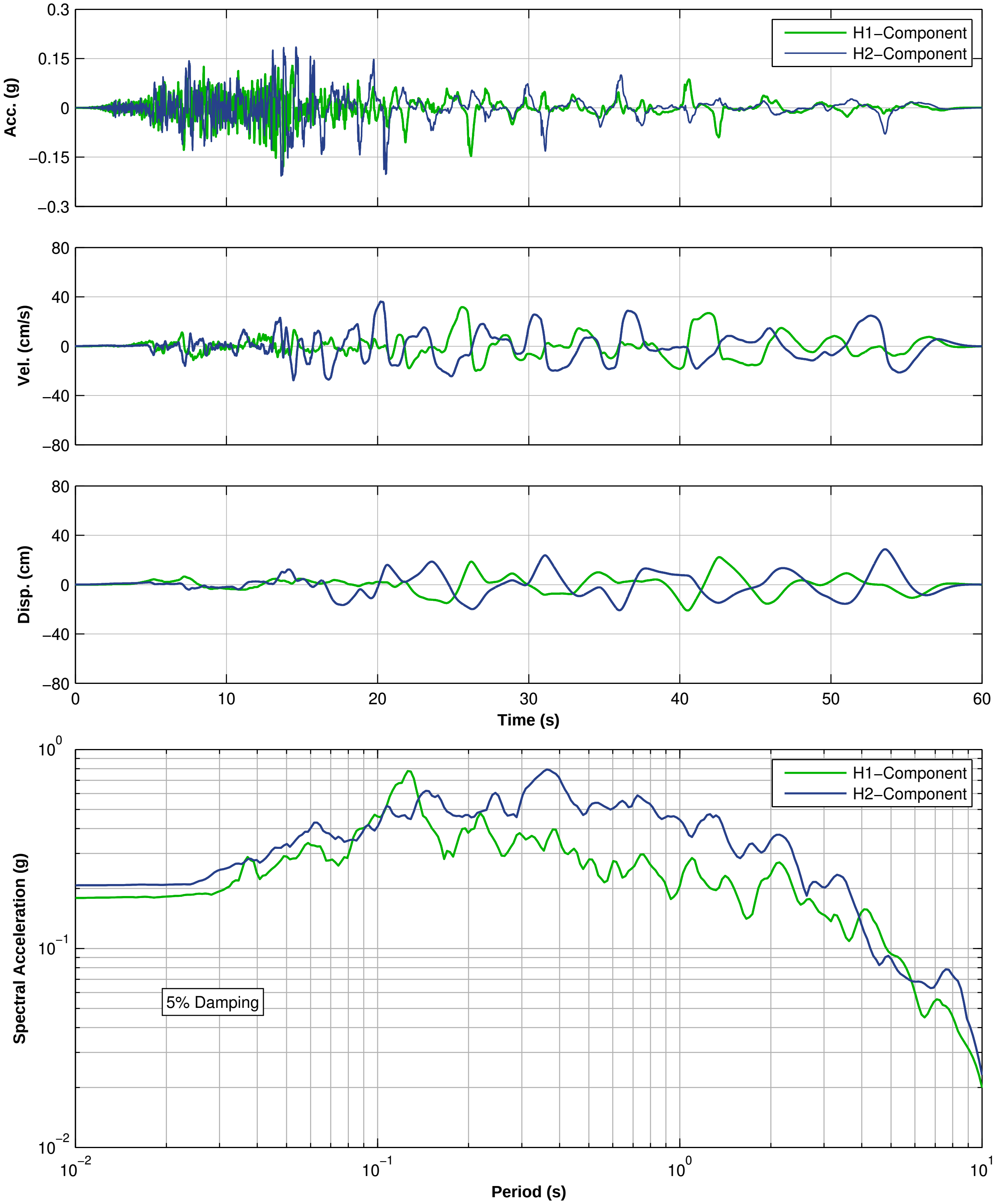


Plate 1.1: Seed Motion

1987, Superstition Hills-02 EQ, Imperial Valley Wildlife Liquefaction Array, RSN0729

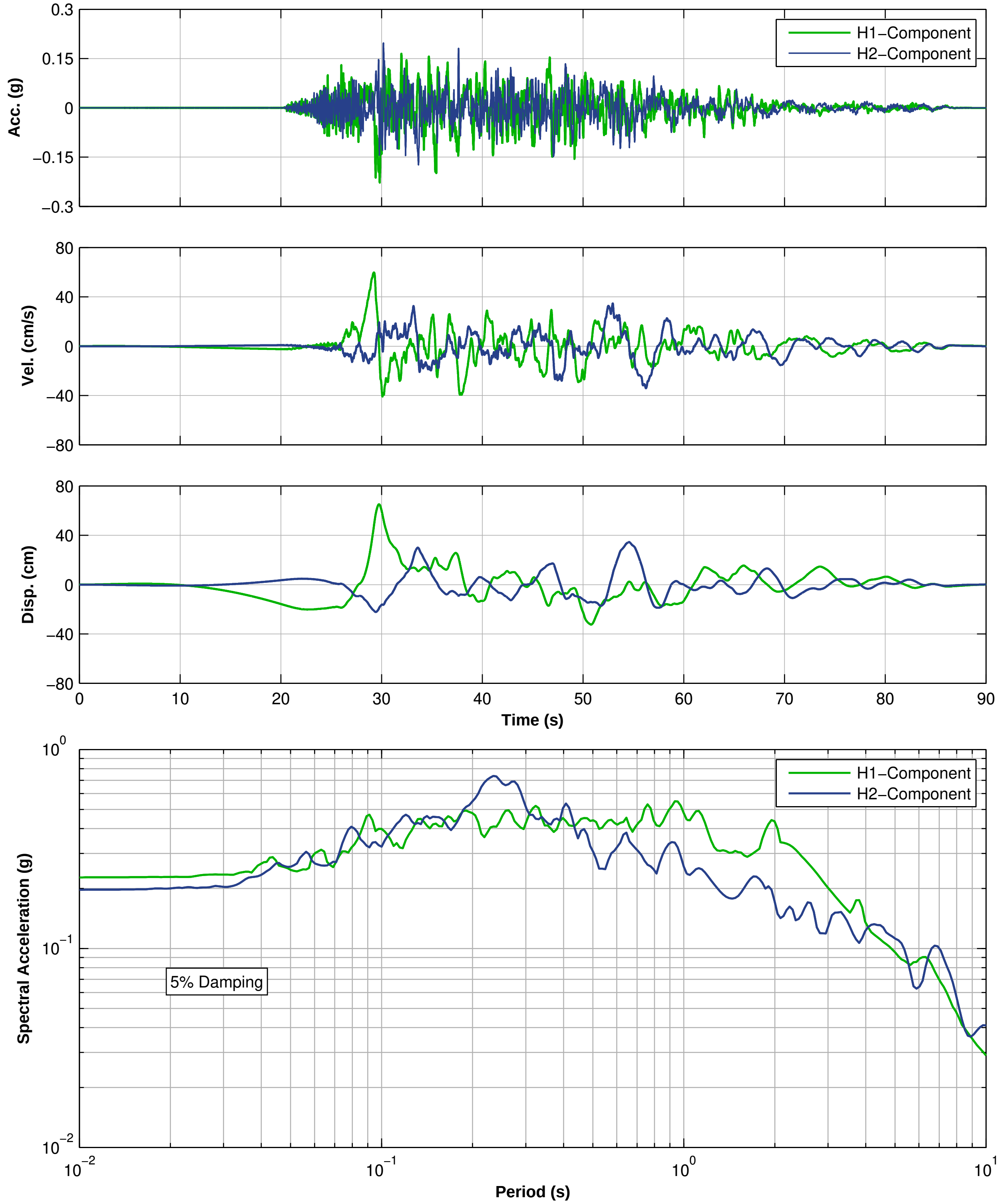


Plate 1.2: Seed Motion

1999, Chi-Chi, Taiwan EQ, TCU120, RSN1545

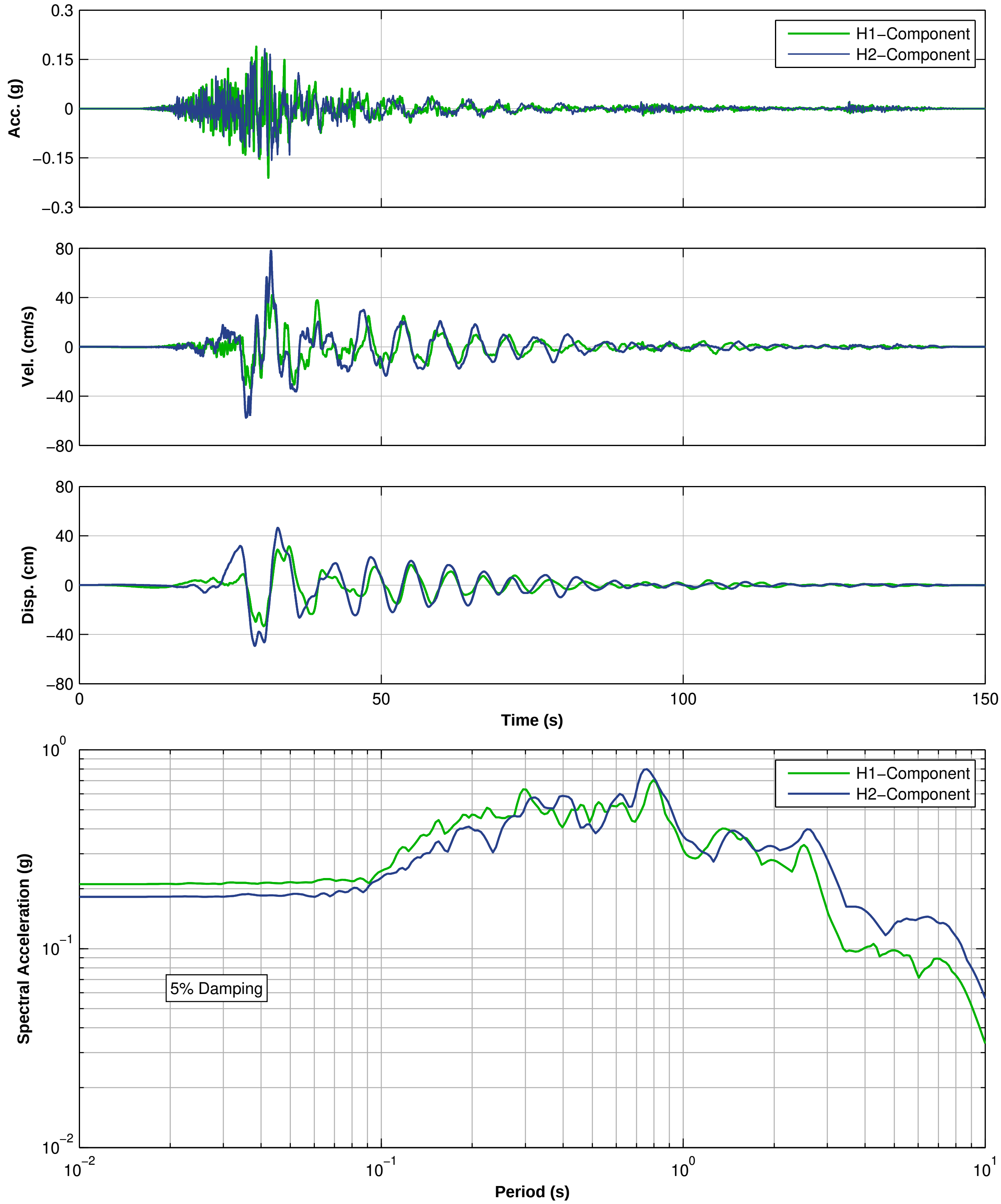


Plate 1.3: Seed Motion
2010, Darfield New Zealand EQ, Papanui High School, RSN6952

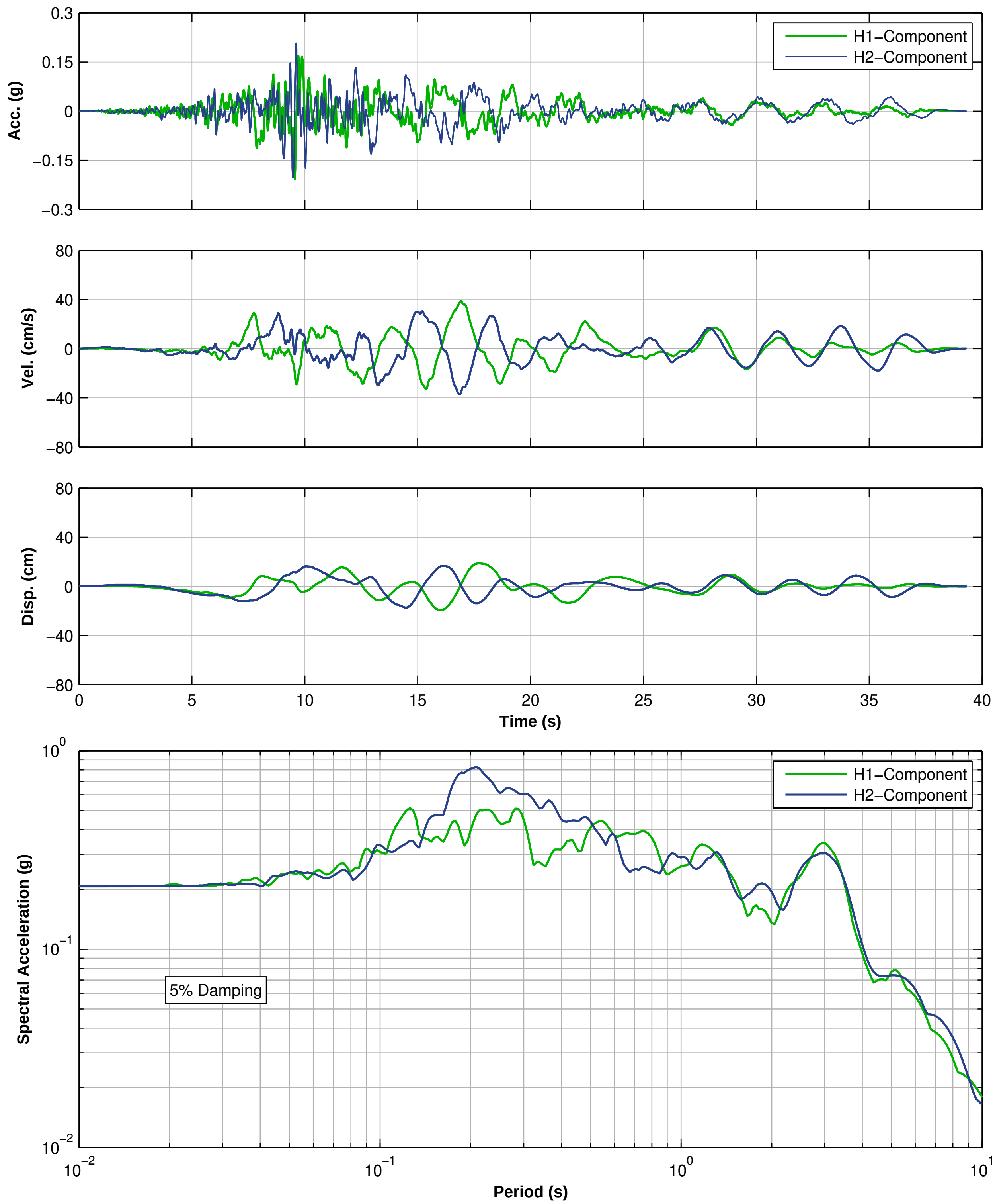


Plate 1.4: Seed Motion

1989, Loma Prieta EQ, Sunnyvale - Colton Ave., RSN0806

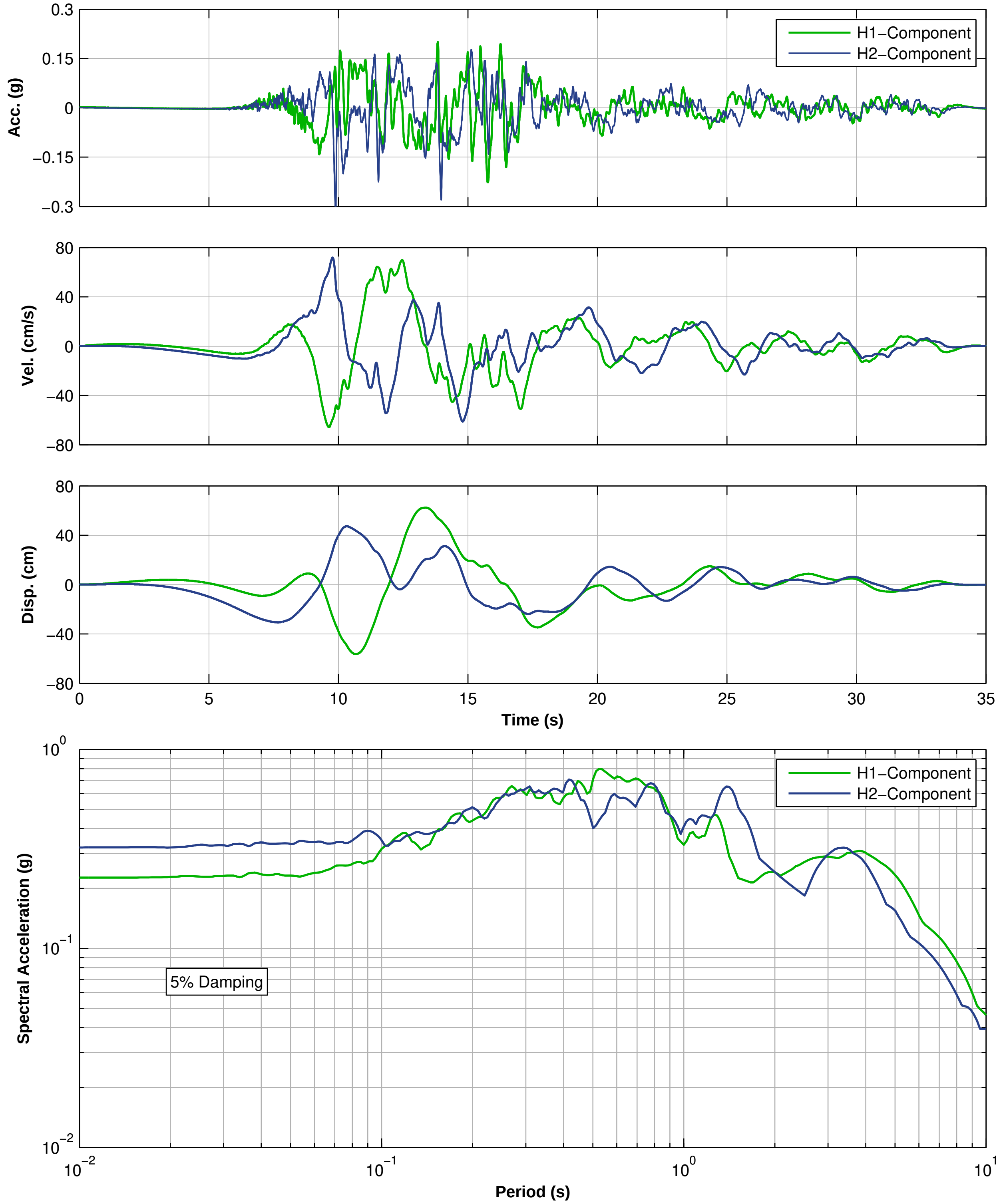


Plate 1.5: Seed Motion
1999, Kocaeli, Turkey EQ, Yarimca, RSN1176

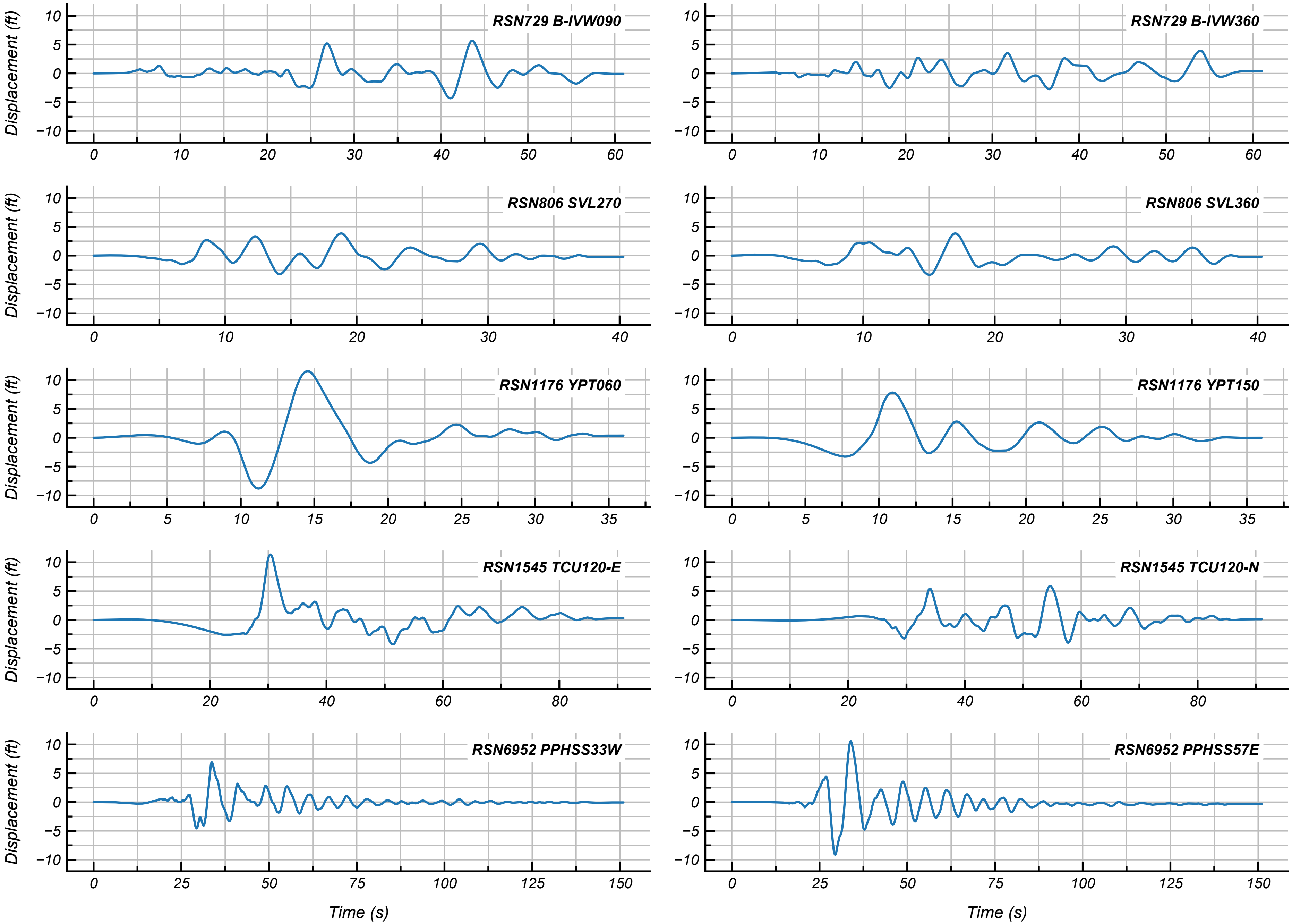


Plate 2.1: Displacement time histories at surface for P1

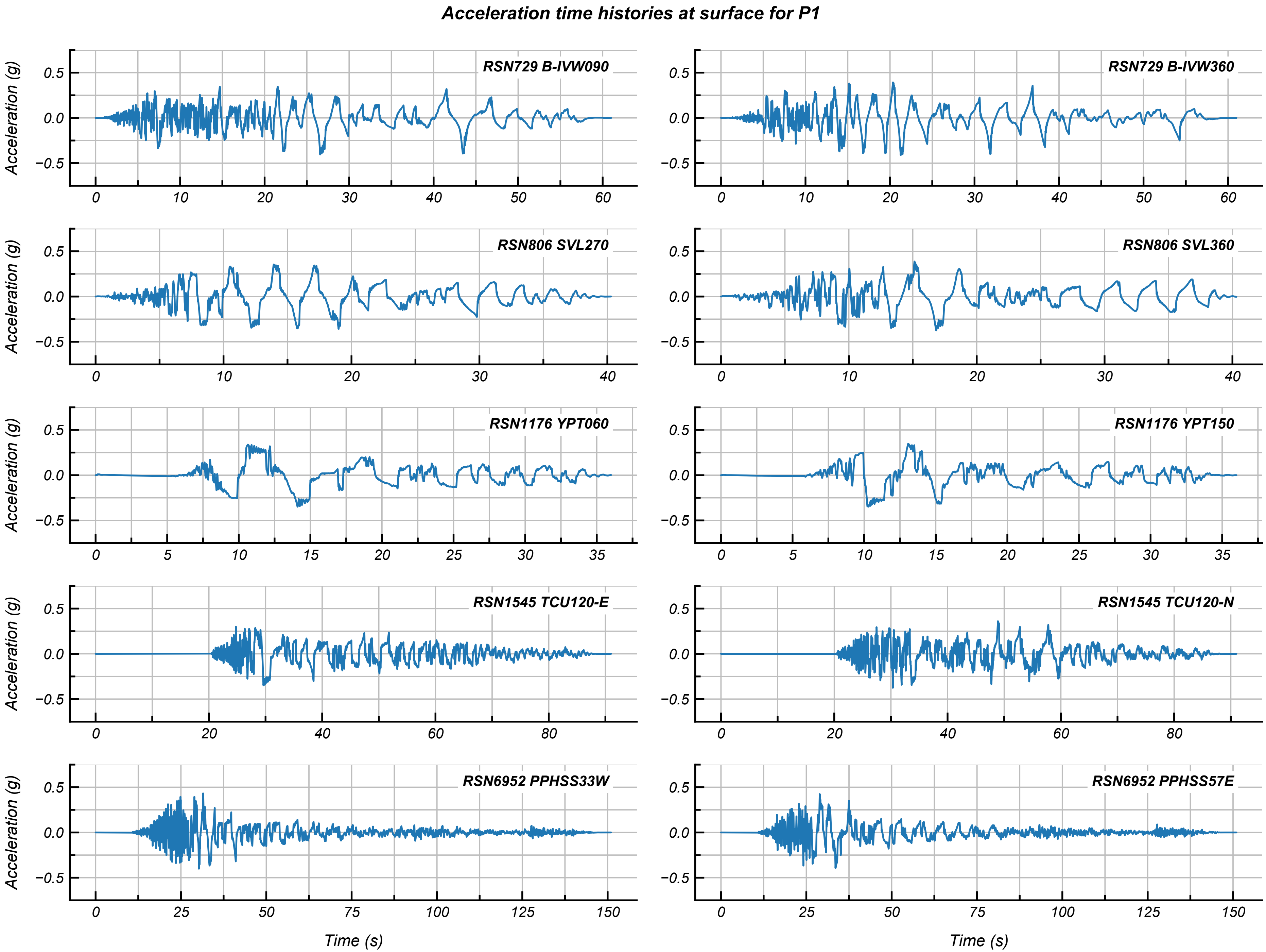
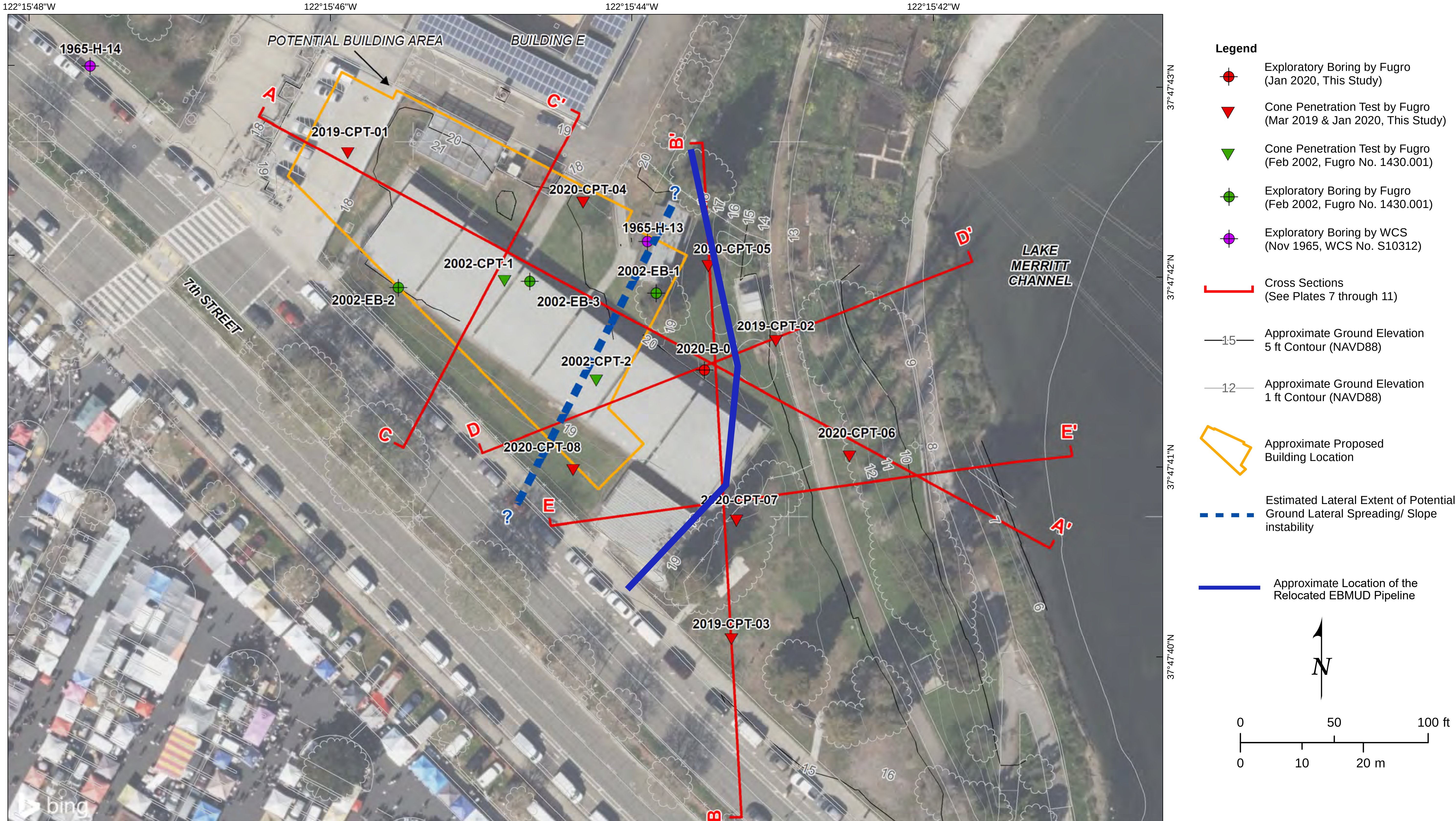


Plate 2.2: Displacement time histories at surface for P1

PERALTA COMMUNITY COLLEGE DISTRICT
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA



Aerial imagery from Bing Maps. Topo contours provided by CSW/Stuber-Stroeh, April 2019. Proposed building location provided by Noll and Tam Architects, January 2020.

SITE PLAN

PLATE 3

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2020_01_09_GeotechReport\MXD\3_SitePlan.mxd, 6/23/2021, e.isleyen

Fugro Document No. 04.72190021

Seismic Coefficient Used in Screening Analysis of Seismic Slope Stability

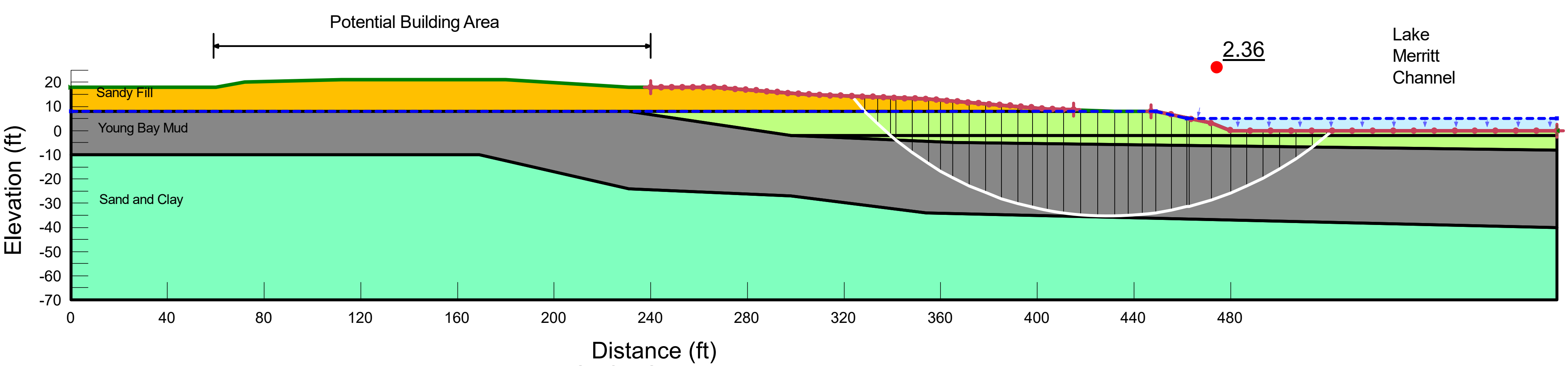
Recommended by DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California (June 2002)

Maximum Horizontal Acceleration	**MHAr/g =**	0.54	
Mode Magnitude	**M =**	7.6	
Mode Distance	**r =**	6.8	km
Threshold	**u =**	76.2	cm
Significant Duration of Shaking	**D_{5-95} =**	24.0	sec
	C_1 =	0.411	
	C_2 =	0.0837	
	C_3 =	0.00208	
	ε_T =	0.437	
Mean Period	**Tm =**	0.53	sec
Factor of Nonlinear Response of Materials above Slide Plane	**NRF =**	0.90	
	feq =	0.27	
Seismic Coefficient	**k =**	0.15	

Plate 4: Determination of Horizontal Sesimic Coefficient

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

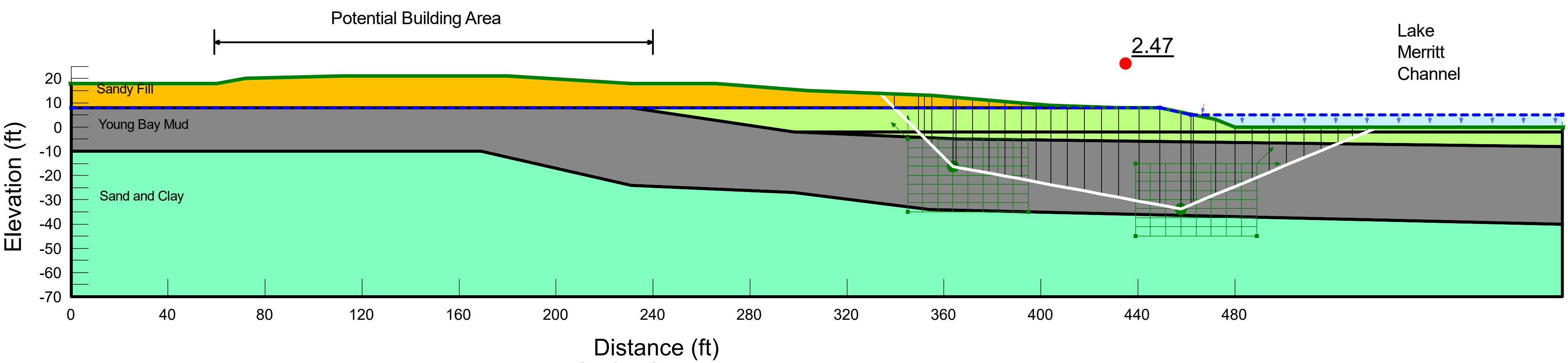


SECTION A-A'

Plate 5-1

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR -Non-Circular.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

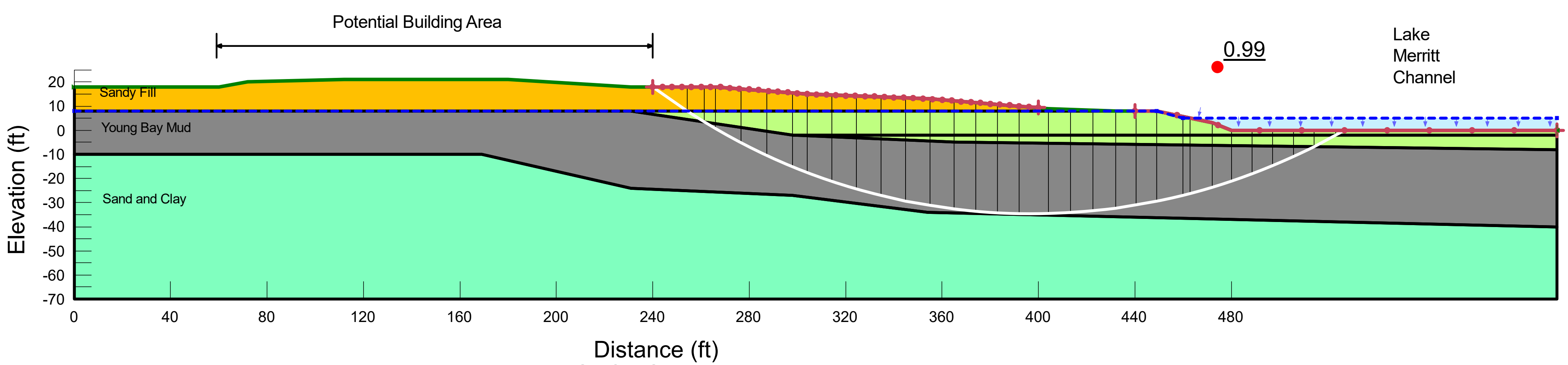


SECTION A-A'

Plate 5-2

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.1
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

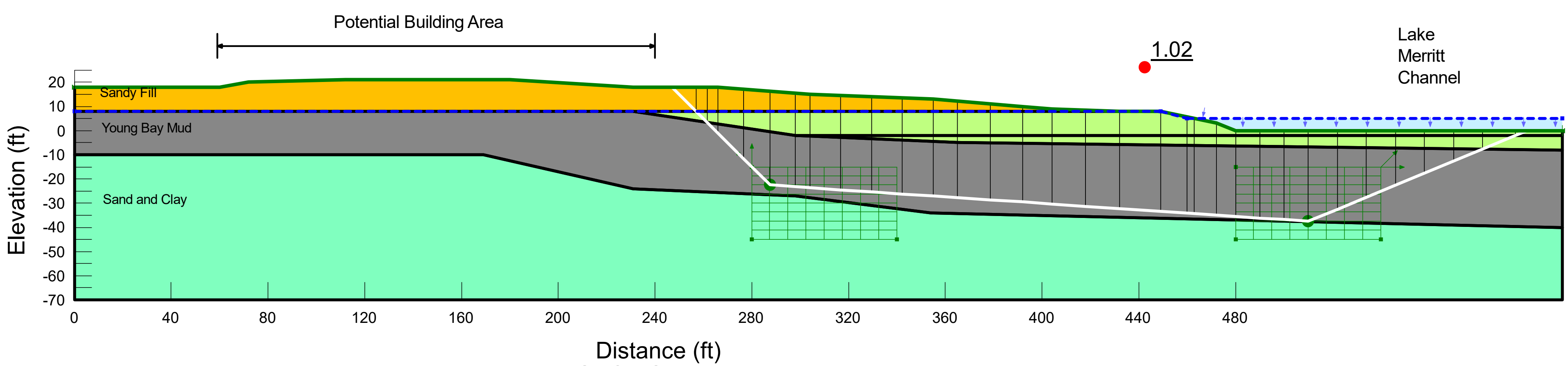


SECTION A-A'

Plate 5-3

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR -Non-Circular.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.1
Method: Spencer

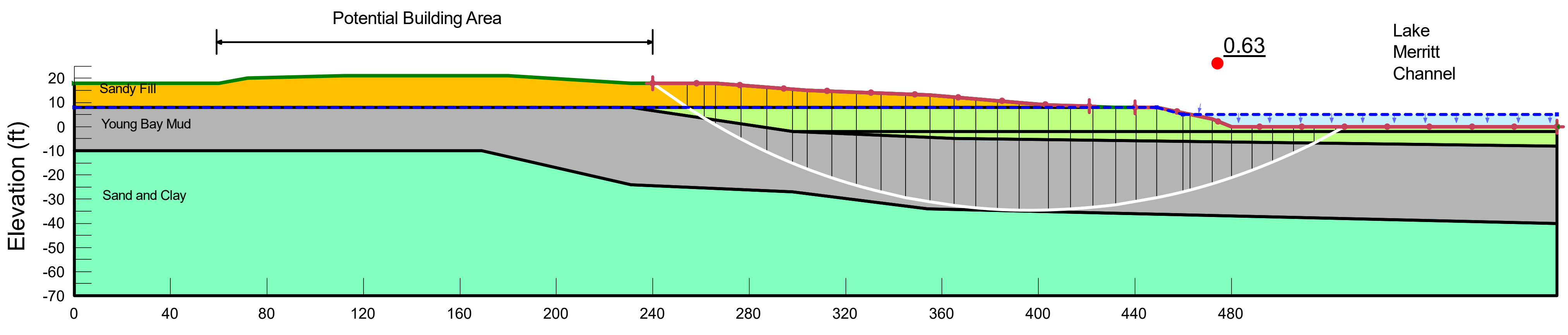
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350



SECTION A-A'

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR.gsz
Description: Case 3 - Pseudo-Static k = 0.15g Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280

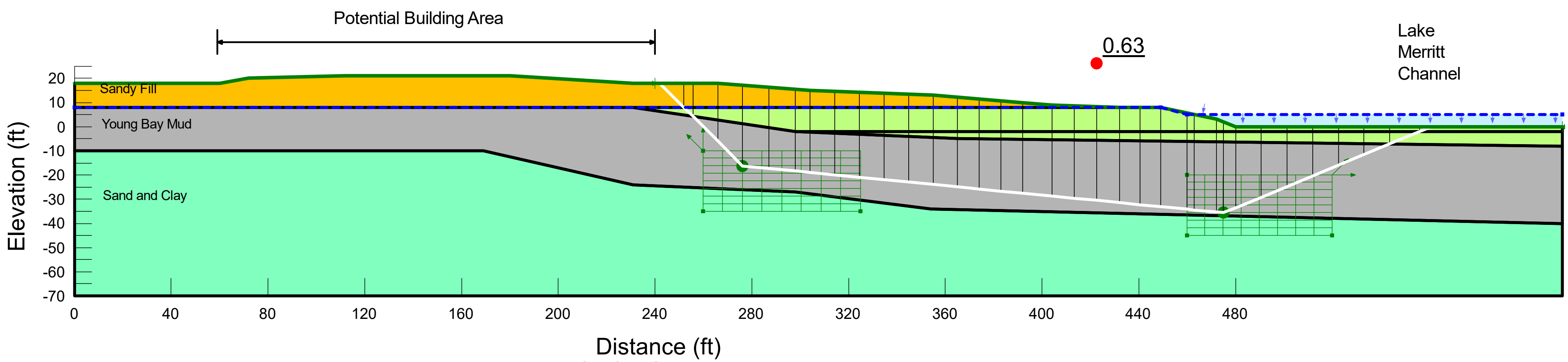


SECTION A-A'

Plate 5-5

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR -Non-Circular.gsz
Description: Case 3 - Pseudo-Static k = 0.15g Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280

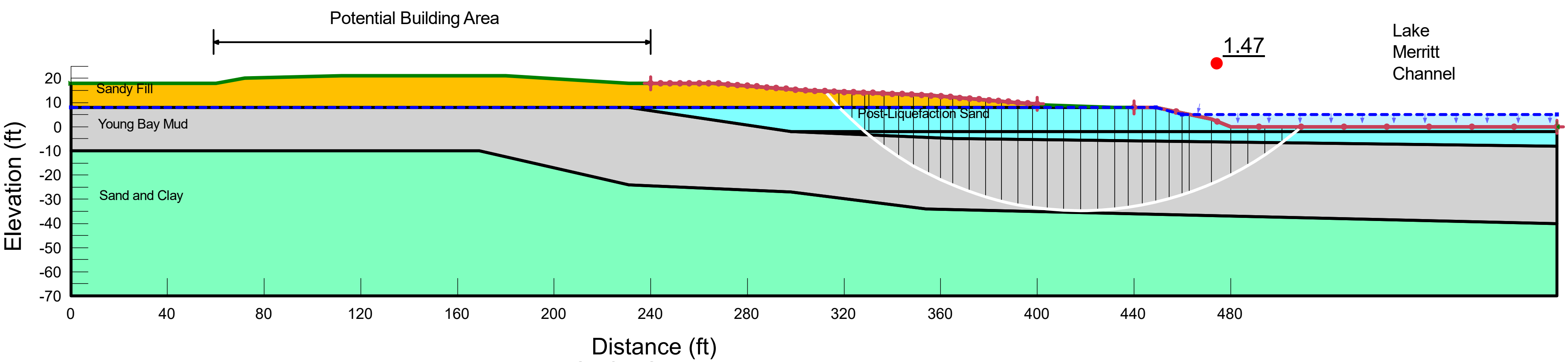


SECTION A-A'

Plate 5-6

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Piezometric Line	Cohesion' (psf)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (psf)
	Post-Liquefaction Sand	S=f(datum)	110	100	20	500	8	1				
	Sand and Clay	Mohr-Coulomb	130					1	0	40		
	Sandy Fill	Mohr-Coulomb	120					1	0	35		
	Young Bay Mud (Post Earthquake)	S=f(overburden)	90					1			0.17	210

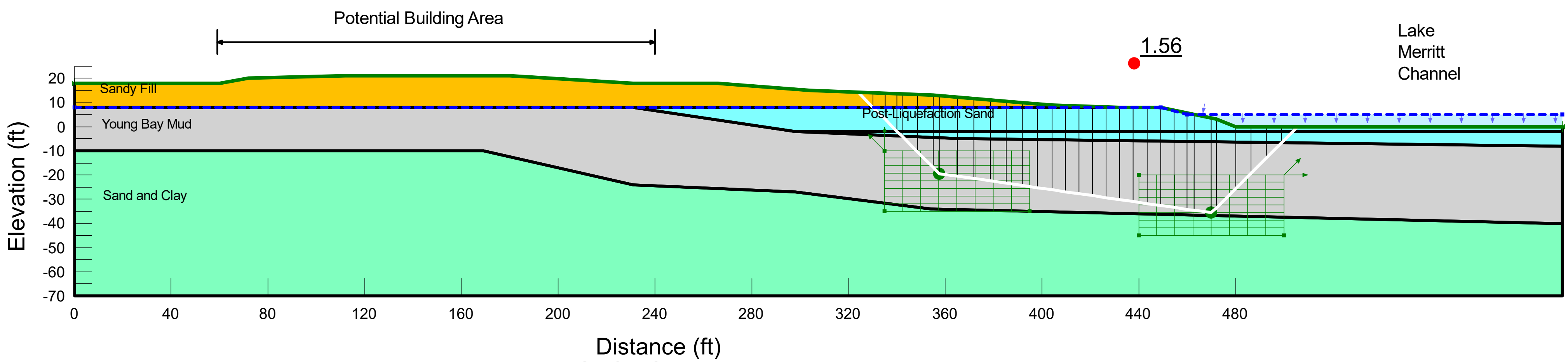


SECTION A-A'

Plate 5-7

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR -Non-Circular.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Piezometric Line	Cohesion' (psf)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (psf)
	Post-Liquefaction Sand	S=f(datum)	110	100	20	500	8	1				
	Sand and Clay	Mohr-Coulomb	130					1	0	40		
	Sandy Fill	Mohr-Coulomb	120					1	0	35		
	Young Bay Mud (Post Earthquake)	S=f(overburden)	90					1			0.17	210

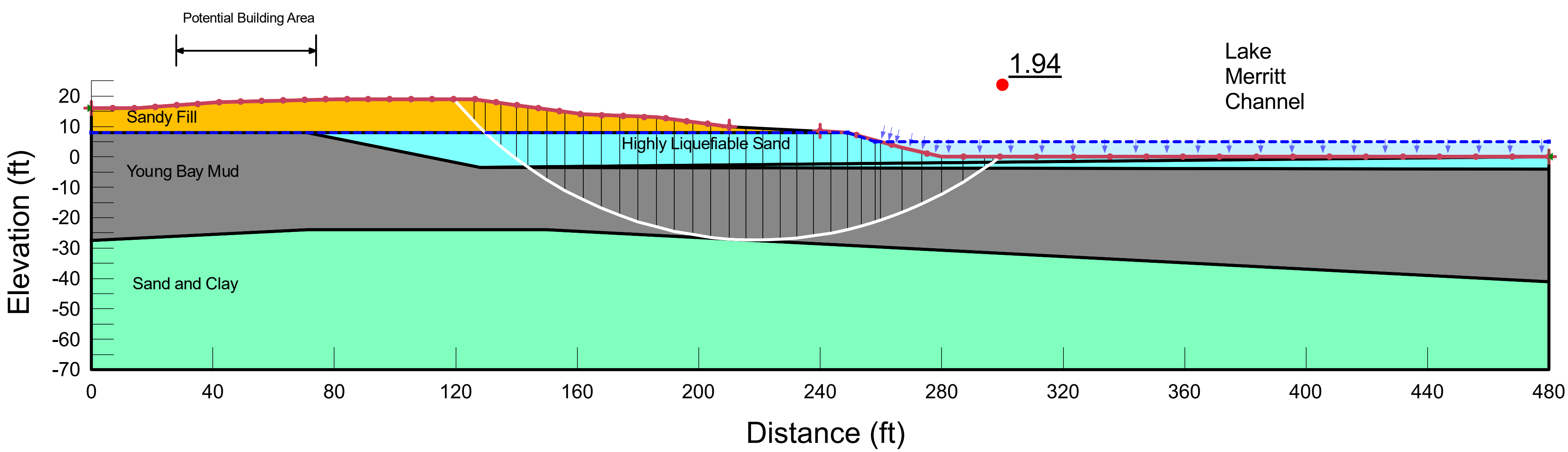


SECTION A-A'

Plate 5-8

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

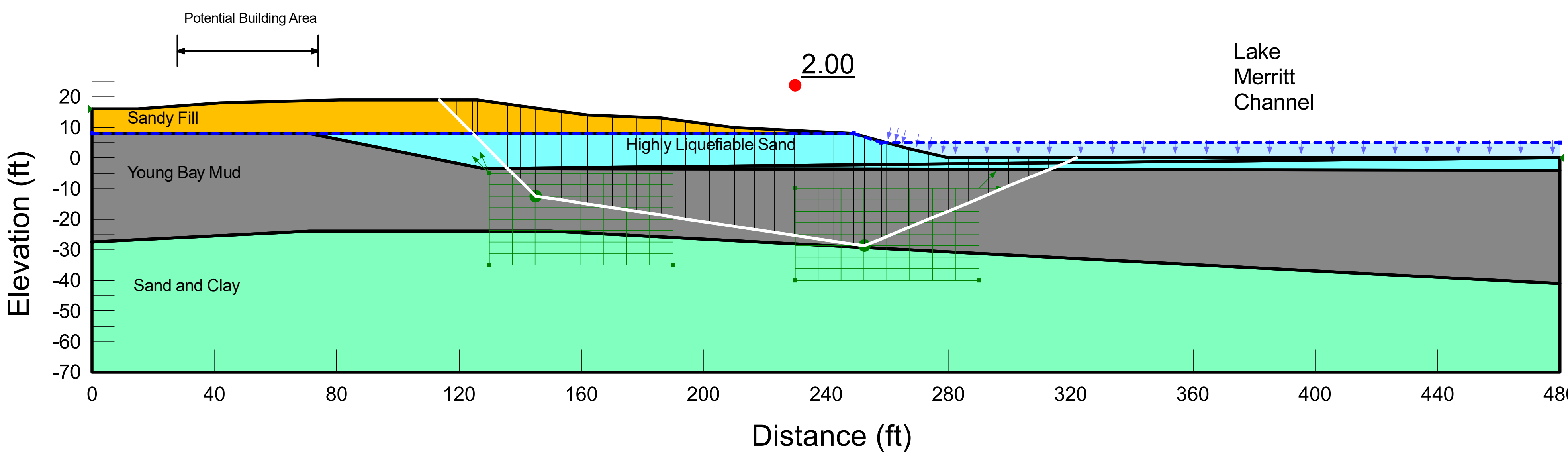


SECTION D-D'

Plate 5-9

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR - Non-Circular.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer:
Slip Surface Option: Block search

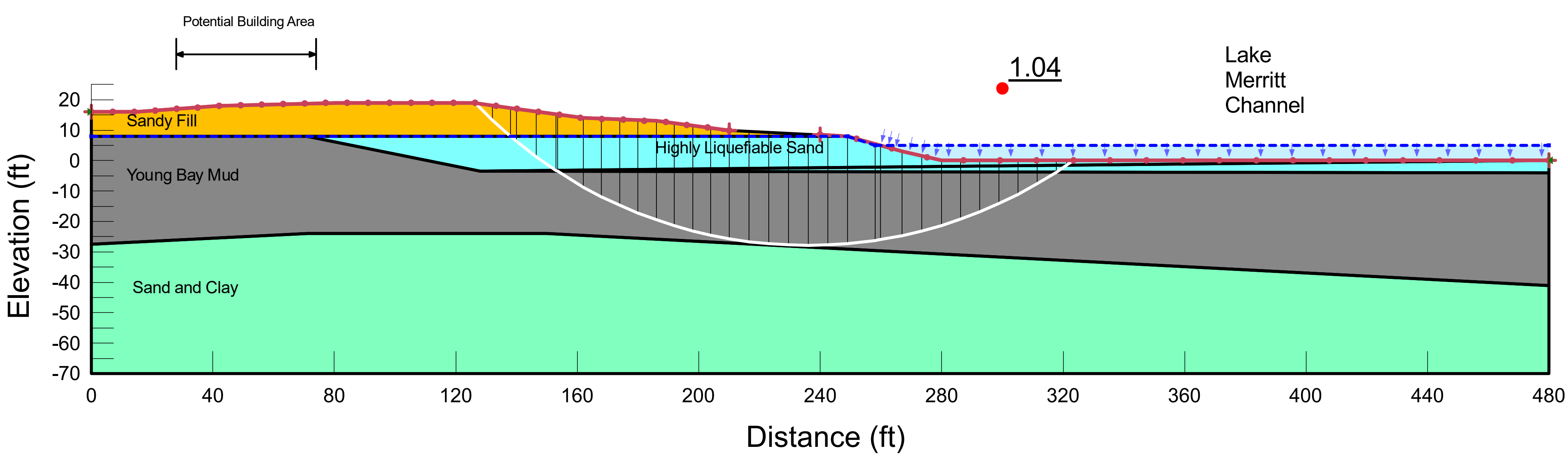
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350



SECTION D-D'

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.09
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

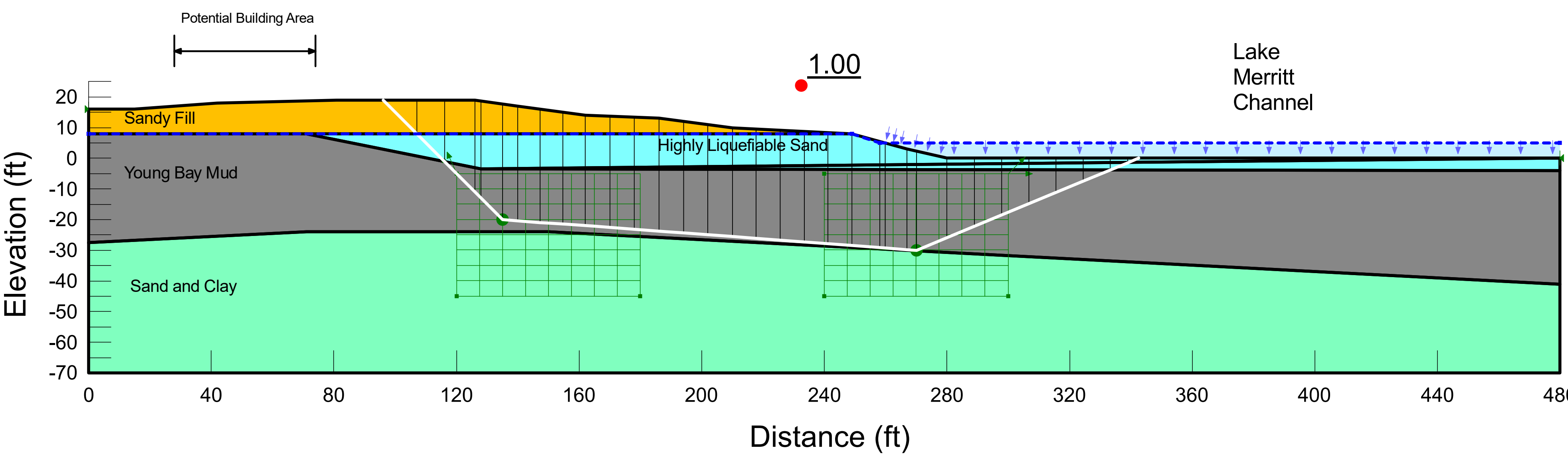


SECTION D-D'

Plate 5-11

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR - Non-Circular.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.1
Method: Spencer:
Slip Surface Option: Block search

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

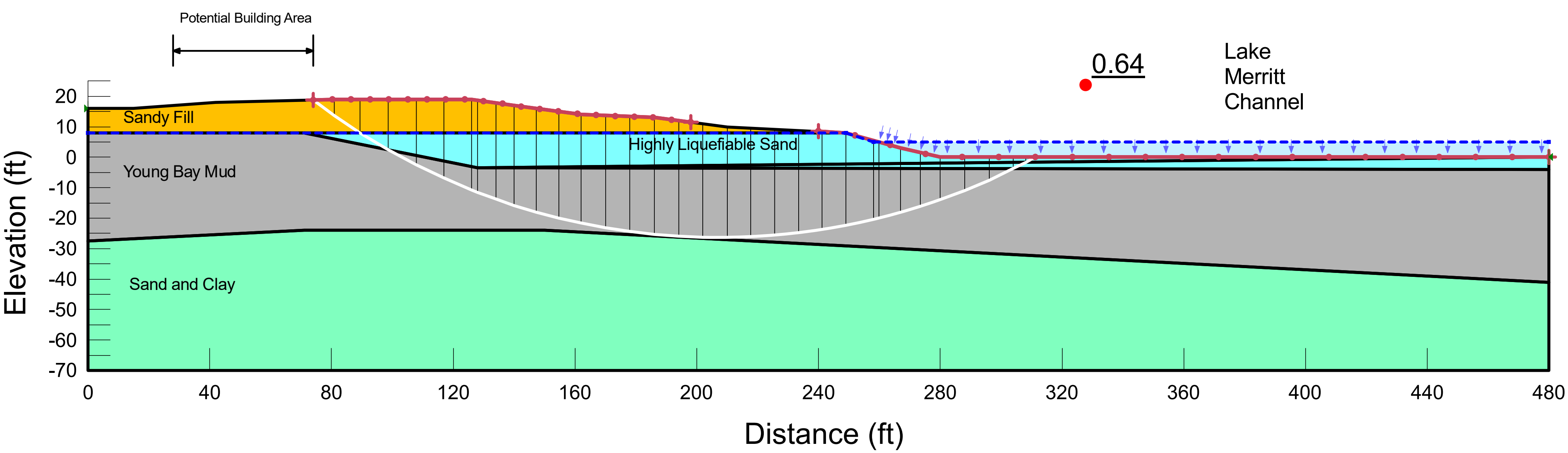


SECTION D-D'

Plate 5-12

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR.gsz
Description: Case 3a - Pesudo-Static k = 0.15g - Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280

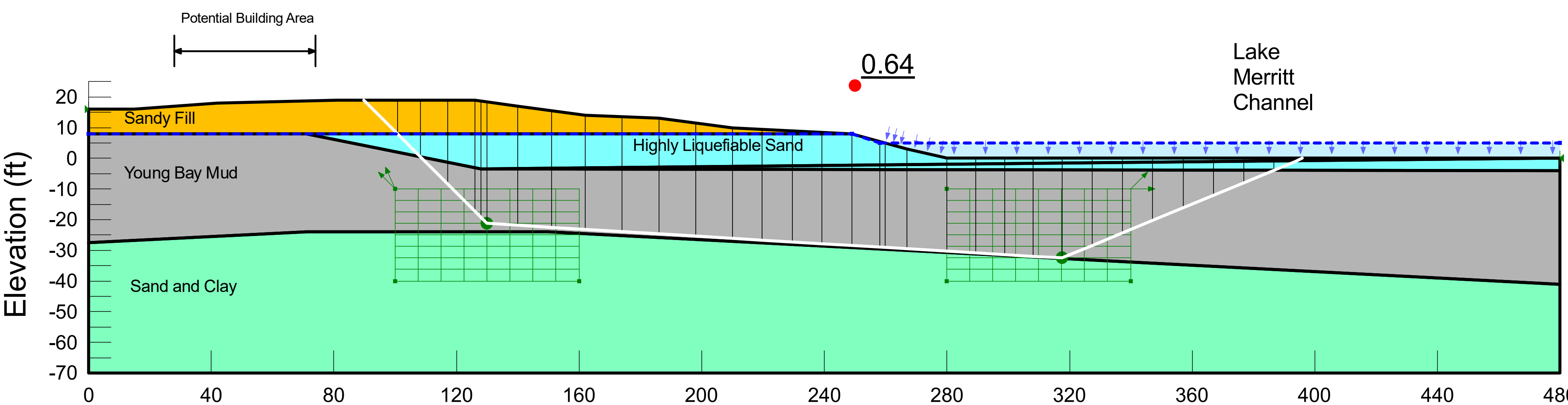


SECTION D-D'

Plate 5-13

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR - Non-Circular.gsz
Description: Case 3a - Pesudo-Static k = 0.15g - Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer:
Slip Surface Option: Block search

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280

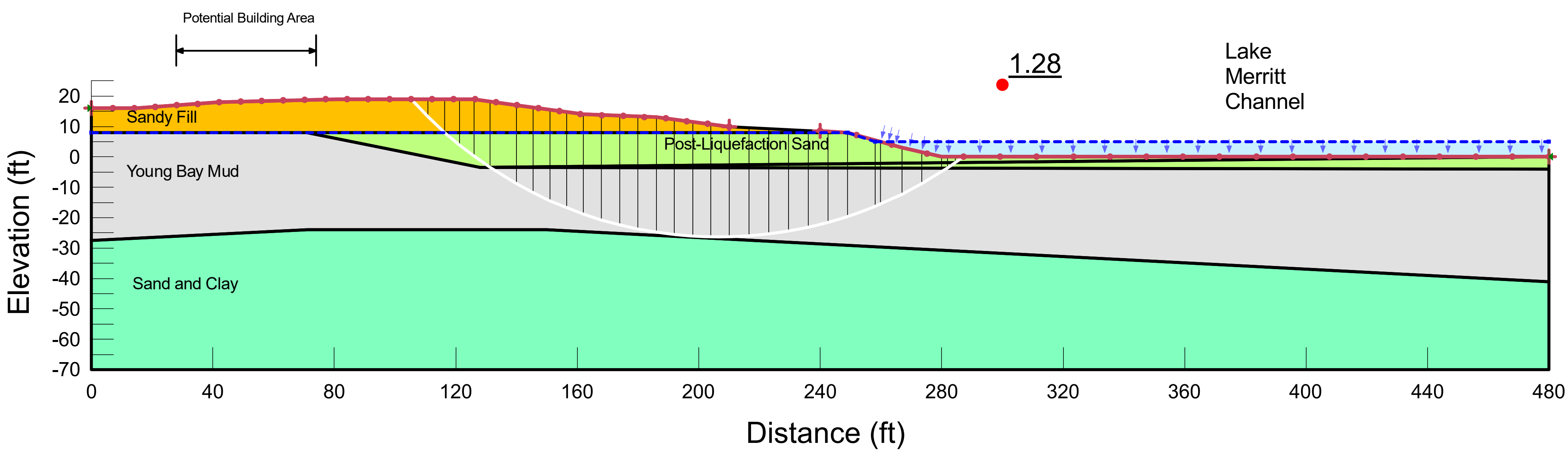


SECTION D-D'

Plate 5-14

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Piezometric Line	Cohesion' (psf)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (psf)
	Post-Liquefaction Sand	S=f(datum)	110	100	20	500	8	1				
	Sand and Clay	Mohr-Coulomb	130					1	0	40		
	Sandy Fill	Mohr-Coulomb	120					1	0	35		
	Young Bay Mud (Post Earthquake)	S=f(overburden)	90					1			0.17	280

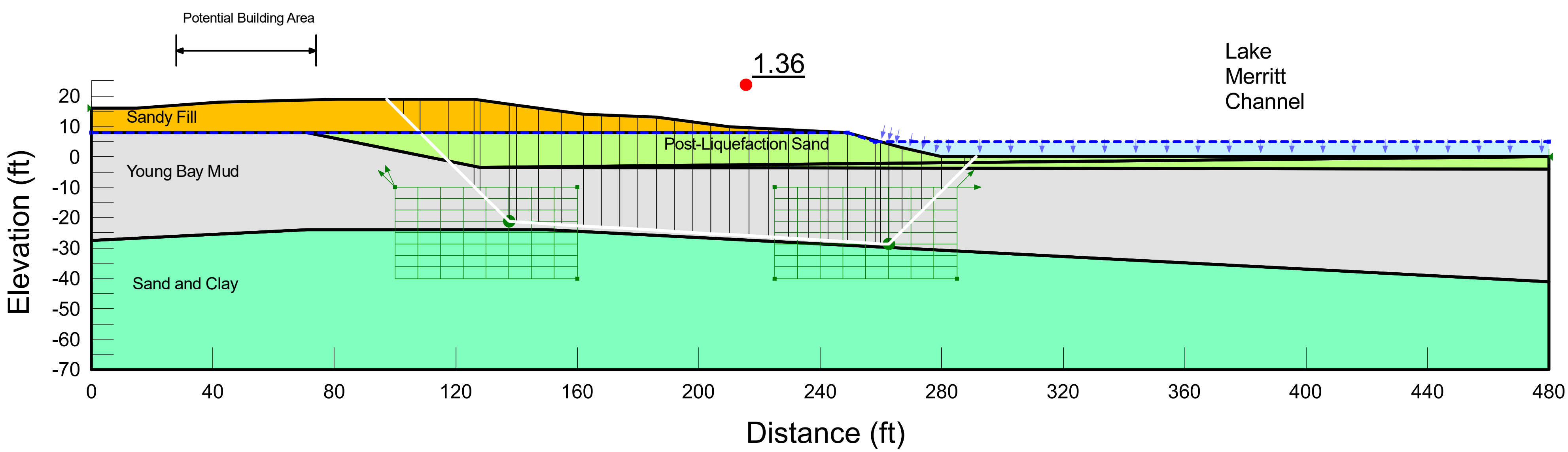


SECTION D-D'

Plate 5-15

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR - Non-Circular.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer:
Slip Surface Option: Block search

Color	Name	Model	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Piezometric Line	Cohesion' (psf)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (psf)
	Post-Liquefaction Sand	S=f(datum)	110	100	20	500	8	1				
	Sand and Clay	Mohr-Coulomb	130					1	0	40		
	Sandy Fill	Mohr-Coulomb	120					1	0	35		
	Young Bay Mud (Post Earthquake)	S=f(overburden)	90					1			0.17	280

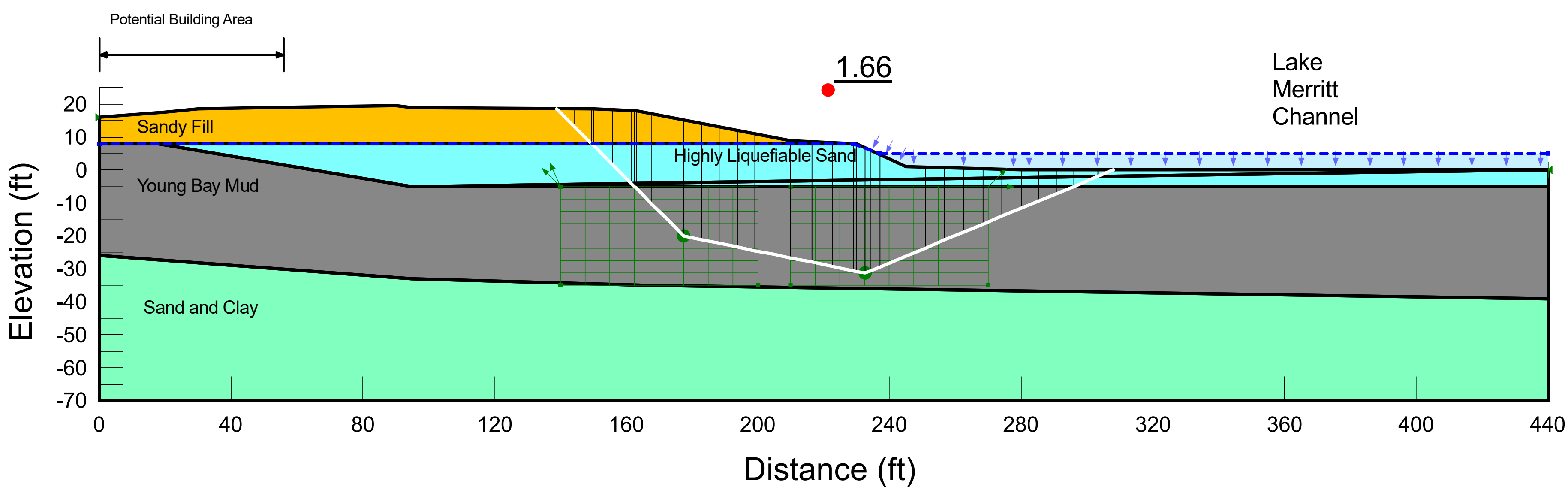


SECTION D-D'

Plate 5-16

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR - Non-Circular.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

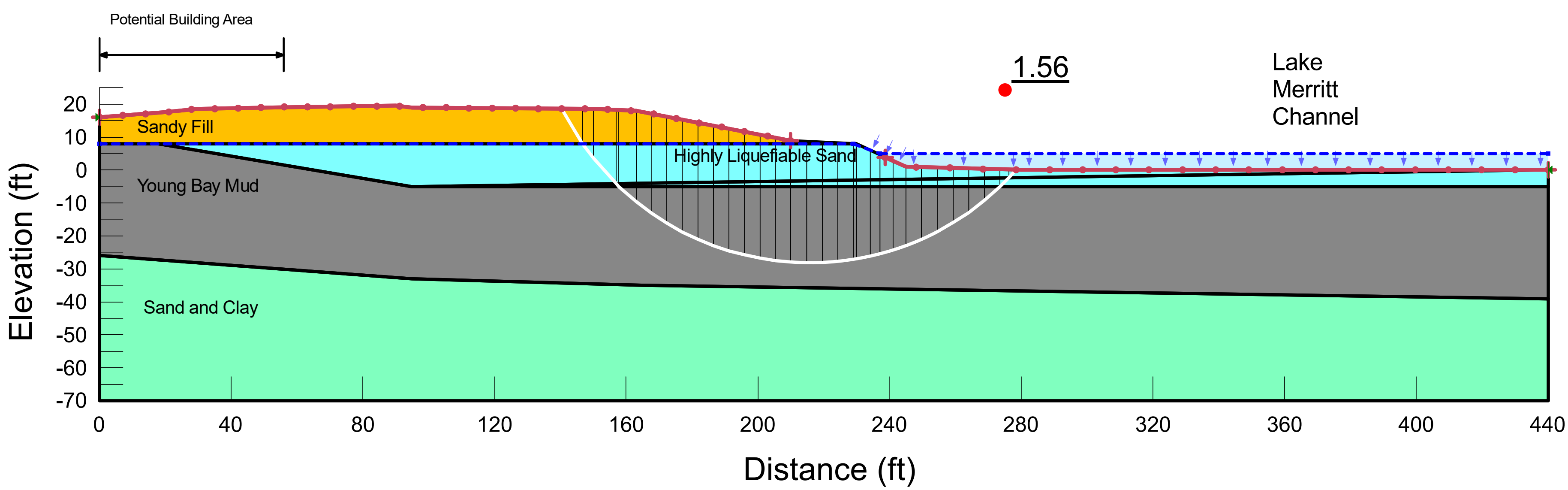
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350



SECTION E-E'

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

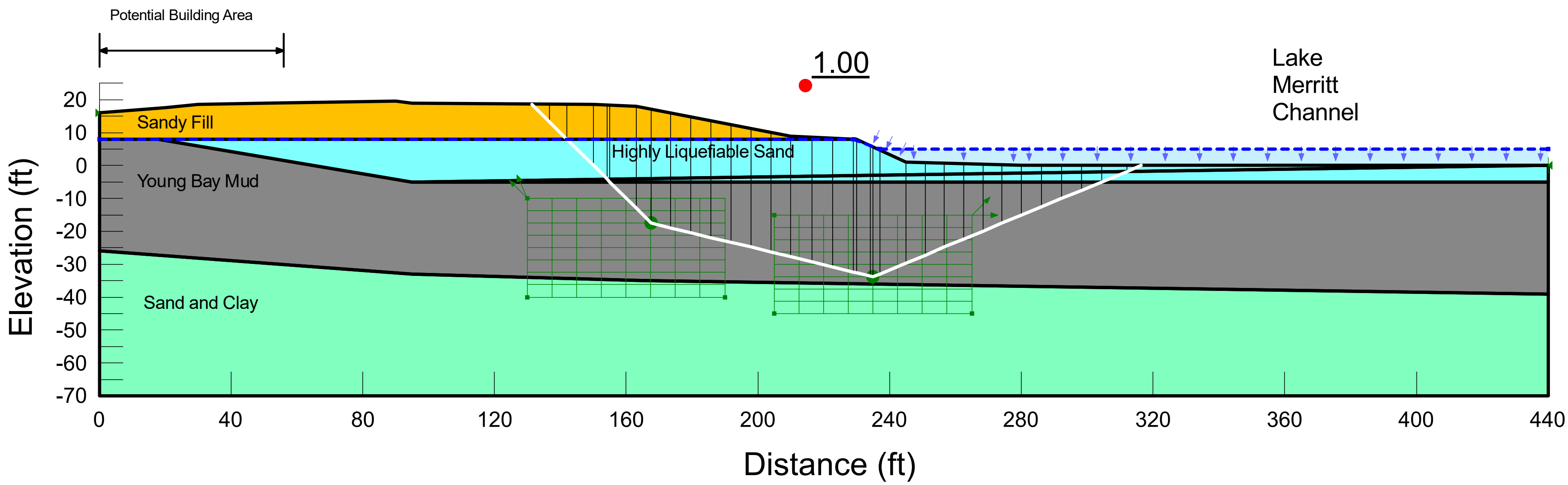


SECTION E-E'

Plate 5-18

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR - Non-Circular.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.09
Method: Spencer

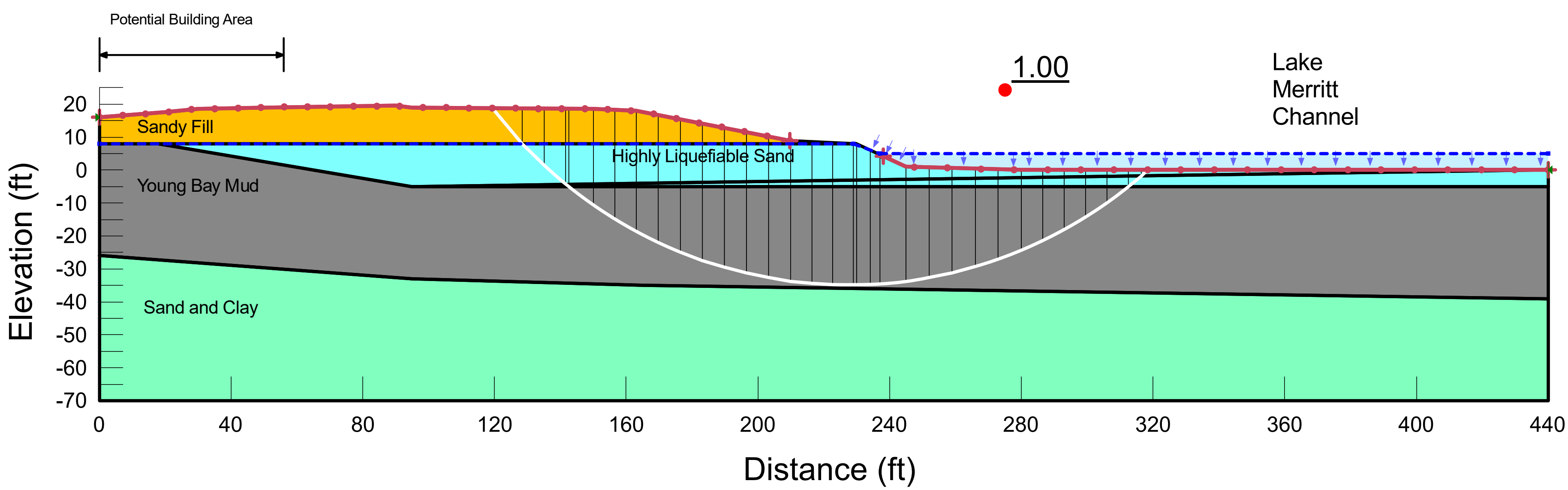
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350



SECTION E-E'

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.08
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.28	350

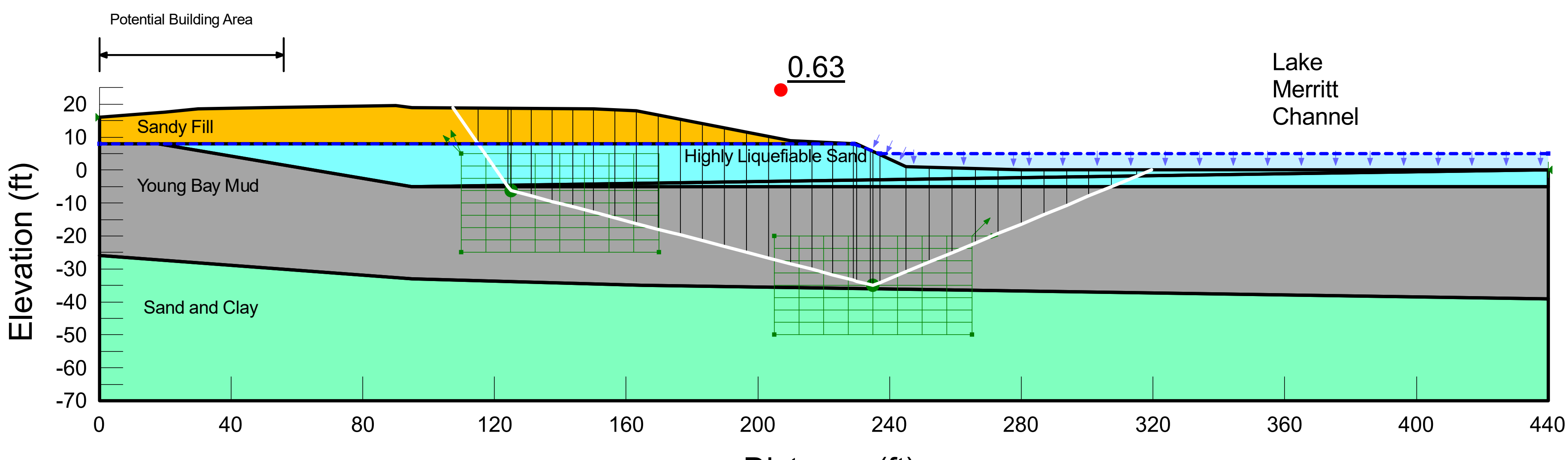


SECTION E-E'

Plate 5-20

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR - Non-Circular.gsz
Description: Case 3a - Pesudo-Static k = 0.15g - Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280

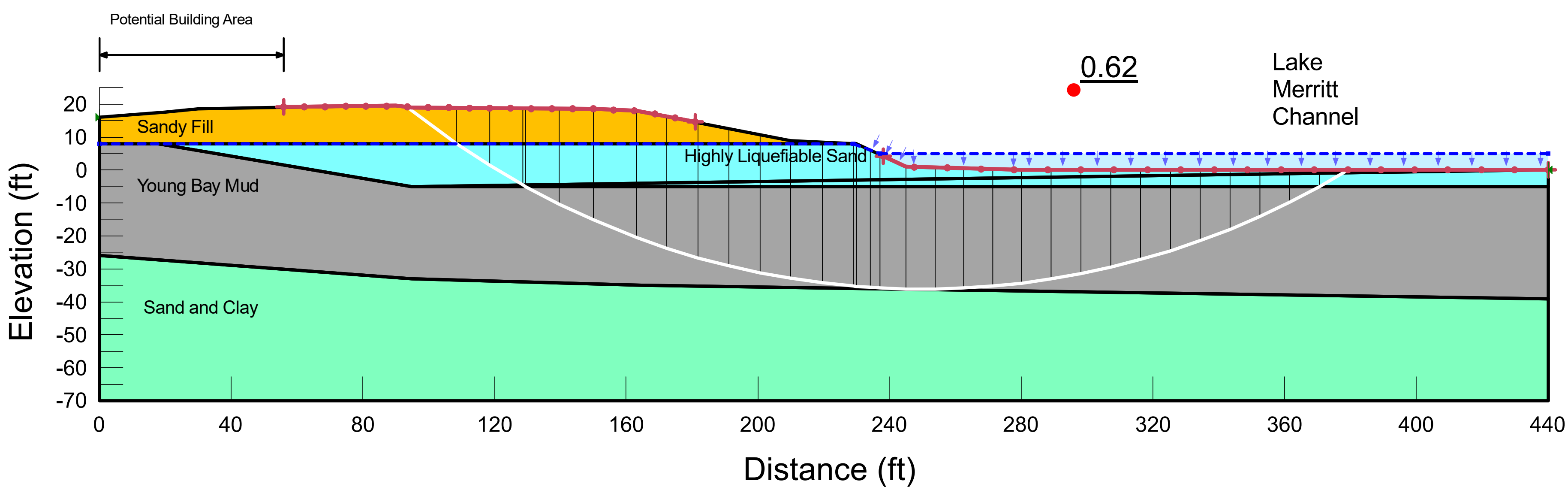


SECTION E-E'

Plate 5-21

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR.gsz
Description: Case 3a - Pesudo-Static k = 0.15g - Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer

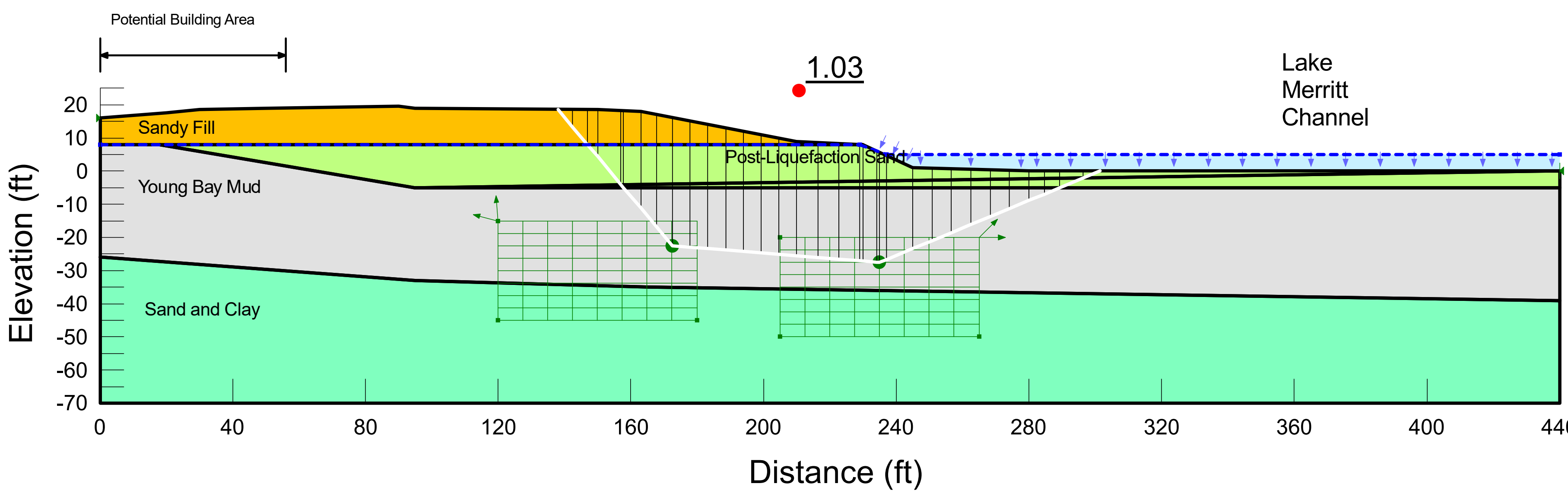
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280



SECTION E-E'

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR - Non-Circular.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Piezometric Line	Cohesion' (psf)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (psf)
	Post-Liquefaction Sand	S=f(datum)	110	100	20	500	8	1				
	Sand and Clay	Mohr-Coulomb	130					1	0	40		
	Sandy Fill	Mohr-Coulomb	120					1	0	35		
	Young Bay Mud (Post Earthquake)	S=f(overburden)	90					1			0.17	210

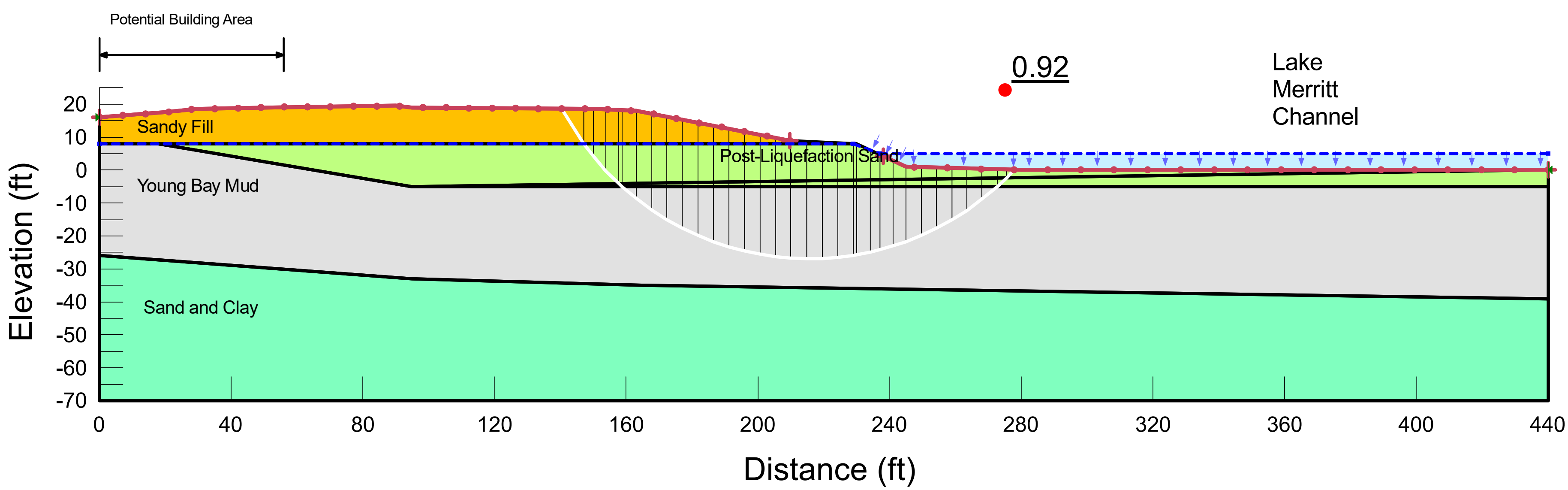


SECTION E-E'

Plate 5-23

Title: Laney College Library Learning Resource Center
File Name: Section E-06172021_RR.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Piezometric Line	Cohesion' (psf)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (psf)
	Post-Liquefaction Sand	S=f(datum)	110	100	20	500	8	1				
	Sand and Clay	Mohr-Coulomb	130					1	0	40		
	Sandy Fill	Mohr-Coulomb	120					1	0	35		
	Young Bay Mud (Post Earthquake)	S=f(overburden)	90					1			0.17	210

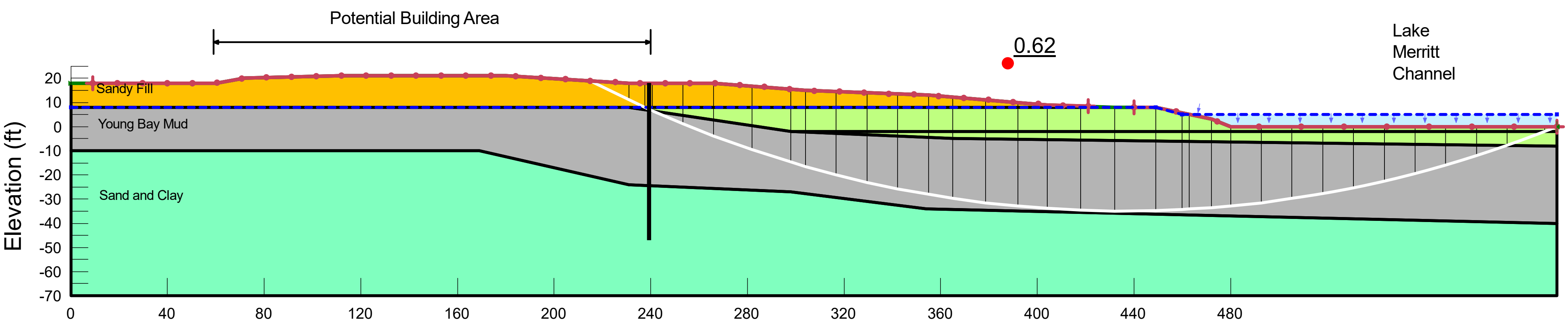


SECTION E-E'

Plate 5-24

Title: Laney College Library Learning Resource Center
File Name: Section A-06172021_RR.gsz
Description: Case 3 - Pseudo-Static k = 0.15g Non- Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280

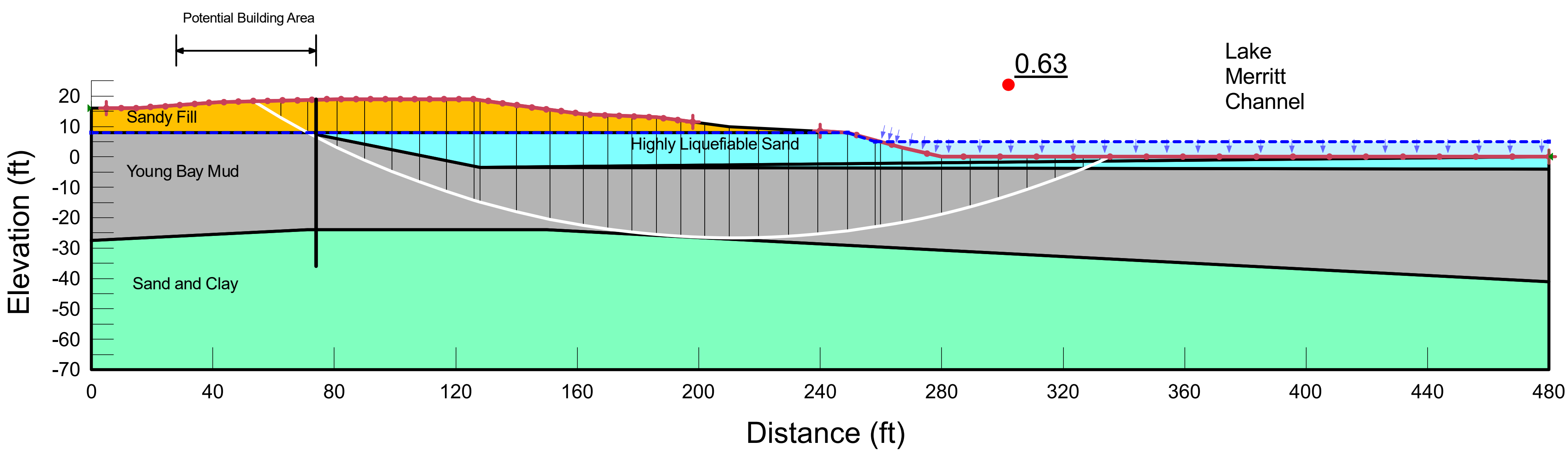


SECTION A-A'

Plate 6-1. Lateral Spreading/Slope Instability Extent

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_RR.gsz
Description: Case 3a - Pesudo-Static k = 0.15g -Non Fixed Slip Surface
Horz Seismic Coef.: 0.15
Method: Spencer

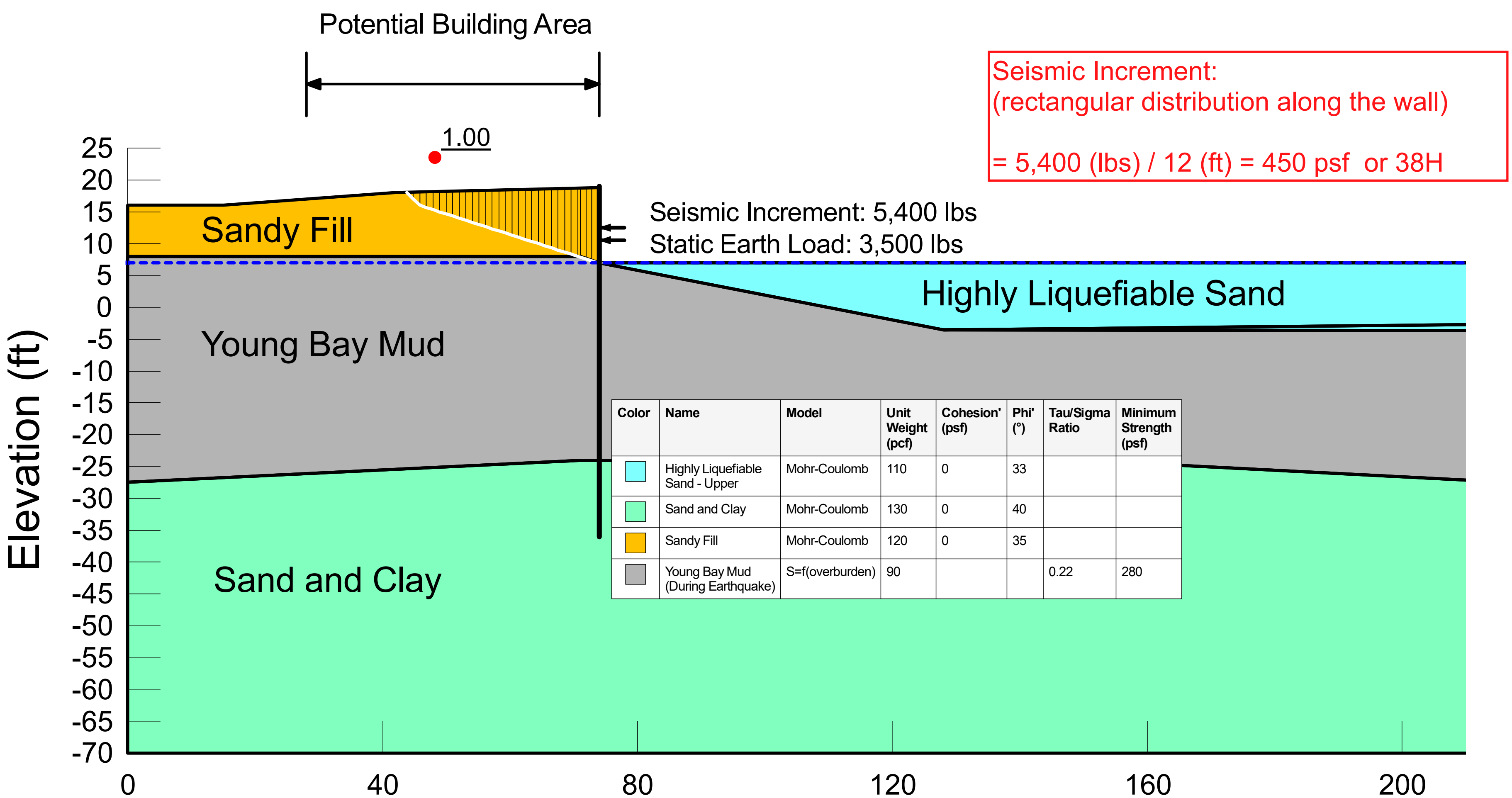
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33	1		
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			1	0.22	280



SECTION D-D'

Plate 6-2. Lateral Spreading/Slope Instability Extent

Title: Laney College Library Learning Resource Center
File Name: Section D-06172021_EarthPressure_RR.gsz
Description: Case 3a - Pesudo-Static k = 0.15g -Seismic Earth Pressure
Horz Seismic Coef.: 0.54
Method: Spencer



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (psf)
	Highly Liquefiable Sand - Upper	Mohr-Coulomb	110	0	33		
	Sand and Clay	Mohr-Coulomb	130	0	40		
	Sandy Fill	Mohr-Coulomb	120	0	35		
	Young Bay Mud (During Earthquake)	S=f(overburden)	90			0.22	280

Plate 7. Seismic Earth Pressure Calculation

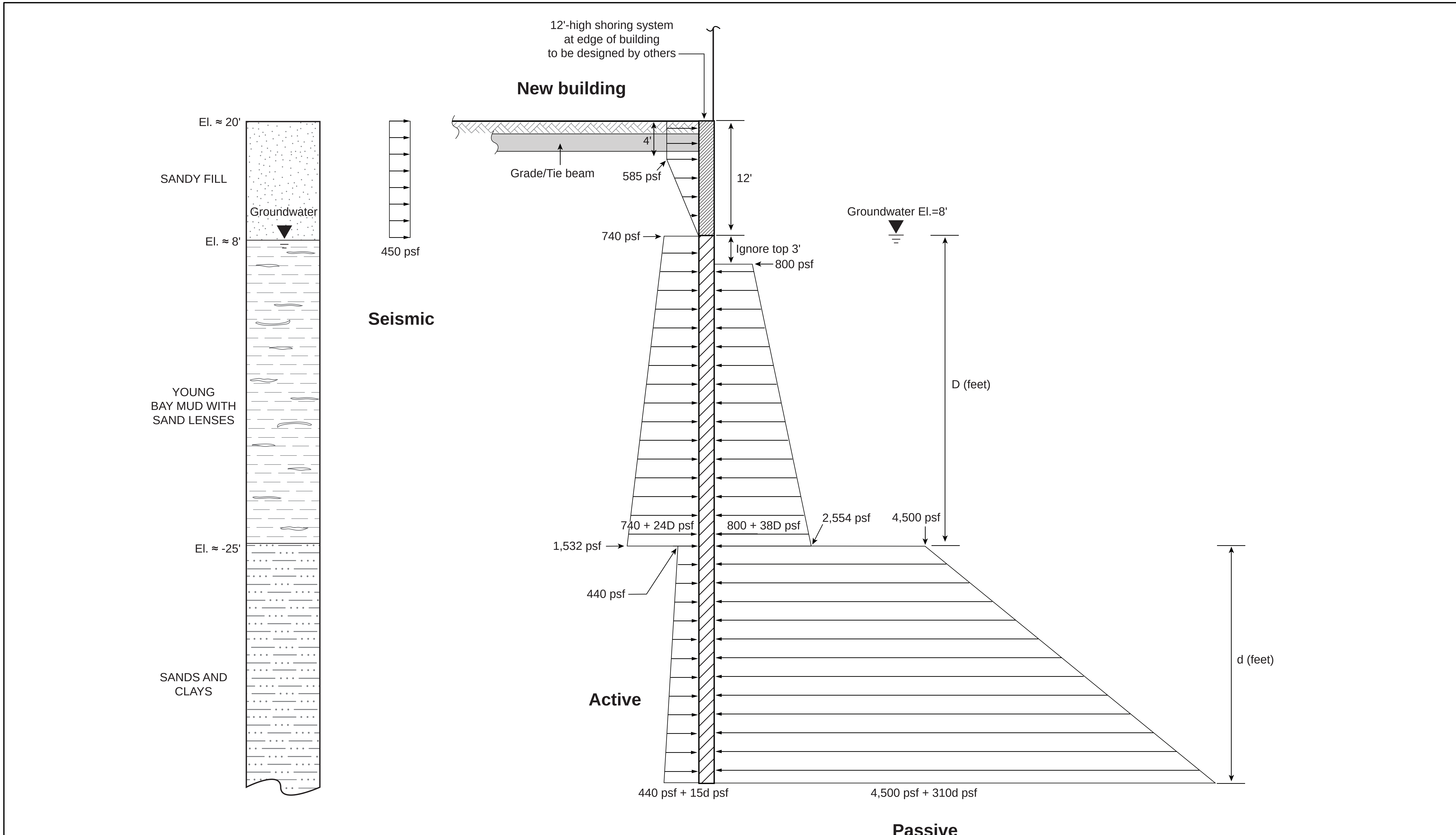


Plate 8.1: Recommended Lateral Pressures for 12-foot High Shoring System

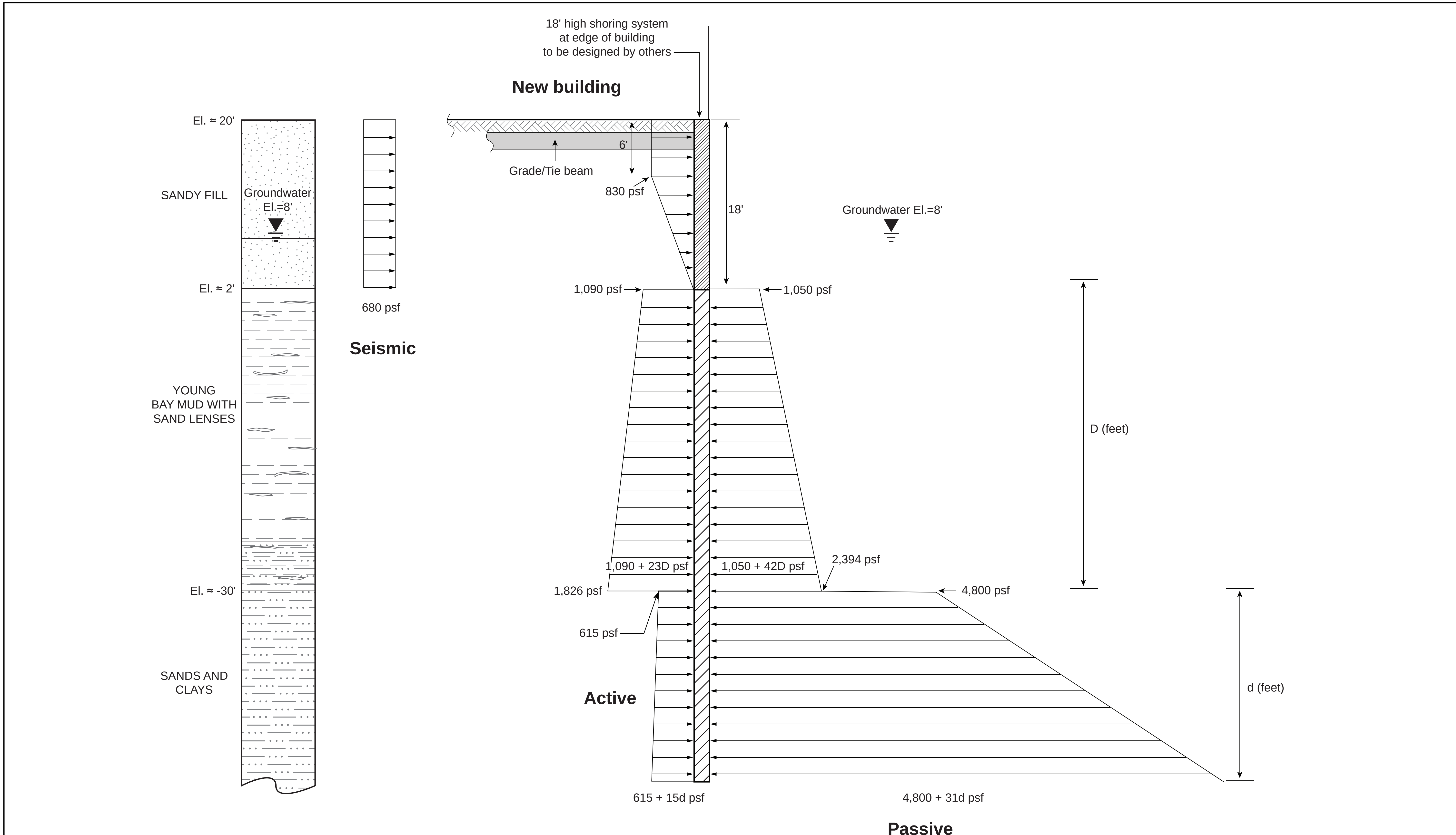


Plate 8.2: Recommended Lateral Pressures for 18-foot High Shoring System

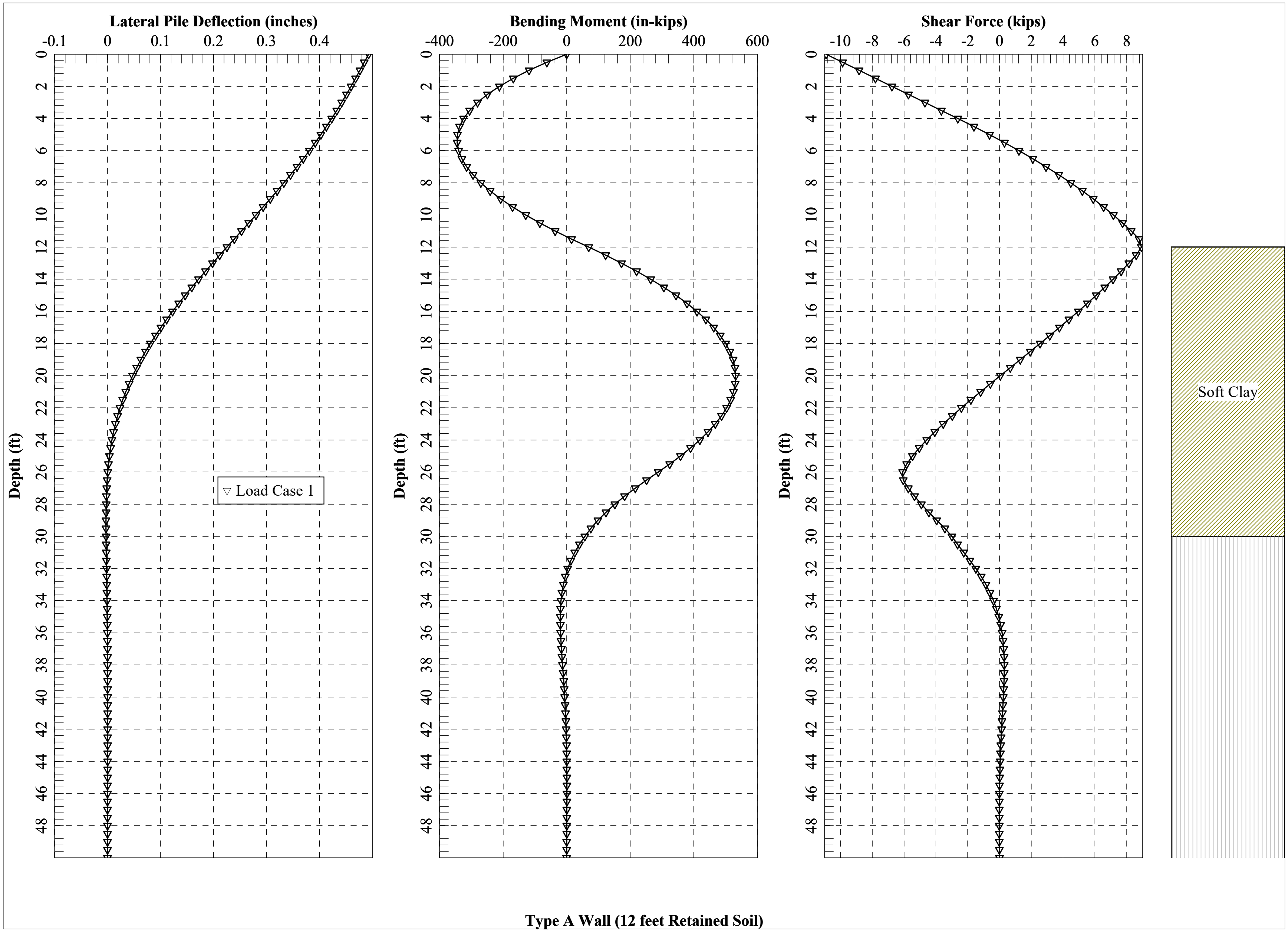


Plate 9.1: Deflection, Bending Moment, and Shear Diagrams for 12-foot High Shoring System

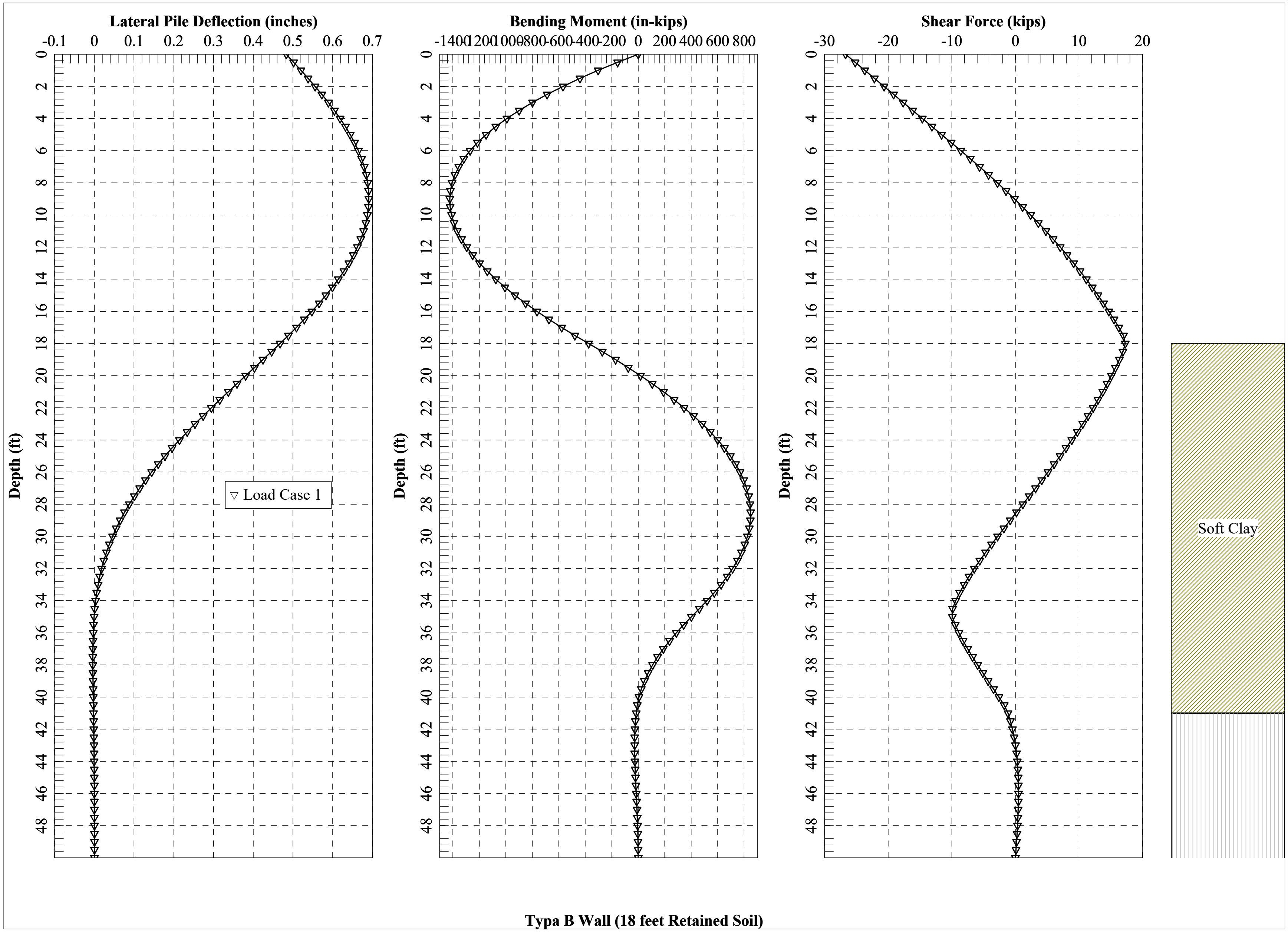


Plate 9.2: Deflection, Bending Moment, and Shear diagrams for 18-foot High Shoring System

Appendix A

CGS Review Comments

Atheria Smith
Director of Facilities Planning and Development
Peralta Community College District
Attn Dept of General Services
333 E. 8th Street
Oakland, CA 94606

December 1, 2020

Subject: **Engineering Geology and Seismology Review for**
Laney College – Laney Library & Learning Resource Center
900 Fallon Street, Oakland, CA
CGS Application No. 01-CGS4416

Dear Atheria Smith:

In accordance with your request and transmittal of documents received on May 8, 2020, the California Geological Survey (CGS) has reviewed the engineering geology and seismology aspects of the consulting report prepared for the subject project at Laney College in Oakland. It is our understanding that this project involves construction of a new three-story library building. This review was performed in accordance with Title 24, California Code of Regulations, 2019 California Building Code (CBC) and followed CGS Note 48 guidelines. We reviewed the following report:

> **Geotechnical Investigation and Geologic Hazards Evaluation, Laney College, Library Learning Resource Center, Oakland, California:** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94564 ; company Project No. 04.72190021, document No. 04.72190021-PR-001, Issue No. 02, report dated February 28, 2020, 56 pages, 21 tables, 8 appendices.

Based on our review of the data and reports presented by Fugro USA Land Inc., the consultants provide a reasonable geotechnical assessment of engineering geology and seismology issues with respect to the proposed improvements. They present a relatively thorough evaluation in which they report the potential hazards associated with fault deformation are not design concerns for the project. However, they have not addressed a few aspects of the engineering geology and seismology issues at the site, which require additional detail. Specifically, it appears additional information is needed regarding the general procedures ground motion analysis, site response analysis, liquefaction triggering and seismic settlement calculations, potential for cyclic softening of soft clay soils, and analyses and design recommendations for mitigation of potential lateral spreading and associated concerns regarding stability of the sloping ground between the proposed building and the adjacent Lake Merritt Channel. Additional information is provided in the attached Checklist Comments.

In conclusion, ***the engineering geology and seismology issues at this site are not adequately assessed in the referenced report***. It is recommended that additional information be provided as requested in the attached Note 48 Checklist Review Comments portion of this letter. The consultants are reminded that one copy of all supplemental documents should include the CGS application number, and should be uploaded directly to CGS at this link: https://www.conservation.ca.gov/cgs/upload-school. If you have any further questions about this review letter, please contact the primary reviewer at ante.mlinarevic@conservation.ca.gov.

Respectfully submitted,

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Concur:

Jennifer Thornburg
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PG 5476, CEG 2240

CERTIFIED ENGINEERING GEOLOGIST
Jennifer Thornburg
No. 2240
STATE OF CALIFORNIA

Enclosures:

Note 48 Checklist Review Comments
Keyed to: *Note 48 - Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*

Copies to:

Donald Wells, *Certified Engineering Geologist*,
Ronald Bajuniemi, *Registered Geotechnical Engineer*, and
Taiming Chen, *Registered Geotechnical Engineer*
Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94564

Chris Noll, *Architect*
Noll & Tam Architects, 729 Heinz Avenue #7, Berkeley CA 94710

Karen Van Dorn, *Senior Architect*
Division of State Architect, 1515 Clay Street, Suite 1201, Oakland, CA 94612

Note 48 Checklist Review Comments

In the numbered paragraphs below, this review is keyed to the paragraph numbers of California Geological Survey Note 48 (November, 2019 edition), *Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.*

Project Location

1. Site Location Map, Street Address, County Name: Adequately addressed.
2. Plot Plan with Exploration Data with Building Footprint: Adequately addressed.
3. Site Coordinates: Adequately addressed. Latitude and Longitude provided in report: 37.794899°N, 122.262363°W

Engineering Geology/Site Characterization

4. Regional Geology and Regional Fault Maps: Adequately addressed.
5. Geologic Map of Site: Adequately addressed.
6. Geologic Hazard Zones: Adequately addressed. The consultants report the site is not located within an Alquist-Priolo Earthquake Fault Zone. They also report the site is in a Zone of Required Investigation for liquefaction.
7. Subsurface Geology: Adequately addressed. The consultants provide a clear and detailed characterization of the complex site subsurface conditions. They report the site is underlain by an 8- to 25-foot thick layer of historical (pre-1965) artificial fill consisting of clays, sands, and gravels with concrete, brick, and wood debris overlying a layer of Holocene estuarine mud (Young Bay Mud, YBM) consisting of very soft to soft, high plasticity clays with lenses of sands. They report the thickness of the YBM varies from about 15 to 35 feet across the site and that up to 15 feet of loose to medium dense sands were encountered between the surficial fill materials and the YBM to the east and southeast of the proposed building, extending to the adjacent Lake Merritt Channel. The consultants also report that below the YBM are more competent soils consisting of lean clays and sands extending to the depths explored. They report groundwater was estimated to be as shallow as 5 feet below the ground surface based on results from CPT pore pressure dissipation tests. The consultants utilized information from one recent solid-stem auger/rotary wash boring drilled to a depth of 76-½ feet, and eight recent CPT soundings to a maximum depth of 75-½ feet. They also utilized three previous hollow-stem auger borings to a maximum depth of 75 feet, one previous continuous flight auger boring to a depth of 72 feet, and one previous CPT to a depth of 45-½ feet.
8. Geologic Cross Sections: Adequately addressed. The consultants provide five detailed geologic cross-sections of an appropriate scale.
9. Geotechnical Testing of Representative Samples: Adequately addressed.
10. Consideration of Geology in Geotechnical Engineering Recommendations: Adequately addressed. Given the presence of highly compressible YBM and liquefiable soils, the consultants recommend **the building be should supported by a deep pile foundation system with pile caps and grade beams** to resist horizontal seismic shear. They also recommend **the interior floor slab should consist of structural slabs that are designed to span between pile foundations**. The consultants further recommend a “permanent shoring system” be designed and constructed along the southeast end of the building to mitigate the impacts from potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. In order to reduce consolidation-induced

downdrag on pile foundations, the consultants recommend that **site grading should be designed so that "zero net load" will be imposed on underlying YBM soils**. Altogether, the consultants' general recommendations for foundations and site grading appear to be reasonable and appropriate for the site geologic conditions.

11. Conditional Geotechnical Topics:
    B. Deep Foundations: Adequately addressed. The consultants provide design and construction recommendations for driven or drilled piles to support the proposed building. They **recommend that piles extend to depth of at least 70 feet** to transfer bearing loads to the competent soils below the YBM. The consultants recommend values of ultimate skin friction for axial capacity and negative skin friction due to liquefaction-induced downdrag that appear to be reasonable for design of piles given the reported geotechnical site conditions. The consultants also report results of their analysis of pile lateral capacities and deflections considering four idealized soil profiles that appear to reasonably represent the variable subsurface conditions at the site. Altogether, the consultants' analyses and recommendations for design and construction of pile foundations (including indicator test program for driven piles) appear to be appropriate and reasonable for the reported site conditions, based on our understanding that effects of lateral spreading/slope instability between the building and the Lake Merritt Channel will be mitigated by a "permanent shoring system" such that there will not be a loss of capacity for piles supporting the building.

Seismology & Calculation of Earthquake Ground Motion

12. Evaluation of Historic Seismicity: Adequately addressed. The consultants provide a summary of historical seismicity in the region and historical liquefaction at the site.
13. Classify the Geologic Subgrade (Site Class): Adequately addressed. The consultants classify the site soil profile as Site Class F because of the presence of potentially liquefiable soils. CGS notes the consultants' Site Class F designation appears to be reasonable based on the data in the boring logs and CPT soundings presented.
14. General Procedure Ground Motion Analysis: **Additional information is requested.** The consultants report the site is classified as Site Class F and site response analyses are required to calculate the design ground motions at the ground surface. However, **the consultants should report the mapped S_S and S_1 parameters**.
15. Site-Specific Ground Motion Hazard Analysis: **Additional information is requested.** The consultants provide a site response analysis, as required for Site Class F.

    <u>Development of Base Target Spectrum:</u> The consultants provide a site-specific ground motion analysis to develop input ground motions for site response analyses. They use V_{s30} = 260 m/s to represent soil condition at the base of the site response model, which appears reasonable based on-site data presented.

    The consultants' site-specific ground motion analyses follow most of the provisions of ASCE 7-16, and their probabilistic MCE spectrum appear reasonable based on comparison with results from the National Seismic Hazard Model (Petersen and others, 2014). However, they do not provide adequate documentation for CGS review. Also, there are minor inconsistencies with code procedure. Specially, we request the consultants address the following issues:

    - Provide pertinent parameters for all controlling faults in deterministic analysis, including fault name, characteristic magnitude, and site-to-fault distance. Present $Z_{1.0}$ and $Z_{2.5}$ values or report if GMM default values are used.

- The discussion regarding design response spectrum for base of the YBM in Section G.4 and design response spectrum presented in Table G.4 and Figure G.4-2 are irrelevant and can lead to confusion. Only site-specific MCE_R spectrum at the base of YBM is needed for site response analysis. Site-specific design response spectrum at the surface is derived from site response analyses.
- Add spectral-dependent maximum direction scaling factors and risk coefficients to Table G.4 to facility CGS review.
- The "deterministic lower limit" appears to be outdated but does not affect the final spectra. The consultants are reminded that the "deterministic lower limit" has been superseded, see ASCE 7-16, Supplemental No.1, §21.2.2.

Time History Selection and Scaling: The consultants select five pairs of recorded acceleration time histories and scale them linearly to the approximate level of the site-specific MCE_R spectrum at base of YBM. The selection and scaling appear appropriate. However, potential near-source effects should be reflected in time history selection for a near-fault site (defined in Section 11.4.1, ASCE 7-16). The consultants are requested to modify their time history selection **to include records with velocity pulses** or present pulse periods in Table G.5 if any of their ten (10) time histories contain velocity pulses. They also are requested to present scaled input time histories (including acceleration, velocity, and displacement) and, at a minimum, one set of example time histories at the ground surface.

Site Condition Modeling and Site Response Analyses: The consultants' site response analyses, for the most part, follow the procedure in Section 21.1, ASCE 7-16. However, their documentation is insufficient for CGS review. In addition, some modeling aspects can be improved. The consultants are requested to address the following issues:

- The idealized shear wave velocity profile (Figure G.2-3) appears appropriate for YBM and the underlying sand and clay layers. However, for fill layers above the YBM, it does not reflect either the in-situ Vs measurements from CPT-07 or the correlation results from Andrus et al. (2007) and is much lower than both these datasets. The consultants are requested to justify their selection or modify Vs to be consistent with site data.
- Indicate detailed layering used in FLAC modeling for site responses for each of the three idealized stratigraphy shown in Figure G.2-6. Present and justify Vs and unit weight for each layer. The consultants are further requested to: i) justify why it is sufficient to include one single liquefiable layer in only one of the three profiles modeled; ii) clarify whether FLAC analyses is total stress or effective stress analyses, and iii) discuss the effect of liquefaction process in the liquefiable layer on surface ground motion.
- It is important for CGS to evaluate whether uncertainty in shear wave velocity is adequately incorporated in site response analyses. The consultants are requested to clearly present their upper bound and lower bound Vs profiles for each of the three idealized stratigraphic models and demonstrate that these profiles cover the uncertainty range adequately. They are further requested to present the surface spectra from all site response sensitivity cases (in terms of stratigraphy and shear wave velocity) and to demonstrate that the final recommended surface MCE_R spectrum sufficiently envelops surface spectra from all analysis cases. We note it is appropriate to use mean surface-to-base spectral ratios from all time histories as transfer function for each soil profile case. However, it is inadequate to take the mean spectrum from all soil profile cases as

the recommended surface spectrum. Instead, the recommended surface MCE_R spectrum should sufficiently envelop surface spectra from all sensitivity profile cases.

CGS notes it appears the consultants use modified F_a (1.0) and F_v (4.0) values associated with Site Class E in their input site-specific ground motion analysis for the site response analysis, which appears to be in accordance with ASCE 7, §21.3.

16. Deaggregated Seismic Source Parameters: Adequately addressed.
17. Time Histories of Earthquake Ground Motion: See item 15.

Fault Rupture Hazard Evaluation

18. Active Faulting & Coseismic Deformation Across Site: Adequately addressed. The consultants report there are no known active fault traces within, adjacent to, or trending towards the project site. They also report the closest known active fault is the Hayward Fault, located approximately 3.6 miles to the northeast. The consultants conclude no other faults are mapped or known to occur near the project and the potential for surface fault rupture at the site is considered to be very low. The data appears to support their conclusion.

Liquefaction/Seismic Settlement Analysis

19. Geologic Setting for Occurrence of Liquefaction: Adequately addressed. The consultants state several areas of historical liquefaction have been documented in the vicinity of the proposed improvements, including lateral spreading along the adjacent bank of Lake Merritt during the 1906 San Francisco earthquake. They also state the original Laney Swimming pool sustained damage from the 1989 Loma Prieta earthquake related to soil liquefaction and was subsequently replaced. The consultants report there are saturated, loose to medium dense sand layers of various thicknesses located both above and within the YBM layer that have a high potential for liquefying when they are subjected to an MCE earthquake event. The data presented appear to support this conclusion.
20. Seismic Settlement Calculations: **Additional information is requested.** The consultants report they used a PGA_M of 0.810*g* and set groundwater at an elevation of +8 feet in their liquefaction analysis of the site, which appear to be reasonable. However, the consultants **should utilize the maximum considered earthquake magnitude associated with the Hayward Fault** in accordance with 2019 CBC §1803A.5.12, Subsection 2 in their analysis of liquefaction and seismic settlement for the proposed improvements.
21. Other Liquefaction Effects: **Additional information is requested**. The consultants report the loose to medium dense, near-surface sand layers encountered by CPTs and borings in the area adjacent to the Lake Merritt Channel are liquefiable and have a **high potential to undergo lateral spreading** during soil liquefaction resulting from an MCE event. This general conclusion appears reasonable based on the information provided. The consultants have estimated the areal extent and potential horizontal displacements from lateral spreading using the same input parameters of earthquake magnitude and peak ground acceleration as for their analysis of liquefaction triggering. However, as noted above in Item 20, the consultants **should utilize the maximum considered earthquake magnitude associated with the Hayward Fault in their analysis of liquefaction and associated effects, including lateral spreading**. The consultants are therefore requested to update their analysis of potential lateral spreading and to review and revise their conclusions regarding the likely areal extent and estimated horizontal displacements of lateral

spreading at the site, if warranted. The consultants are further requested to review their recommendations for mitigation of lateral spreading and associated instability of the gently sloping ground adjacent to the Lake Merritt Channel, if warranted, based on the results of their updated analysis.

22. Mitigation Options for Liquefaction/Seismic Settlement: **Additional information is requested.** The consultants reasonably recommend the building be supported by pile foundations bearing in competent soils at depths well below liquefiable soils and designed to resist downdrag due to liquefaction-induced settlement. They also recommend using flexible connections for pipes/utilities to mitigate potential damage resulting from liquefaction-induced settlement of soils outside the building relative to the pile-supported structure.

    Based on their assessment of the limits and estimated horizontal displacements due to lateral spreading, the consultants recommend the southeast side of the new building foundation should include a "permanent shoring system" to mitigate the detrimental effects from the potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. While this general recommendation appears, reasonable, **the consultants are requested to review the results of updated lateral spreading analysis requested in Item 21 and to update their mitigation recommendations if warranted**. Further discussion and request for additional information regarding the recommended "permanent shoring system" is provided in Item 28 of this checklist.

Slope Stability Analysis

23. Geologic Setting for Occurrence of Landslides: Adequately addressed. The consultants report global site slope stability was evaluated using a two-dimensional, limit equilibrium computer program; however, they clarify the seismic slope stability of the site is most likely governed by the extent of possible ground lateral spreading during major liquefaction events.
24. Determination of Static and Dynamic Strength Parameters: **Additional information is requested.** The consultants report soil engineering properties were developed based on the field exploration and laboratory testing results by Fugro and others, and typical engineering correlations. They provide unit weights and static and seismic shear strength parameters for the sandy fill, highly liquefiable sand, and sand and clay soil layers which appear to be reasonable based on the data provided. The consultants also provide soil properties for YBM, which include a total unit weight, a minimum undrained shear strength, and an undrained shear strength ratio used in both static and seismic analyses. However, **the undrained shear strength ratio appears to be high** based on review of the reported laboratory test data (TXUU) and does not appear to consider the potential for cyclic softening and loss of strength during seismic loading (refer also to Item 31I). The consultants should provide further discussion and justification for the higher than anticipated undrained shear strength ratio for the YBM and consider **potential for reduced strength in YBM due to cyclic softening**.
25. Determination of Pseudo-Static Coefficient (Keq): **Additional information is requested.** The consultants report a pseudo-static coefficient (k) of 0.15*g*. They report calculating slope displacement based on maximum horizontal acceleration of 0.81g from a modal magnitude 7.5 causative earthquake located 6.8 km from the site, which appears to be reasonable. However, they should **provide supporting calculations utilizing these input values for our review**.

26. Identify Critical Slip Surfaces for Static and Dynamic Analyses: **Additional information is requested.** The consultants present analyses of slope failure for three cross sections (A-A', D-D', and E-E') extending from/through the proposed library building to the adjacent Lake Merritt Channel. They analyze four cases for each section: long-term (static), pseudo-static (yield acceleration), pseudo-static (fixed surface and pseudo-static coefficient), and post liquefaction (static) scenarios. The consultants report by fixing the slip surface daylight location at the edge of the proposed building location, factors of safety against slope failures for the pseudo-static analyses are below 1 for each of the three sections. They conclude the analyses fail to meet the commonly accepted minimum factor of safety value of 1.15 for seismic performance and that seismic slope stability of the site is most likely governed by the extent of possible lateral spreading. While CGS considers these general conclusions reasonable, we **request the consultants discuss and consider if non-circular failure surfaces would better represent potential lateral spreading masses and may result in lower factors-of-safety and/or greater potential hazard to the proposed building than the analyzed circular surfaces**. CGS also requests the consultants consider revised strength parameters for the YBM soils in their additional analyses of circular and tabular or wedge failure surfaces that may better reflect likely deformation mechanisms.
27. Dynamic Site Conditions: **Additional information is requested.** CGS notes that seismic slope stability should consider site response analysis in determining the psuedo-static coefficient (see Items 15 and 25).
28. Mitigation Options/Other Slope Failure: **Additional information is requested.** As noted previously (refer to Items 10, 11B, and 22), the consultants recommend the southeast side of the building foundation should include a "permanent shoring system" to mitigate the detrimental effects from the potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. They further recommend that the shoring system be designed to retain from 12 to 18 feet of soils, and they provide recommended lateral pressures for the shoring system design on a pair of Plates that indicate the system is "to be designed by others." Given our requests for the consultants to provide additional information and updated analyses of lateral spreading and slope stability that inform the consultants' recommendations for the shoring system, CGS requests the consultants **address the potential for greater areal extent/depth of lateral spreading/slope instability to be mitigated by the shoring system.** CGS further requests the consultants **provide discussion and justification for the lateral pressures recommended for design and address the potential for load transfer/surcharge from the building grade beams and/or pile foundations to the shoring system that should be incorporated in the design of the system.** When available, the design and plans for the shoring system should also be provided to CGS for review.

Other Geologic Hazards or Adverse Site Conditions

29. Expansive Soils: Adequately addressed. The consultants report the near-surface soils encountered at the site were predominately man-made fills that consist of silty sands and lean clays. They conclude the expansion potential of the near-surface soils at this site is considered **low to moderate**, which appears to be reasonable based on the laboratory test results presented.

30. Corrosive/Reactive Geochemistry of the Geologic Subgrade: Adequately addressed. The consultants report the onsite near-surface soils should be considered as "moderately" corrosive based on resistivity measurements, which appears the be reasonable based on the laboratory test results presented.
31. Conditional Geologic Assessment: Selected geologic hazards addressed by the consultant are listed below:
    C. Flooding: Adequately addressed. The consultants report that based on the FEMA Flood Insurance Rate Map, the project building area is located outside a 100-year flood zone. They also report the site is not located within any dam failure inundation areas and conclude the potential for flooding or inundation of the project site is considered to be very low. The data presented appears to be reasonable.
    D. Tsunami and Seiche Inundation: Adequately addressed. The consultants report the project building area is located adjacent to but outside the mapped CGS Inundation Map boundary, and CGS notes the project building area also appears to be outside the ASCE tsunami design zone. They conclude the potential inundation by a tsunami at the project building area is low. This conclusion appears to be reasonable based on the data presented.
    I. Clays and cyclic softening: **Additional information is requested**. **The consultants should evaluate and provide explicit discussion of the potential for cyclic softening and loss of strength in the very soft to soft and weak YBM soils at the site due to seismic shaking**. If the consultants determine that significant potential for cyclic softening and loss of strength exists at the site, the associated hazards of lateral spreading/instability of slopes and impact upon the project design and the proposed improvements should be further addressed as noted previously in Items 11A, 24, 26, and 28.

Report Documentation

32. Geology, Seismology, and Geotechnical References: Adequately addressed.
33. Certified Engineering Geologist: Adequately addressed.
    Donald Wells, Certified Engineering Geologist #2120
34. Registered Geotechnical Engineer: Adequately addressed.
    Ronald Bajuniemi, Registered Geotechnical Engineer #112
    Taiming Chen, Registered Geotechnical Engineer #2924

FUGRO
Fugro USA Land, Inc.
1777 Botelho Drive, Suite 262
Walnut Creek, California 94596
T +1 925 949-7100

Peralta Community College District
900 Fallon Street
Oakland, California 94607

April 4, 2022

Dear Mr. Krill,

Fugro is pleased to provide the following responses to the California Geological Survey (CGS) second round of review comments on our Geotechnical Investigation and Geologic Hazard Evaluation report for the Laney College Library Learning Resource Center, Oakland, California, dated February 28, 2020.

The CGS review comments letter is titled "Second Engineering Geology and Seismology Review for Laney College & Learning Resource Center, 900 Fallon Street, Oakland, CA, CGS Application No. 01-CGS4416", dated January 31, 2022.

Based on CGS comments, reasonable geotechnical assessment of engineering geology and seismology issues for the project was provided by Fugro. However, CGS has requested more details and clarifications on a few aspects of the seismology and lateral spreading issues at the site. Below, we have provided some clarifications on those comments of concern. A copy of CGS review letter is presented in Appendix A.

Response to CGS Review Comments

The review comments by CGS are indicated below in italics, which are followed by Fugro's responses and clarifications.

Development of Base Target Spectrum

> ***CGS**: The consultants insist that the design-level spectrum at the base of the YBM is needed. We note this is a misunderstanding and would refer the consultants to ASCE 7-16, Section 21.1.1 for development of ground motions at the base of a soil model. We reiterate that only the MCER spectrum is needed at the base of a soil model for site response analysis. The check of the site-specific MCER spectrum against the 150 percent of the site-specific design response spectrum is referring to the final MCER spectrum at the ground surface, not at the base of the soil model.*
>
> *We appreciate the new Table 1 provided in Report 4 summarizing pertinent fault information for deterministic ground motion hazard analysis. Because deterministic spectrum controls the final MCER*

spectrum, we are requesting more information on deterministic ground motion results to facilitate our review. Specifically, ***the consultants are requested to discuss which fault source (or sources) controls the 84th percentile deterministic spectrum tabulated in Table 2 of Report 4 (revised Table G.4 of Report 1)****. provide a figure showing deterministic spectra of individual fault sources along with the final spectrum that envelopes spectra of individual sources if multiple faults contribute to the final deterministic spectrum.*

Fugro: The development of the design response spectrum at the base of the YBM will be removed from the revised report. Please note that this change doesn't affect the calculated MCE_R response spectrum at the base of YBM and used as input to the site response analyses.

The 84th percentile deterministic spectrum is controlled by the Hayward fault for all spectral periods.

Time History Selection and Scaling

CGS: *We appreciate the seed and surface time histories presented in the new Plates 1 and 2 provided in Report 4. However, we do not see velocity time histories at the surface (from site response analysis) and would like them presented in the consultants' final report. We would also like to see the two components of time histories be plotted separately to facilitate visual inspection.*

Fugro: The velocity time histories at the surface will be presented in final report.

CGS: *The surface acceleration time histories for RSN1176 from site response analysis have some peculiar characteristics. There are apparent high frequency wavelets riding on top of the long period waves, particularly for the first few cycles.* ***The consultants are requested to ensure that these somewhat unusual features are not artifacts from site response analysis.***

Fugro: These features are related to: (1) characteristics of the RSN1176 seed motion (Plate 1.5 of Report 4) and (2) lowpass filtering of simulated surface acceleration time histories to remove high frequencies that cannot be propagated by the discretised 1D soil column. The RSN1176 seed motion contains displacement and velocity pulses and exhibits significant long period accelerations in approximately the first 15 seconds of the recording. These characteristics are reflected in the first few long period cycles of the surface response. Additionally, shorter period accelerations occurring simultaneously appear more regular in the acceleration times histories presented in Plate 2.2 of Report 4 because the discretised model cannot physically propagate frequencies greater than about 30 Hz.

Site Condition Modelling and Site Response Analyses:

We appreciate the additional discussion and information provided by the consultants in response to our request on site condition modeling. Much of the discussion refers to the original report. However, it is still not clear how the idealized Vs profiles shown, for example, in Figure G.2-3 of Report 1 for the YBM, is mapped to each of the three stratigraphic profiles for site response analysis. ***The consultants are***

requested to plot out the depth-dependent shear wave velocity for each of the three idealized stratigraphic profiles *shown in Figure G.2-6 of Report 1 (for the entire depth range and can be plotted next to each stratigraphic profile). It would also be helpful to add unit weights for each layer in these profiles. Plots of depth-dependent variation of shear wave velocity and unit weight used for each of the three stratigraphic profiles help us further evaluate whether site conditions and uncertainties in soil properties are adequately captured in site response analysis.*

Fugro: Plots of shear wave velocity and unit weight profiles used in the baseline and parametric analyses are presented on Figure 1 for each of the three stratigraphic profiles.

We are having trouble picturing what 2.5-foot square zones look like for discretization of a one-dimensional model in Fugro's site response analysis using FLAC. A simple figure might help.

Fugro: Figure 2 presents the one-dimensional model used in the FLAC site response analyses for soil profile 1.

Discussion of Liquefaction and Seismic Settlement:

Discussion of Lateral Spreading and Slope Stability Concerns:

Discussion of Mitigation for Lateral Spreading

Closing

The conclusions and recommendations provided in this letter are meant to respond to the CGS's January 31, 2022, comments. Our conclusions and recommendations are solely professional opinions and were made in accordance with generally accepted local and current geotechnical engineering principles and practices. We make no warranty, either express or implied. Should you have any questions or require additional information, please contact us.

Sincerely,

Reza Rahimnejad, PhD
Project Engineer

Ronald L. Bajuniemi, PE, GE
Principal Consultant

Document Information

Project Title	Laney College Library Learning Resource Center
Document Title	Response to CGS Review Comments (CGS Application No. 01-CGS4416)
Fugro Project No.	04.72190021
Fugro Document No.	04.72190001-PR-003
Issue Number	01
Issue Status	Draft

Revision History

Issue	Date	Status	Comments on Content	Prepared By	Checked By	Approved By
01	April 5, 2022	Draft	For review	RR/AP/AF		

Project Team

Initials	Name	Role
RR	Reza Rahimnejad, RR	Project Manager
RLB	Ronald L. Bajuniemi, PE, GE	Project Principal
AP	Adam Price	Project Engineer
AF	Alfredo Fernandez	Principal Engineer
JU	Jose Ugalde	Principal Engineer

List of Figures

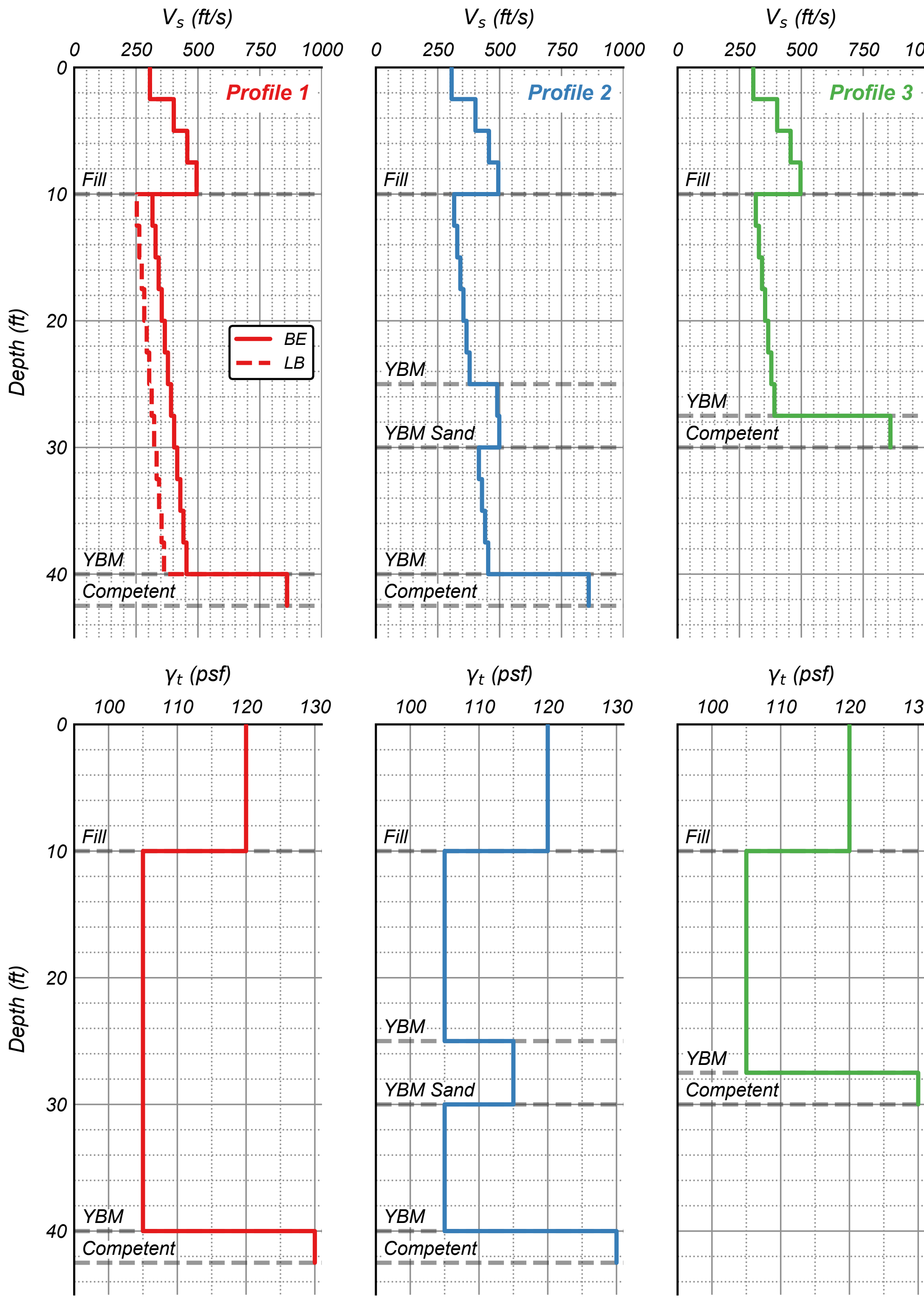

Figure 1: Shear Wave Velocity and Unit Weight Profiles used in the Site Response Analyses

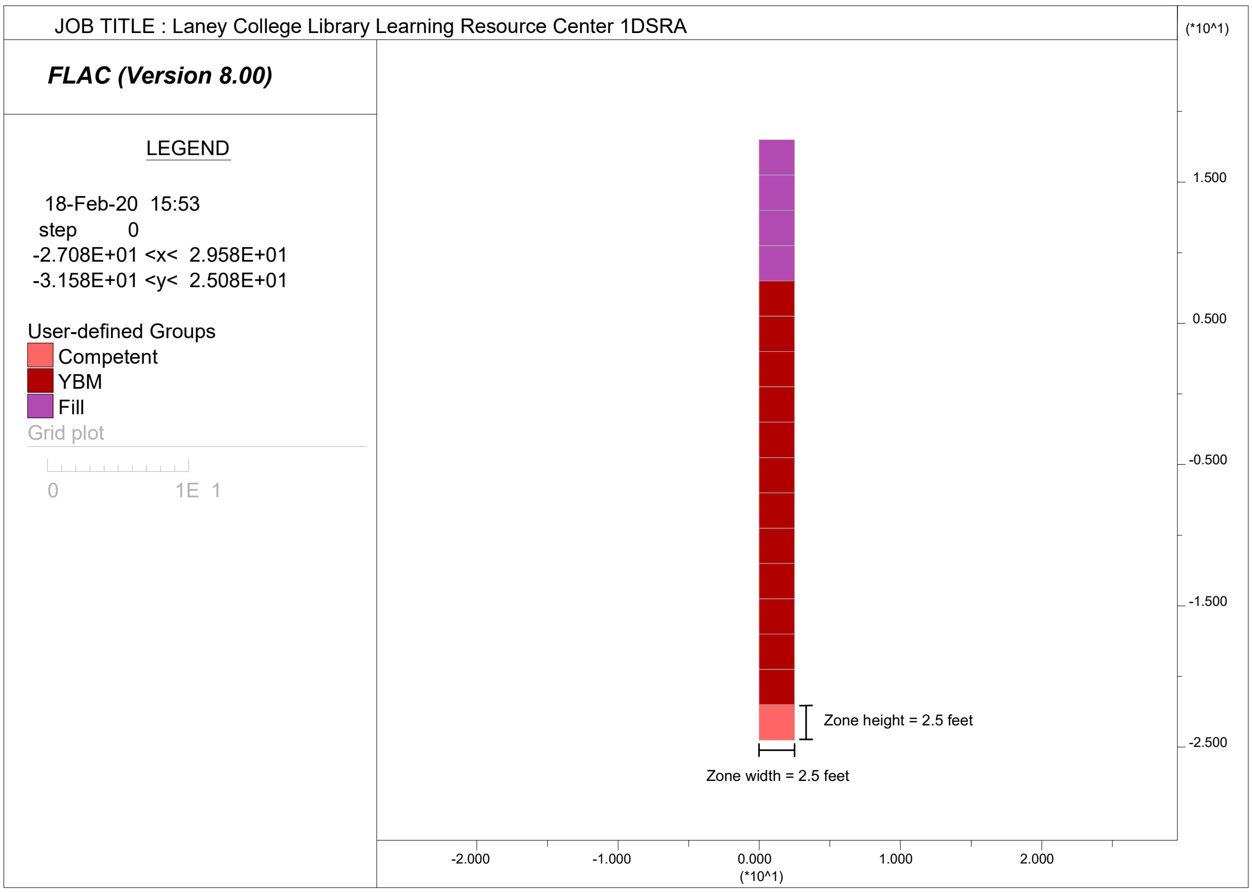

Figure 2: FLAC Model used in the Site Response Analyses for Profile 1

Appendix A

CGS Review Comments

Atheria Smith
Director of Facilities Planning and Development
Peralta Community College District
Attn Dept of General Services
333 E. 8th Street
Oakland, CA 94606

January 31, 2022

Subject: **Second Engineering Geology and Seismology Review for Laney College – Laney Library & Learning Resource Center 900 Fallon Street, Oakland, CA CGS Application No. 01-CGS4416**

Dear Atheria Smith:

In accordance with your request and transmittal of documents received on May 8, 2020 and January 7, 2022, the California Geological Survey (CGS) has reviewed the engineering geology and seismology aspects of the consulting reports and associated design documents prepared for the subject project at Laney College in Oakland. It is our understanding that this project involves construction of a new three-story Library & Learning Resource Center (LLRC) building. This second review was performed in accordance with Title 24, California Code of Regulations, 2019 California Building Code (CBC) and followed CGS Note 48 guidelines. We reviewed the following documents for this second review of the project:

(2) **Peralta Community College District, Laney Library & Learning Resource Center (Building 100 Replacement):** Noll & Tam Architects, 729 Heinz Avenue #7, Berkeley CA 94710; company project No. 21942, issue for permit structural/foundation plans dated November 2, 2020, 16 sheets.

(3) **Laney College Learning Resource Center Foundation Calculations:** Thornton Tomasetti, 301 Howard Street, Suite 1030, San Francisco, CA 94105, company project No. U20005.00, structural design and calculation package dated December 14, 2021, 92 pages.

(4) **Response to CGS Review Comments (CGS Application No. 01-CGS4416):** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94596; company Project No. 04.72190021, document No. 04.72190021-PR-002, Issue No. 02, report dated December 22, 2021, 18 pages, 6 tables, 3 figures, 40 plates,1 appendix.

(5) **Geotechnical Engineering Civil Plans Review for Laney College Learning & Library Resource Center Project, Oakland, California:** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94596; company document No. 04.72190021-L-001, Issue No. 01, letter report dated January 7, 2022, 2 pages.

(6) **Peralta Community College District, Laney Library & Learning Resource Center (Building 100 Replacement):** Noll & Tam Architects, 729 Heinz Avenue #7, Berkeley CA 94710; company project No. 21942, issue for bid civil design plans dated January 13, 2022, 12 sheets.

In addition, we previously reviewed the following report:

(1) **Geotechnical Investigation and Geologic Hazards Evaluation, Laney College, Library Learning Resource Center, Oakland, California:** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94596 ; company Project No. 04.72190021, document No. 04.72190021-PR-001, Issue No. 02, report dated February 28, 2020, 56 pages, 21 tables, 8 appendices.

CGS previously documented our review and submitted our findings regarding this project in a letter dated December 1, 2020, in which additional information was requested regarding the general procedures ground motion analysis, site response analysis, liquefaction triggering and seismic settlement calculations, lateral spreading, and the potential for cyclic softening of soft clay soils. In addition, the consultants were requested to provide analyses and design recommendations for mitigation of potential lateral spreading and associated concerns regarding stability of the sloping ground between the proposed building and the adjacent Lake Merritt Channel.

Discussion of Earthquake Ground Motions

Per the request from our review letter dated December 1, 2020, in Report 4 the consultants report the map values of $S_S = 1.74g$ and $S_1 = 0.66g$, which appear reasonable. They provide a new table with fault names, characteristic magnitudes, and site-to-fault distances, which appear appropriate. They clarify that the Ground Motion Model (GMM) default $Z_{1.0}$ and $Z_{2.5}$ values were used in ground motion hazard analysis and add spectral-dependent maximum direction scaling factors and risk coefficients. The consultants also provide additional information and discussions in Report 4 regarding design spectrum at the base of the Young Bay Mud (YBM), velocity pulses in selected time histories, and site condition modeling. We found this additional information and discussion helpful, but inadequate in some cases as discussed in the following paragraphs.

<u>Development of Base Target Spectrum:</u>

The consultants insist that the design-level spectrum at the base of the YBM is needed. We note this is a misunderstanding and would refer the consultants to ASCE 7-16, Section 21.1.1 for development of ground motions at the base of a soil model. We reiterate that only the MCE_R spectrum is needed at the base of a soil model for site response analysis. The check of the site-specific MCE_R spectrum against the 150 percent of the site-specific design response spectrum is referring to the final MCE_R spectrum at the ground surface, not at the base of the soil model.

We appreciate the new Table 1 provided in Report 4 summarizing pertinent fault information for deterministic ground motion hazard analysis. Because deterministic spectrum controls the final MCE_R spectrum, we are requesting more information on deterministic ground motion results to facilitate our review. Specifically, **the consultants are requested to discuss which fault source (or sources) controls the 84th percentile deterministic spectrum tabulated in Table 2 of Report 4 (revised Table G.4 of Report 1)**. It would also be helpful for the consultants to provide a figure showing deterministic spectra of individual fault sources along with the final spectrum that envelopes spectra of individual sources if multiple faults contribute to the final deterministic spectrum.

Time History Selection and Scaling:

We appreciate and agree with the consultants' assessment that velocity pulses would have negligible effect given that the expected pulse period is much longer than the natural periods of the proposed structure and of the site.

We appreciate the seed and surface time histories presented in the new Plates 1 and 2 provided in Report 4. However, we do not see velocity time histories at the surface (from site response analysis) and would like them presented in the consultants' final report. We would also like to see the two components of time histories be plotted separately to facilitate visual inspection.

The surface acceleration time histories for RSN1176 from site response analysis have some peculiar characteristics. There are apparent high frequency wavelets riding on top of the long period waves, particularly for the first few cycles. **The consultants are requested to ensure that these somewhat unusual features are not artifacts from site response analysis**.

Site Condition Modeling and Site Response Analyses:

We appreciate the additional discussion and information provided by the consultants in response to our request on site condition modeling. Much of the discussion refers to the original report. However, it is still not clear how the idealized Vs profiles shown, for example, in Figure G.2-3 of Report 1 for the YBM, is mapped to each of the three stratigraphic profiles for site response analysis. **The consultants are requested to plot out the depth-dependent shear wave velocity for each of the three idealized stratigraphic profiles** shown in Figure G.2-6 of Report 1 (for the entire depth range and can be plotted next to each stratigraphic profile). It would also be helpful to add unit weights for each layer in these profiles. Plots of depth-dependent variation of shear wave velocity and unit weight used for each of the three stratigraphic profiles help us further evaluate whether site conditions and uncertainties in soil properties are adequately captured in site response analysis.

We are having trouble picturing what 2.5-foot square zones look like for discretization of a one-dimensional model in Fugro's site response analysis using FLAC. A simple figure might help.

Developing site-specific ground motions from site-response analysis is probably a deliberate decision of the project team. Nevertheless, we note that because building period is reported to be less than 0.5 s, site response analysis is not required (see Exception in ASCE 7-16, Section 20.3.1(1)), although site response analysis can be used for any structure as stated in ASCE 7-16, Section 11.4.8.

Discussion of Liquefaction and Seismic Settlement

In Report 1, the consultants reported they used a PGA_M of 0.810*g,* a deaggregated mean modal earthquake magnitude of 7.0, and set groundwater at an elevation of +8 feet in their liquefaction analysis of the site. CGS was concerned that the consultants' use of the mean modal magnitude is significantly lower than the maximum considered earthquake (MCE) magnitude associated with the Hayward Fault. We are concerned by the potential for additional settlement and greater depth/extent of liquefiable soils that should be considered in the estimation of potential lateral spreading extent and displacement. The consultants responded to our comment by reporting they performed sensitivity analysis by changing the earthquake Magnitude to 7.6. They state the sensitivity analyses show the estimated settlements are not sensitive to earthquake Magnitude; however, **the consultants did not provide the sensitivity analysis for our review**. The consultants should provide all data and calculations for the revised liquefaction and seismic settlement analyses.

Discussion of Lateral Spreading and Slope Stability Concerns

As discussed in our first review letter, in Report 1 the consultants had reported a high potential for lateral spreading within the loose to medium dense near surface liquefiable layers in the area adjacent to the Lake Merritt channel and east of the proposed LLRC building. They estimated the areal extent and potential horizontal displacements from lateral spreading based on results of empirical methods for analysis of SPT and CPT data. However, in doing so they had not utilized the MCE magnitude associated with the Hayward fault. Additionally, the undrained shear strength ratio associated with the Young Bay Mud (YBM) soils was high compared to reported lab test results, and evaluation of potential for reduced strength in YBM due to cyclic softening was not provided in the consultants' slope stability analysis models of potential lateral spreading masses. We also requested the consultants to provide calculations justifying the reported pseudo-static coefficient (K_{eq}) input to their seismic stability analyses. Furthermore, we requested the consultants to revise their slope stability models and analyses of lateral spreading masses as necessary based on their responses to these issues.

In Report 4, the consultants provide the results of their updated empirical based analyses of potential lateral spreading displacements utilizing SPT and CPT data from multiple explorations and reasonable input values for distance to fault, MCE magnitude, and PGA_M. They report estimated horizontal displacements varying from 2.8 to 5.2 feet at the locations of the analyzed explorations at the site. CGS observes the consultants report estimated lateral spreading displacements of about 3 to 5 feet for the analyzed soil profiles from explorations located within the reported footprint of the LLRC building. They also report a potential maximum lateral displacement of 7.5 feet on the slope to the east. CGS notes that these results appear to be reasonable based on the information provided, and the consultants appear to have adequately identified that **a significant lateral spreading hazard exists in the area of the Laney College campus adjacent to the Lake Merritt channel that includes the proposed LLRC building site**. Consequently, the consultants reiterate their recommendation that **permanent shoring should be installed at the east side of the LLRC building to protect it from effects of lateral spreading**. The consultants' additional analysis of potential extents and displacements of lateral spreading/slope failure masses and their recommendations for design of the permanent shoring are discussed further in the following sections of this letter.

In our first review letter, we requested the consultants address several concerns and provide additional information regarding their slope stability modeling and analysis of potential lateral spreading masses, including determination of shear strength and potential for cyclic softening/strength loss for the YBM soils, justification of the pseudo-static coefficient used in the seismic stability models, and consideration of non-circular failure surfaces in the analyses. In Report 4, the consultants plot interpreted undrained shear strength values for YBM soils based on CPT data and mention that the laboratory triaxial strength (TXUU) test results fall at the lower end range of the values interpreted from CPT data; they associate that difference with likely sample disturbance. They further report that a **revised average strength ratio was selected for the YBM soils** using a conservative parameter for analysis of the CPT data. The revised shear strength of YBM soils reported by the consultants appears reasonable based on the information provided. The consultants also **provide strength reduction factors associated with cyclic softening of YBM soils** for dynamic loading, and with post-earthquake/post-cyclic softening conditions for static loading. These factors appear to be reasonable based on the information provided, and appear to have been determined in accordance with the methodology of the cited references.

In Report 4, the consultants report they determined the seismic coefficient to be input to pseudo-static slope stability analyses considering design-level PGA, which is reasonable and appropriate. CGS also observes they input reasonable earthquake magnitude and source distance values. However, **the basis of the displacement of 76.2 centimeters (30 inches) utilized in the calculation of the seismic coefficient is not clear**. The consultants are requested to provide further discussion and justification for consideration of the relatively large displacement in their determination of the seismic coefficient used for the pseudo-static stability analyses.

In Report 4, the consultants also provide revised slope stability analyses utilizing the updated YBM soil strength parameters and that consider both circular and non-circular failure surfaces. They present results of analyses that appear to depict the extent of critical failure surfaces in relation to the eastern edge of the LLRC building based on the K_{eq} value considered for a displacement of 30 inches. They also report estimated displacements of the slope between the LLRC building and the Merritt Lake channel resulting from lateral spreading and loss of strength in YBM soils that may occur during an MCE event. However, CGS notes **the extent/geometry of the reported critical failure surfaces may be underpredicted with respect to encroachment upon the LLRC building footprint given the failure surface geometry is dependent on the input seismic coefficien**t. Additionally, the reported displacements of potential sliding masses appear to be underestimated when considering the reported yield accelerations. CGS therefore requests the consultants to provide additional discussion and to review and update their analyses of potential lateral spreading/slope failures, as warranted, considering an appropriate displacement for calculation of the seismic coefficient relative to the structural tolerances of the LLRC building foundations and the proposed shoring wall.

Discussion of Mitigation for Lateral Spreading

In Reports 1 and 4, the consultants recommend that a permanent shoring wall should be constructed along the east and southeast side of the LLRC building to mitigate the effects of lateral spreading upon the LLRC building. They provide recommended values of lateral earth pressures for design of the shoring system considering their evaluation and analyses of potential extent and depth of lateral spreading/slope failures of the soils between the LLRC building and the Merritt Lake Channel. As noted in our first review, CGS requested additional information from the consultants regarding their lateral spreading and slope stability analyses that inform their recommendations for design of the retaining/shoring wall, and we requested the consultants consider the interaction of the LLRC building piles/pile caps and shoring wall. We also requested the retaining/shoring wall design calculations and drawings be provided for our review.

Based on our review of Report 4, CGS understands **the shoring system is planned to be constructed as a tangent pile wall comprised of a single row of adjacent 24-inch diameter**

cast-in-place (CIP) piles that is structurally connected to the pile caps of the LLRC building in order to limit the deflection of the top of the wall. Regarding the design of the planned tangent pile wall, in Report 4 the consultants mention the active, passive and seismic earth pressures recommended for use in design of the wall are developed based on the Caltrans trenching and shoring Manual and the AASHTO A11.3.3 General limit equilibrium method. Although the consultants appear to have followed the cited design procedures to calculate the applicable seismic earth pressure and active pressure distribution along the wall, it is not clear whether the recommended earth pressures on Plates 8.1 and 8.2 are intended to represent restrained (utilizing the at-rest coefficient of 0.6) or unrestrained (utilizing the at-rest coefficient of 0.4) conditions. The consultants are requested to **clarify which conditions, at-rest or active, the recommended earth pressures on Plates 8.1 and 8.2 of Report 4 are associated with, and to report what level of lateral deflection the tangent pile wall can be designed for using the recommended earth pressures**. Additionally, the lateral earth pressures recommended by the consultants for design of the retaining/shoring wall appear to be incorrect and inconsistent with consultants' interpreted soil profile as provided in Table 4 of Report 4 and the soil profile input for lateral spreading/slope stability analyses. **CGS requests the consultants to provide justification and detailed calculations that clearly show the utilized earth pressure coefficients and values of soil cohesion and unit weights used to develop the earth pressure diagram recommended for design of the tangent pile wall.** These earth pressures are critical for design of the tangent pile wall and establishing its required embedment depth.

The consultants also report that LPILE software was utilized to analyze lateral loading behavior and evaluate the effect of structurally connecting the shoring wall to the pile caps of the LLRC building foundation system. However, based on the provided information, it is not clear how the interaction between the shoring wall and the building foundation system was evaluated and if this evaluation is consistent with the consultants' findings from analyses of potential horizontal displacement due to lateral spreading/slope instability. **CGS requests the consultants to provide further detailed discussion of their evaluation procedure and provide copies of the LPILE computer analyses input/output files for our review.** The consultants should also consider group effects in evaluation of lateral pile load-deformation behavior if piles are spaced closer than 8D (in direction of loading) in accordance with 2019 CBC Section 1810A.2.5.

CGS also notes that lateral spreading of surrounding soils to the north and south of the LLRC can cause loss of support for piles and pile cap at the edges of the structure where soils are not restrained by the tangent pile wall. We therefore **request the geotechnical consultants to evaluate whether the layout of the planned tangent pile wall will provide adequate protection against loss of support due to lateral spreading at the edges of the LLRC structure not restrained by the wall.** We further request the consultants to consider if the tangent pile wall should extend beyond the limits of the northern/southern edges of the eastern end of the LLRC structure and/or be wrapped around the northern and southern edges of the of the LLRC to provide sufficient protection for the entire LLRC structure. The structural design of the tangent pile wall should be revised in accordance with any changes to recommendations made by the geotechnical consultants in response to these comments.

Conclusion

In conclusion, ***the engineering geology and seismology issues at this site are not adequately assessed in the referenced reports***. It is recommended that additional information be provided as requested in this letter. The consultants are reminded that one copy of all supplemental documents should be submitted, should include the CGS application number, and should be uploaded directly to CGS at this link: https://www.conservation.ca.gov/cgs/upload-school. If you have any further questions about this review letter, please contact the primary reviewer at ante.mlinarevic@conservation.ca.gov.

Respectfully submitted,

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Atheria Smith
Director of Facilities Planning and Development
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Oakland, CA 94606

July 22, 2022

Subject: **Third Engineering Geology and Seismology Review for Laney College – Laney Library & Learning Resource Center 900 Fallon Street, Oakland, CA CGS Application No. 01-CGS4416**

Dear Atheria Smith:

In accordance with your request and transmittal of documents received on June 22, 2022, the California Geological Survey (CGS) has reviewed the engineering geology and seismology aspects of the consulting reports and associated design documents prepared for the subject project at Laney College in Oakland. It is our understanding that this project involves construction of a new three-story Library & Learning Resource Center (LLRC) building. This third review was performed in accordance with Title 24, California Code of Regulations, 2019 California Building Code (CBC) and followed CGS Note 48 guidelines. We reviewed the following documents for this third review of the project:

(7) **DMM Design and Recommendations, Laney College, Library Learning Resource Center, Oakland, California:** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94596; company Project No. 04.72190021, document No. 04.72190021-PR-001 (02) ADD-001, report dated June 22, 2022, 22 pages, 6 tables, 10 plates, 1 appendix.

In addition, we previously reviewed the following reports:

(1) **Geotechnical Investigation and Geologic Hazards Evaluation, Laney College, Library Learning Resource Center, Oakland, California:** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94596; company Project No. 04.72190021, document No. 04.72190021-PR-001, Issue No. 02, report dated February 28, 2020, 56 pages, 21 tables, 8 appendices.

(2) **Peralta Community College District, Laney Library & Learning Resource Center (Building 100 Replacement):** Noll & Tam Architects, 729 Heinz Avenue #7, Berkeley CA 94710; company project No. 21942, issue for permit structural/foundation plans dated November 2, 2020, 16 sheets.

(3) **Laney College Learning Resource Center Foundation Calculations:** Thornton Tomasetti, 301 Howard Street, Suite 1030, San Francisco, CA 94105, company project No. U20005.00, structural design and calculation package dated December 14, 2021, 92 pages.

(4) **Response to CGS Review Comments (CGS Application No. 01-CGS4416):** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94596; company Project No. 04.72190021, document No. 04.72190021-PR-002, Issue No. 02, report dated December 22, 2021, 18 pages, 6 tables, 3 figures, 40 plates,1 appendix.

(5) **Geotechnical Engineering Civil Plans Review for Laney College Learning & Library Resource Center Project, Oakland, California:** Fugro USA Land Inc., 1777 Botelho Drive, Suite 262, Walnut Creek, CA 94596; company document No. 04.72190021-L-001, Issue No. 01, letter report dated January 7, 2022, 2 pages.

(6) **Peralta Community College District, Laney Library & Learning Resource Center (Building 100 Replacement):** Noll & Tam Architects, 729 Heinz Avenue #7, Berkeley CA 94710; company project No. 21942, issue for bid civil design plans dated January 13, 2022, 12 sheets.

CGS previously documented our second review and submitted our findings regarding this project in a letter dated January 31, 2022, in which additional information was requested regarding the ground motion analysis, slope stability analysis, and the design recommendations for mitigation of potential lateral spreading and associated concerns regarding stability of the sloping ground between the proposed building and the adjacent Lake Merritt Channel.

Discussion of Earthquake Ground Motions

The consultants previously provided a site response analysis for the proposed improvement, as required for a Site Class F soil profile considering the significant liquefaction potential at the site. However, they now propose ground improvement and report a Site Class D soil profile for the proposed improvement, which appears to be reasonable.

The consultants' deterministic and probabilistic MCE spectra, for a reported Site Class D soil profile and default $Z_{1.0}$ and $Z_{2.5}$ parameters, appears reasonable based on comparison with results from the National Seismic Hazard Model (from Petersen and others, 2014). The consultants report their site-specific seismic design parameters are: **S_{DS} = 1.30*g*** and **S_{D1} = 1.52*g***. Alternatively, S_a values presented in the last column of Table 5.1 in report 7 may be used with the equivalent lateral force procedure, per ASCE 7, §21.4. The site-specific ground motion analysis presented appears to be reasonable and in accordance with the provisions of ASCE 7-16.

Discussion of Liquefaction and Lateral Spreading Mitigation

As discussed in our second review letter, in Report 4, the consultants adequately identified that a significant lateral spreading hazard exists in the area of the Laney College campus adjacent to the Lake Merritt channel that includes the proposed LLRC building site. Therefore, the consultants had recommended that a permanent shoring wall should be constructed along the east and southeast side of the LLRC building to mitigate the effects of lateral spreading upon the LLRC building.

In Report 7, the consultants report that due to high seismic demand, presence of shallow liquefiable soils and soft Young Bay Mud, proximity to the Lake Merritt Channel, and constraints from a utility easement on the northeast side of the planned building, **ground improvement by the deep mixing method (DMM) beneath the building footprint** is the most suitable, robust, and cost-effective technique to mitigate the liquefaction and lateral spread hazards at the planned building site. The consultants also report the DMM ground improvement will also allow for the use of a shallow foundation system for the LLRC in lieu of deep pile foundations originally planned, though they continue to recommend structural floor slabs be used (to span between the DMM panels). They recommend DMM elements be installed to depths of up to 55 feet within the southeastern portion of the building footprint to provide adequate resistance to lateral spreading and they provide a detailed slope stability analysis considering the proposed DMM system. Based on our review of Report 7, we understand **the DMM system is designed to reduce lateral spreading displacement to 4 inches or less**. The DMM system is designed as grids of overlapping mixed soil-cement columns installed using DMM equipment and methods and with a design minimum unconfined compressive strength (UCS) of 125 psi or 150 psi for minimum area replacement ratios (ARRs) of 50% and 42%, respectively. The design indicates the DMM columns will be comprised of 3 to 6-foot diameter soil-cement columns with minimum overlap of 30% of the column diameter between adjacent columns. They recommend **the depth of the DMM columns should range from 30 to 55 feet below the anticipated working platform elevation and the DMM system should reduce total seismic and static settlements to less than 2-inch, and 3/4-inch, respectively.** They provide recommended ultimate shallow foundation bearing capacity as a function of ARR beneath footings.

Altogether and based on the information provided, the geotechnical design and conceptual plans for the proposed mitigation method and the recommendations for construction by means of DMM appear to be reasonable and appropriate for the reported site conditions. The DMM system design appears to adequately address the seismic hazards to be mitigated for this project. The consultants have also provided specifications for the installation of the DMM columns that include well-defined acceptance criteria based on geometric tolerances and demonstration of strength and uniformity of mixed soil-cement materials. Therefore, **no further information regarding the DMM ground improvement design is requested by CGS at this time**. However, given the consultants' recommendation that the final depths of DMM columns should be based on interpretations of additional CPTs, we request the consultants to provide the additional information from any supplemental exploration for our review along with any revisions/updates made to the analysis, and design for the proposed DMM ground improvement system.

The design team is advised that the Division of the State Architect (DSA) may have separate comments and/or require additional information regarding specifications for the ground improvement and/or the structural design of the classroom building to be supported by improved site soils as part of their plan check review.

The geotechnical consultants should be engaged to provide monitoring of the DMM ground improvement program, including all DMM installation, verification testing, and required special inspections, under their authority as the Geotechnical Engineer of Record (GEOR) for the project. **After completion of the recommended and accepted final ground improvement program, the consultants should provide a comprehensive final report for CGS review**. The report should document their observations, testing, and analysis, including the data collected to satisfy the specified acceptance criteria. The report should include (at minimum):

- All DMM installation logs/records, field testing records, as-built plan and record of installed DMM elements, and daily field reports from both contractor and consultants' field representative(s).
- All equipment calibration reports, QA/QC data and records of DMM installation data.
- All DMM coring logs, any downhole televiewer logs, and laboratory test results, including summary and calculations of the UCS values of the DMM materials.
- Any other pertinent data gathered and/or observations made during the performance of the ground improvement program that are considered in assessing the satisfaction of the design objectives.
- Discussion and conclusion(s) regarding satisfaction of the DMM design and performance requirements for the project.

Conclusion

In conclusion, *the engineering geology and seismology issues at this site are adequately assessed in the referenced reports*. ***The project is provisionally accepted***, as we request additional documentation from 1) the consultants prior to initiating the ground improvement program; and 2) from the consultants following the completion of the ground improvement program, as discussed further above. The consultants are reminded that one copy of all supplemental documents should be submitted, should include the CGS application number, and should be uploaded directly to CGS at this link: https://www.conservation.ca.gov/cgs/upload-school. If you have any further questions about this review letter, please contact the primary reviewer at ante.mlinarevic@conservation.ca.gov

Respectfully submitted,

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Appendix J

DMM Design and

Recommendations Report

DMM Design and Recommendations

Laney College Library Learning Resource Center

CGS Application No. 01-CGS4416

Addendum | Oakland, California

04.72190021-PR-001(02)ADD-001 03 | March 31, 2023

Final

Peralta Community College District

Document Control

Document Information

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Document Title	DMM Design and Recommendations
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Fugro Document No.	04.72190021-PR-001(02)ADD-001
Issue Number	03
Issue Status	Final
Fugro Legal Entity	Fugro USA Land, Inc.
Issuing Office Address	1777 Botelho Drive, Suite 262, Walnut Creek, California 94596

Client Information

Client	Peralta Community College District
Client Address	900 Fallon Street, Oakland, California 94607
Client Contact	Albert Wege

Revision History

Issue	Date	Status	Comments on Content	Prepared By	Checked By	Approved By
01	June 10, 2022	For Review	For Client Review	RR/AP	JU	RLB
02	June 22, 2022	Final	For CGS Review	RR/AP	JU	RLB
03	March 31, 2023	Final	For DSA Review	RR/AP	RLB	RLB

Project Team

Initials	Name	Role
RR	Reza Rahimnejad	Senior Engineer
AP	Adam Price	Senior Project Engineer
JU	Jose Ugalde	Principal Engineer
RLB	Ronald L. Bajuniemi	Senior Principal

FUGRO
Fugro USA Land, Inc.
1777 Botelho Drive, Suite 262
Walnut Creek, California 94596

Peralta Community College District
333 E 8th Street
Oakland, California 94606

March 31, 2023

Dear Mr. Daniels,

Fugro is pleased to submit this Deep Mixing Method (DMM) design and recommendation addendum to the Peralta Community College presenting our methodologies for the design of DMM, conclusions and recommendations for DMM installations at the proposed Laney College Library & Learning Resource Center (LLRC), Oakland, California. The DMM performance specifications which were prepared in conjunction with this addendum are attached as Appendix A.

Fugro previously performed geotechnical investigation and geologic hazards evaluation for the Laney College Learning & Library Resource Center Project and our results were presented in a report dated February 28, 2020. This design and recommendation addendum is meant to supplement and/or supersede relevant design and construction recommendations provided in our previous report for building foundations and DMM ground improvement. This Addendum is attached as an Appendix J to the updated geotechnical investigation report dated March 31, 2023.

The below opinions, conclusions, and recommendations are meant to supplement our previous report; all previous conditions and limitations apply. If you have any questions regarding the information presented in this addendum, please contact us.

Sincerely,

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1. Introduction

Liquefaction and seismic slope stability analyses performed during project geotechnical investigation and geologic hazards evaluation, indicated potential for significant lateral spreading (up to several feet) and liquefaction-induced settlements (generally 1 to 4 inches and up to about 6 inches closer to the Lake Merritt Channel). The loose to medium dense sand layers of various thicknesses located both above and within the Young Bay Mud layer have a high potential for liquefying when subjected to an MCE (Maximum Considered Earthquake) event. These sand layers were encountered within depths of about 30 to 40 feet (above elevation -15 feet). The seismic slope stability at the planned building location is affected by both liquefaction and the presence of relatively soft Young Bay Mud (YBM). For further description of the seismic slope stability and liquefaction hazards refer to the project geotechnical report (Fugro, 2020).

It has been decided that the foundation soil will be improved using Deep Mixing Method (DMM) columns and grids under the entire footprint of the Laney College Library & Learning Resource Center (LLRC) building, which will be supported by shallow foundations (e.g., footings, and structural slab, and grade beams). This Addendum presents our methodology for the DMM design, provide DMM specifications, and to provide geotechnical design parameters for the design of the LLRC structure and its foundation system.

2. Proposed Structure

The location of the LLRC building is shown on **Plate 1**. The proposed new structure will be constructed at approximately the existing grades without basements. The footprint of the LLRC building is approximately 23,750 square feet. This building will be supported by shallow spread footings with interior structural first floor slabs and grade beams.

3. Subsurface Conditions

The subsurface soils below the site generally consist of predominately medium dense sandy fills that extend to depths of about 8 to 25 feet (Elevations of about +8 feet to -5 feet). Clayey fills of about 2 to 4 feet thick were also encountered in some areas. These fills are heterogenous and locally contain various amounts of concrete, brick, and wood debris. Most of these fills appear to be derived from the historical filling of the natural Lake Merritt outlet channel between 1860s and 1940s, and the later development of the Laney College campus in 1960s. Most likely these fills were not compacted to current acceptable geotechnical engineering standards.

Below the surficial fill layer, very soft to soft, high moisture content, and low shear strength Young Bay Mud was encountered to a depth of about 30 feet (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet (Elevation of about -30 feet) at the southeast side of the proposed building location. Some thin loose to medium dense sand lenses about 2 to 6 feet thick were also encountered within the Young Bay Mud layer. About 15-feet of loose to medium dense sands were also encountered between the surficial fill and the Young Bay Mud layers at the east edge of the building, extending towards the channel. These sands could be either historical fills placed in the natural Lake Merritt outlet channel or natural sand deposits that existed within the channel.

4. DMM Ground Improvement

4.1 Purpose

Several alternatives were considered for mitigating the lateral spread hazard at the planned building site, including installation of a retaining wall and the deep mixing method (DMM) beneath the building footprint. Considering the high seismic demand, presence of shallow liquefiable soils and soft Young Bay Mud, proximity to the Lake Merritt Channel, and constraints from the PG&E easement on the north side of the planned building, it is our experience and opinion that continuous grids of deep mixed shear walls are the most suitable, robust, and cost-effective technique to mitigate the lateral spread hazard at the planned building site. The grids of deep mixed shear walls will provide support for shallow foundation systems for seismic loading and transfer bearing loads deeper to the medium dense to very dense sands and stiff to hard clays, reducing total and differential building settlements. In addition, we recommend using structural slabs to span between DMM deep mixed shear walls, assuming that the untreated soils within the grid walls may still develop post-liquefaction reconsolidation settlements below slabs. The deep mixed shear walls will also affect the composite ground response to horizontal ground motions. This section presents a brief overview of the deep mixing method (DMM), our design approach, DMM design properties, and results of our evaluation process, including results of seismic stability analyses. Seismic design parameters incorporating the composite response of the deep mixed zone are presented in **Section 5**, herein.

4.2 Description of the Deep Mixing Method

The deep mixing method (DMM) is a soil improvement technique used to treat soils in place without excavation or dewatering. A rig that is typically equipped with multi-shaft mixing augers (containing auger flights and mixing paddles) is used to inject a cementitious grout and blend it with the in-situ soils. When the design depth is reached, the augers are withdrawn while mixing on the way to the surface, leaving in-place a stabilized soil mass that is stronger, less permeable, and has improved engineering properties. A multi-shaft mixing rig creates interconnected soil mixed elements formed by partially overlapping columns. The elements can be arranged to form walls, grids, and blocks of deep mixed soil-cement. There are various diameters of multi-shaft mixing augers and they typically range from 3 to 5 feet.

While there are other methods of creating deep mixed grids, such as by Cutter Soil Mixing (CSM) or Trench Cutting and Remixing (TRD), we believe multi-shaft auger systems will be most efficient for creating the deep mixed grids, and there are several contractors locally who have such systems. We do not recommend using single shaft soil mixing equipment for creating deep mixed shear walls and grids because, in our experience, uniform mixing is more difficult to control when using single shaft soil mixing equipment.

The body of literature (case histories, numerical simulations, and physical model tests) on the effectiveness of DMM grids for mitigating liquefaction effects demonstrates that grid configurations are more effective than columns, with benefits including reduced ground settlements and lateral spread displacements, reduced earthquake-induced shear stress and strain within untreated soils bounded by the grid walls, containment of liquefied soils within the grid walls if liquefaction occurs, and reduced migration of excess pore pressure between unimproved and improved zones (Namikawa et al., 2007; Siddharthan & Porbaha, 2008; Nguyen et al., 2013; Yamashita et al., 2015; Tsukuni & Uchida, 2015 and 2017; Boulanger et al, 2018; Boulanger & Shao, 2021). The effectiveness of DMM grids to mitigate liquefaction-induced displacements depends on a variety of factors, including the treatment geometry and area replacement ratio (A_r) and deep mixed ground strength and stiffness. The area replacement ratio is defined as the ratio of the surface area of treated soil-cement to the total surface area within a given treatment zone. As area replacement ratio increases, the composite shear strength of the deep mixed zone increases, and earthquake-induced shear stresses decrease.

4.3 Design Approach

Design of the DMM ground improvement generally follows the approach described by the FHWA guidelines (Bruce et al., 2013) and involves the following steps:

1. Establish trial geometry (area replacement ratio, column diameter, and shear wall spacing) and deep mixed ground properties,
2. Evaluate global slope stability (static, seismic, and post-seismic),
3. Evaluate other potential external modes of failure of the deep mixed zone (overturning and bearing)
4. Evaluate internal stability of the deep mixed zone (racking failure, crushing of deep mixed shear walls at the outside toe),
5. Evaluate static and seismic settlements,
6. Repeat steps 1-5 until performance is satisfactory,
7. Evaluate deep mixed column bearing capacity for support of structural loads, and
8. Refine layout and add additional deep mixed columns to reduce floor free span distance, where appropriate, to reduce floor slab costs.

The following sections describe the evaluation process for the key steps (1 through 5) listed above.

4.4 DMM Design Properties

DMM design properties and geometries were initially selected based on rule of thumb and review of relevant case histories and published design guidelines and technical papers on the subject. Following the design approach presented above, DMM properties and geometries were iteratively adjusted to achieve acceptable performance. The final DMM properties used for design are summarized in the **Table 4.1**.

Table 4.1: DMM Design Properties

Parameter	Design Value
Unconfined Compressive Strength ($q_{dm,spec}$)	125 psi
Shear Strength (s_{dm})	74 psi (curing time = 365 days) (40% of unconfined compressive strength)
Young's Modulus (secant modulus at 50% mobilized strength; E_{50})	37,500 psi (300 times unconfined compressive strength)
Shear Modulus Ratio ($G_r = G_{dm}/ G_{soil}$)	5-20
*dm = deep mixed	

4.4.1 Area Replacement Ratio

An area replacement ratio (Ar) of 50 percent was selected for design to limit lateral spread displacement to an acceptable magnitude. The basis for tolerable lateral spread displacement and our evaluation of seismic slope displacement are presented in the following section. An area replacement ratio of 50 percent with conventional DMM strengths will also provide adequate support for all footings and moment frame grade beams. In addition, case histories indicate good performance against soil liquefaction hazards with A_r greater than approximately 20 percent.

For example, the 14-story International Hotel in Kobe Japan used an A_r of about 20 percent. During the Kobe earthquake, this structure performed very well despite having the ground surrounding the building liquefy, laterally spread several meters, and settle significantly. DMM grids of about 34 feet deep and with an A_r of about 30 percent were reportedly used for several two-story new school buildings at Jordan High School in Long Beach, California (completed in 2017) to reduce liquefaction-induced settlements and support the buildings on shallow foundation systems. A design 28-days DMM unconfined compressive strength of 150 psi was used at the Jordan High School project.

The West Dowling Road Overcrossing in Anchorage Alaska was built in 2014 with deep mixed shear walls and columns supporting the approach abutment footings in part to mitigate earthquake-induced lateral deformations within shallow soft peat and liquefiable silt layers (Boulanger & Shao, 2021). The deep mixed walls and columns were spaced to produce area replacement ratios of approximately 90 percent beneath the footings (shear walls with overlapping columns) and 50 percent in the area surrounding the footings (shear walls only). The bridge performed well in the 2018 Anchorage earthquake (M = 7.1 and PGA at the overcrossing estimated to be 0.35 to 0.45) with deformations kept to acceptable levels.

We have used the deep mixing ground improvement method and our latest experience was the Agnews Campus project located in San Jose, California. The project included sixteen one- to three-story building on an approximately 55-acre site. For that project, estimated liquefaction-induced ground surface settlements exceeded the project settlement design criteria for buildings and structures that were supported on shallow foundations without ground improvement. DMM grids of about 40 feet deep with an A_r of 40% and unconfined compressive strength of 250 psi was used for the ground DMM design.

Based on available literature, case histories, and the analyses presented in the following sections, we judge that a minimum A_r of 40 to 50 percent is reasonable for support of the planned Laney College LLRC building. The design calculations presented in the following sections are based on A_r = 50%.

4.5 Global Slope Stability Analyses

4.5.1 Seismic Slope Displacement

The methodology and results of seismic slope displacement evaluations for the DMM ground improvement are presented in this section. Based on results of the stability analysis and variable Young Bay Mud (YBM) thickness encountered at the site, we performed 10 additional CPTs on November 17, 18, and 22, 2022 to better define the bottom elevation of the YBM layer. The new cross sections are shown on **Plates 2a through 2f**, herein. The updated cross sections together with results of stability analysis were used to develop the DMM depths and the zonation shown on **Plate 1**.

4.5.1.1 Methodology

Simplified Seismic Slope Displacement Procedures

Seismic slope displacement was evaluated for an idealized section representative of cross section A-A’ [**Plate 1**; the interpretive cross section is presented in the LLRC Geotechnical Report (Fugro, 2020)] using the simplified procedures developed by Bray and Macedo (2019) and Rathje and Antonakos (2011). In general, these procedures are based on regression of Newmark sliding block type analyses performed for a wide range of slope conditions (i.e., slope height, soil stiffness, and yield acceleration) and substantial databases of ground motions. The two models used herein differ with respect to:

1. Their representation of the dynamic response of the sliding block,
2. The ground motion databases available at the time of their development, and
3. Their parameterization of the ground motion for regression (for building their predictive models).

The Bray and Macedo (2019) procedure is based on fully coupled stick-slip sliding block analyses and the NGA-West2 ground motion database (>6,000 ground motion recordings were used to develop their predictive model). Their coupled model simultaneously captures the nonlinear dynamic response of the sliding mass and its effect on sliding episodes. The Rathje and Antonakos (2011) procedure is based on decoupled analyses, where calculations for the dynamic response of the sliding block and plastic slip (i.e., sliding) are performed independently. Their predictive model is based on an earlier version of the NGA strong ground motion database (>2,000 ground motion recordings were used to develop their predictive model). Coupled analyses are more rigorous and considered superior, although any sliding block type analysis represents potential slope deformations with a very simplistic failure mechanism, and results should be interpreted as an index of slope performance. While both models use earthquake magnitude as a proxy for shaking duration, they employ different parameterization of the seismic demand. The Bray and Macedo (2019) model uses the spectral acceleration (at the base of the sliding mass) at a degraded period equal to 1.3 times the initial period of the sliding mass to represent the seismic demand. The Rathje and Antonakos (2011) model used herein uses the PGA at the base of the sliding mass to represent the seismic demand. Both models require the initial fundamental period of the potential sliding mass (Ts) and the slope's yield coefficient (ky).

The initial fundamental period of the potential sliding mass (T_s) was estimated based on the approximate height of the potential sliding mass observed in the pseudostatic limit equilibrium analyses, the range of in-situ shear wave velocities previously idealized for site response analysis (Appendix G of LLRC Geotech Report), shear modulus ratios ($Gr = G_{dm}/ G_{soil}$) ranging from 5 to 20, an area replacement ratio of 50 percent, and the model for shear wave velocity ratio for periodic grid inclusions proposed by Nguyen et al. (2013). The resulting estimates of T_s ranged from approximately 0.17 to 0.28 seconds.

Seismic slope displacements were estimated for a design-level ground motion based on the geometric mean and without risk coefficients (e.g., PGA_M / 1.5) [CGS Note 48 (CGS, 2019)]. Acceleration response spectra for the MCE_R [tabulated in Table G.4 of the LLRC Geotech Report (Fugro, 2020)], MCE_G, and the design-level ground motion used for seismic slope displacement analyses are shown in **Plate 3** for V_{s30} = 260 m/s (i.e., at the base of the YBM). The range of spectral accelerations used for the Bray and Macedo (2019) seismic slope displacement model (corresponding to the estimated range of degraded period of the sliding mass) is annotated on this plate. An earthquake magnitude of 7.6 was used in these analyses based on the maximum considered earthquake associated with the Hayward Fault.

Displacements were estimated using the Bray and Macedo (2019) procedure for both ordinary ground motions and near-fault pulse ground motions. The results presented in the following section were weighted by the expected proportion of pulse motions, which was estimated to be approximately 0.4 based on the Hayden et al. (2014) model. A peak ground velocity (PGV) of 105 cm/s was used for the pulse ground motion predictive model based on the correlation developed by Watson-Lamprey and Abrahamson (2006), which was found to be in good agreement with the 1,000-year PGV computed using the beta web API for the 2018 USGS national seismic hazard maps.

Pseudostatic Slope Stability Analyses and Estimation of Yield Coefficient

The yield coefficient (i.e., the horizontal seismic coefficient that results in a pseudostatic factor of safety of unity) was evaluated by pseudostatic limit equilibrium analyses performed with the commercial software program SLOPE/W (GeoStudio 2019 version 10.0.0.18569; GEOSLOPE, 2019) and the idealized stratigraphy presented in **Table 4.2**. Circular and non-circular slip surfaces were evaluated using the Morgenstern-Price limit equilibrium method (which satisfies equilibrium of both forces and moments). The design of the DMM ground improvement was iteratively adjusted until pseudostatic stability analyses produced yield coefficient values corresponding to tolerable seismic displacements.

The deep mixing treatment zone was represented with composite properties assuming no shear resistance from native soil between the deep mixed grids. The shear strength of the deep mixed ground was computed as $s_{dm} = ½(f_r \times f_c \times q_{dm,spec})$ where $q_{dm,spec}$ is the specified unconfined compressive strength of the deep mixed ground ($q_{dm,spec}$ = 125 psi), f_c is a factor accounting for curing time (f_c = 1.48 for the 365 day curing time assumed for seismic load cases), and f_r is a factor accounting for differences between unconfined peak and confined large-strain strengths taken as 0.8 (Bruce et al., 2013). The composite shear strength of the treatment zone was then estimated as $s_{dm,grid} = f_v \times A_r \times s_{dm} \approx 5{,}000$ psf where f_v is a factor that accounts for the greater variability that typically exists in the strength of deep mixed ground compared to the variability that exists in the strength of clay deposits [f_v was estimated to be 0.95 per the FHWA guidelines (Bruce et al., 2013)], A_r is the area replacement ratio (A_r = 0.5), and s_{dm} is the shear strength of the deep mixed ground defined above.

The Young Bay Mud (YBM) was modelled with an undrained shear strength ratio of 0.22 to approximate cyclic softening behaviours. This softened strength ratio was based on an average peak, static strength ratio of 0.28 and an undrained shear strength reduction factor of 0.8 (i.e., 0.8 × 0.28 = 0.22) based on the cyclic strength (i.e., cyclic resistance ratio, CRR) for M=7.5

suggested by Idriss and Boulanger (2008) for plastic silts and clays, and Fugro's past experience characterizing and modelling YBM.

The liquefiable sands were modelled with residual strength of liquefied soil estimated using the Kramer and Wang (2015) model which depends on energy and effective overburden corrected SPT blow count, $(N_1)_{60}$ and in-situ vertical effective stress. Baseline analyses were performed for $(N_1)_{60}$ =15. Sensitivity analyses were also performed for $(N_1)_{60}$ =10.

Table 4.2: Soil Properties Used in Pseudostatic Slope Stability Analyses

Material	Unit Weight (pcf)	Material Shear Strength	
		Cohesion c′ (psf)	Friction Angle Φ′ (degree)
Sandy Fill	120	0	35
Young Bay Mud with Sand Lenses	90	0.22 x Effective Overburden Stress (psf)	0
Interbedded Clays and Sands	130	0	40
Highly Liquefiable Sands	110	Kramer and Wang (2015) $(N_1)_{60}$=15	0
DMM Composite	120	5,000	0

4.5.1.2 Results

Results of the pseudostatic slope stability analyses and simplified seismic slope displacement estimates are presented herein. **Plates 4a through 4c** show factors of safety and corresponding slip surfaces from pseudostatic stability analyses with a horizontal seismic coefficient (k_h) of 0.35 for block, circular, and optimized circular slip surfaces. The block slip surface (passing through the DMM grid) and the optimized circular slip surface (passing beneath the grid) both have factors of safety of approximately 1.0 (i.e., k_y = 0.35). The circular slip surface (not optimized) exhibits a factor of safety of 1.1 for k_h = 0.35 (i.e., k_y < 0.35). The deep mixed zone is deeper on the east side of the building (Zones A2 and B in **Plate 1**) extending approximately 55 feet deep (Elev. -35 feet; 12 to 22 feet below YBM). This deeper section is approximately 55 feet wide in the slope stability model. Based on the results of the stability analyses the deeper portion of the DMM is extended further back along the north side of the building so that there is at least a 55-foot-wide deep buttress for slip surfaces oblique to the building orientation (e.g., Cross Section F-F′, **Plate 2f**). Additionally, based on the additional CPTs performed in November 2022 the DMM depth was extended in Zone A2 to ensure bearing into the competent sands and clays beneath the YBM. A smaller maximum center-to-center grid spacing is specified for Zone B (**Plate 4**). The deep mixed zone on the western side of the building is shorter (Zone A1 in **Plate 1**), keyed 3-5 feet into the competent sands and clay under the YBM, generally extending to a depth of about 33 feet below ground surface (Elev. -15 feet). Sensitivity analyses with residual strength of liquefied soils based on Kramer and Wang (2015) for $(N_1)_{60}$ =10 had a small effect on the results because only small fractions of the slip surfaces were affected.

The range of estimated seismic displacements for the DMM ground improvement is shown in **Plate 5**. The range is based on: (1) the range of k_y values computed for circular and non-circular slip surfaces and several parameter sensitivity analyses, (2) reasonable ranges of average shear wave velocity of the potential sliding mass, and (3) both the Bray and Macedo (2019) and Rathje and Antonakos (2011) predictive models. Estimated median seismic displacements range from negligible to approximately 13 cm, with the Bray and Macedo (2019) model producing larger displacement estimates for all cases. The T_s = 0.17 and 0.21 seconds analysis cases likely better represent the deep mixed zone in these analyses (Gr ≈ 20 and 10, respectively). The T_s = 0.28 seconds case was included as a reasonable sensitivity analysis where the degraded period of the sliding mass corresponds to the peak spectral acceleration (i.e., $Sa(1.3T_s)$ = 1.2 g). The differences between the two predictive models are partly attributed to differences in how they model the dynamic response of the sliding mass, with the Bray and Macedo (2019) coupled model considered superior. The approximate performance of the existing slope (i.e., without DMM ground improvement) is also annotated on **Plate 5** for reference. Large seismic displacements were estimated for the existing slope for $k_y \approx 0.1$, which corresponds to a range of performance where displacement estimates are very sensitive to small changes in yield acceleration. Conversely, the DMM ground improvement performance falls on a much flatter part of the displacement vs. k_y curves.

Two seismic displacement thresholds are indicated on **Plate 5**. The 15-cm (6-inch) threshold is commonly accepted for screening-level evaluations of earthquake-induced landslide hazard (e.g., SP-117A, 2008). The 15-cm threshold likely distinguishes small to moderate displacements from larger displacements (Blake et al., 2002), and sliding block displacement estimates less than approximately 15 centimetres are unlikely to correspond to serious landslide movement or damage (SP-117A, 2007). The 10-cm (4-inch) threshold corresponds to the ASCE 7-16 upper limit for lateral spreading horizontal ground displacement for shallow foundations for buildings in Risk Category IV (Table 12.13-2 of ASCE 7-16). Therefore, we designed the DMM ground improvement to limit average lateral spread displacement to approximately 10 cm. This corresponds to a yield acceleration of approximately 0.35 (**Plate 4**) which was computed for the best estimate pseudostatic stability analyses previously presented on **Plates 4a and 4c**.

4.5.1.3 Conclusions

Seismic slope displacement estimates based on two methods and incorporating uncertainty in the initial fundamental period of the sliding mass were on average less than 10 cm and are considered tolerable given the ASCE 7-16 upper limit for lateral spreading horizontal ground displacement for shallow foundations (10 cm for buildings in Risk Category IV). Therefore, the design of the DMM ground improvement presented herein is judged to be acceptable with respect to global seismic stability.

The deeper treatment zone modelled on the east side of the building in the two-dimensional pseudostatic stability analyses will need to also wrap around the north side of the building to limit lateral displacement for slip surfaces oblique to the building's principal orientation and to protect the north side of the building against potential soil loss. This configuration is shown in plan on **Plate 1** and provides a similar width of deeper treatment (to what was modelled for Section A-A′) for potential slip surfaces on cross sections oblique to the building orientation. The deeper ground improvement extends along the north side of the LLRC building to the eastern limit of the Building E.

The final depth of the DMM grid should be determined based on the results and interpretation of additional CPTs that we recommend be performed prior to construction. The DMM depths presented in this section (and used for the stability analyses) are minimum depths that may need to be exceeded based on interpretation of the additional CPTs.

4.5.2 Post-Seismic Global Stability

Post-seismic stability analyses demonstrated factors of safety greater than five. Post-seismic global stability was evaluated using the same properties shown in **Table 4.2**, except the YBM was modelled with an undrained shear strength ratio of 0.17 (i.e., a strength reduction factor of 0.6 to represent cyclically softened strength for a static loading rate). Analyses were performed for both the original slope geometry and for a case where the channel side soils (east of the building) are assumed to have displaced towards the channel more than the building, exposing a free face of deep mixed ground approximately 20 feet tall. In both cases the DMM ground improvement is acceptable from a post-seismic global stability perspective.

4.6 Additional Stability Checks

Additional stability checks were performed following FHWA guidelines (Bruce et al., 2013) and including seismic loads. Overall, the design of the DMM ground improvement was controlled by global seismic slope displacements given the favourable aspect ratio and relatively high area replacement ratio needed. External stability was checked for combined overturning and bearing. Internal stability was checked for crushing of the deep mixed shear walls at the outside toe, racking failure (shearing on vertical planes in the deep mixed shear walls), and extrusion of soil between the deep mixed shear walls.

To simplify these checks, we conservatively modelled the shallower treatment zone on the west end of the building and the deeper treatment zone on the east end of the building as separate blocks of soil-cement. Seismic activate earth pressures were estimated using the same limit-equilibrium models used for the pseudostatic analyses and with k_h = 0.35 (following the GLE approach described by Anderson et al., 2008). A horizontal inertial load of the deep mixed ground was also included as the weight of the deep mixed ground (W) times the horizontal seismic coeffect (k_h). Additionally, no passive resistance from the channel side of the deep mixed ground was conservatively assumed. Factors of safety for all additional stability checks were more than 1.3 [the minimum value for static load cases recommended by Bruce et al. (2013)].

4.7 DMM Treatment Zone Settlements

Post-liquefaction reconsolidation settlement within the building footprint was estimated to range between approximately 1 and 4 inches based on the local borings and CPTs. Note that the largest settlement (approximately 6 inches) was estimated for CPT-03 which was performed to the southeast of the building. The potential for the DMM to reduce cyclic shear stresses and limit post-liquefaction reconsolidation settlement was evaluated for shear modulus ratios ($G_r = G_{dm} / G_{soil}$) ranging from 5 to 20 and the design area replacement ratio of 50%. The shear stress reduction factor ($R_d = CSR_I / CSR_U$, where CSR_I and CSR_U and cyclic stress ratio for the improved and unimproved cases, respectively) was estimated using the relationship proposed by Nguyen et al. (2013) for periodic grid arrangement of shear walls. CPT-based post-liquefaction reconsolidation settlement analyses for R_d values ranging from 0.5 (A_r = 50% and G_r = 5) to 0.17 (A_r = 50% and G_r = 20) resulted in negligible (R_d = 0.5) to substantial (R_d = 0.17) reduction in estimated settlements. A R_d value of 0.3 (A_r = 50% and G_r = 10) provides a reasonable estimate of the settlement hazard that accounts for reduced seismic shear stresses in the native soil and resulted in reconsolidation settlements within the building footprint on the order of 1 to 2 inches for the MCE.

We judge that by using DMM to mitigate liquefaction effects, post-liquefaction reconsolidation settlements of 1 to 2 inches can develop between DMM grids beneath the floor slabs. Therefore, we recommend using structural slabs to span between deep mixed shear walls.

In addition, we estimate static settlement of buildings and structures supported on the deep mixed ground will depend on footing layout and service load and will be less than about 3/4 inch.

Underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed considering differential settlements of about 4 inches associated with post-liquefaction reconsolidation between DMM supported structure and unimproved areas adjacent to the building. Additional consideration should be given to the impacts of differential seismic slope movements (lateral and vertical) between the improved and unimproved areas.

5. Seismic Design Parameters

Due to the ground improvement, the combination of the DMM grids and the existing soil will create a stiffer composite medium which has a higher shear wave velocity compared to the native soft YBM and sandy fill material. We estimate that the shear wave velocity of the top 100 feet of soil (V_{s30}) will increase to 270 m/s due to the ground improvement. The corresponding composite average shear wave velocity profile was estimated based on the idealized shear wave velocity profiles previously developed for site response analyses [**Appendix G** of LLRC Geotech Report (Fugro, 2023)] and considering shear modulus ratios, G_r, ranging from 5 to 30 (based on deep mixed soil-cement E_{50} values between 300 and 600 times the specified unconfined compressive strength, as recommended by Boulanger and Shao, 2021). Overall, for the resulting range of estimated V_{s30} values the short period spectral accelerations (that control short period spectral acceleration parameters, S_{DS} and S_{MS}) increase with increasing V_{s30}. Given that the estimated building period is 0.45 seconds, we selected a representative V_{s30} based on a reasonably conservative average G_r value of 20, which for A_r = 50% corresponds with a ratio of $V_{s,av}$ / V_s of approximately 2.5 per the relationship developed by Nguyen et al. (2013) for periodic grid inclusions (where $V_{s,av}$ is the average shear wave velocity in the treatment zone and Vs is the soil's shear wave velocity) The idealized V_s profiles previously developed for site response produce V_{s30} values of approximately 270 m/s when multiplied by 2.5 over the thickness of fill and YBM.

A site-specific Probabilistic Seismic Hazard Analysis (PSHA) was performed for the new Vs30 to estimate the severity of ground motions that may affect the project site for specific design levels of hazard. The design ground motion parameters were calculated following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-16 (ASCE, 2016) as required by the California Building Code (CBC) (CBSC, 2019).

Table 5.1 tabulates the spectral ordinates of the recommended site-specific MCER and design response spectra per ASCE 7-16. The corresponding design acceleration parameters SMS, SM1, SDS, and SD1 are tabulated in **Table 5.2**.

Table 5.1: MCE$_R$ and Design Response Spectra per ASCE 7-16 for a Vs30 of 270 m/sec, 5% Damping

	Horizontal Spectral Acceleration (g)							
Period (sec)	UHRS for Return Period of 2,475 Years	Probabilistic MCE$_R$	84th Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE$_R$	Site-Specific MCE$_R$	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.947	0.959	0.731	0.555	0.804	0.804	0.400	0.536
0.03	0.974	0.987	0.739	0.561	0.812	0.812	0.459	0.542
0.05	1.09	1.11	0.810	0.615	0.891	0.891	0.518	0.594
0.075	1.35	1.37	0.96	0.73	1.06	1.06	0.591	0.704
0.1	1.58	1.60	1.1	0.837	1.21	1.21	0.664	0.807
0.15	1.87	1.90	1.33	1.01	1.47	1.47	0.811	0.978
0.19	2.06	2.09	1.47	1.11	1.61	1.61	0.927	1.08
0.2	2.1	2.13	1.5	1.14	1.65	1.65	0.927	1.1
0.25	2.27	2.36	1.62	1.26	1.83	1.83	0.927	1.22
0.3	2.39	2.52	1.72	1.36	1.97	1.97	0.927	1.32
0.4	2.44	2.64	1.81	1.48	2.14	2.14	0.927	1.43
0.5	2.36	2.62	1.79	1.50	2.17	2.17	0.927	1.45
0.75	1.95	2.24	1.53	1.33	1.93	1.93	0.927	1.29
0.949	1.68	1.97	1.36	1.21	1.75	1.75	0.927	1.17
1	1.63	1.92	1.32	1.19	1.72	1.72	0.88	1.14
1.5	1.16	1.42	0.991	0.924	1.34	1.34	0.587	0.892
2	0.891	1.12	0.785	0.752	1.09	1.09	0.44	0.726
3	0.583	0.759	0.541	0.537	0.778	0.759	0.293	0.506
4	0.413	0.551	0.389	0.396	0.573	0.551	0.22	0.367
5	0.310	0.421	0.290	0.300	0.435	0.421	0.176	0.281
7.5	0.171	0.233	0.145	0.15	0.218	0.218	0.117	0.145
8	0.154	0.21	0.128	0.133	0.192	0.192	0.110	0.128
10	0.107	0.146	0.084	0.087	0.125	0.125	0.070	0.084

Table 5.2: Design Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.95
S_{M1}	2.28
S_{DS}	1.30
S_{D1}	1.52
T_L	8 seconds

Plate 6 shows the mean annual seismic hazard curves for selected spectral periods ranging from 0.01 to 10 seconds for Vs30 of 270 m/s. A spectral period of 0.01 seconds is used to represent the peak ground acceleration (PGA). These hazard curves represent the total mean hazard from combining all seismic sources and ground motion models. These figures also indicate the annual frequency of exceedance corresponding to a return period of 2,475 years. **Plate 7** presents the 5 percent-damped mean horizontal UHRS (Uniform Hazard Response Spectrum) for a return period of 2,475 years and the representative Vs30 value of 270 m/sec. The UHRS for a return period of 2,475 years along with probabilistic response MCE_R response spectrum are illustrated in **Plate 8**.

Plate 9 presents the development of the site-specific MCE_R and design response spectra for the site. In this case, the deterministic MCE_R spectrum is lower than the probabilistic MCE_R spectrum for all spectral periods. The site-specific MCE_R spectrum is the maximum of: 1) the minimum of the probabilistic and deterministic MCE_R, and 2) 150 percent of the design response spectrum. Following ASCE 7-16, the recommended design response spectrum for the site was calculated as the maximum of 2/3 of the site-specific MCE_R and the lower limit specified by ASCE 7-16 (80 percent of the general spectrum for Site Class D, using modified F_a and F_v values provided in Section 21.3 of ASCE 7-16). The transition period from constant velocity to constant displacement, T_L, required to calculate the lower limit, was estimated as 8 seconds using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html).

6. Foundation System

6.1 Spread Footing

We anticipate an area replacement ratio (A_r) of at least 50% will be used in the DMM design. The DMM columns can be constructed in various diameters and selection of the diameter and depth of these columns is project specific. Typically, the overlap of adjacent DMM columns is about 30% of the column diameter. The A_r may vary based on number of DMM columns under the foundation. To achieve the full allowable bearing pressure, DMM should extend laterally beyond footing bases such that the area replacement ratio under the footing is 100%. Otherwise, the bearing capacity should be multiplied by the actual A_r under the foundation.

The axial capacity of the DMM grids is the minimum of the structural capacity and geotechnical capacity of the grids. For the designed DMM grids, the structural capacity is expected to control. Using a design DMM unconfined compressive strength of 125 psi, and following the FHWA design guidelines (Bruce et al., 2013), the ultimate structural capacity of the DMM columns (Ar=100%) is estimated to be 21,300 pounds per square feet (psf) for long term and seismic loads and 14,400 psf for short term construction loads (after 28 days). According to CBC Section 1605A.1.1, the factor of safety for soil bearing shall not be less than the overstrength factor. Factors of safety for allowable stress design are show in **Table 5.3** below.

Table 6.1: Factors of Safety for Axial Loading of Foundations (Allowable Stress Design)

Load Condition	Factor of Safety
Dead Load	max (3, overstrength factor)
Dead plus Live Loads	max (2, overstrength factor)
Total Loads (including wind or seismic)	max (1.5, overstrength factor)

Provided bearing capacities are for shallow foundations supported on DMM with 100% area replacement ratio (A_r). It should be noted that considering the relatively high bearing capacity of the DMM, the A_r under the footing may be decreased depending on the design loads. The allowable bearing pressure is a net value; therefore, the weight of the footing can be neglected for design purposes. For footings supported on DMM with A_r less than 100%, the bearing capacity should be reduced by multiplying by A_r.

Footings should be at least 12 inches wide, and bottom of footings should be founded at least 24 inches below the lowest adjacent finished grade. Estimated static settlement of building supported on the deep mixed ground depends on footing layout and service loads, but should be less than about 3/4 inch.

Resistance to lateral loads may be provided by friction along the base of foundations and by passive pressures acting on the sides of foundations. An allowable friction coefficient of 0.35 may be multiplied by the dead load to evaluate the allowable frictional resistance along the bottom of foundations. Where the footing is poured neat against subgrade soils, an ultimate passive pressure equal to an equivalent fluid pressure of 500 pounds per cubic foot (pcf) can be used for lateral load resistance against the sides of footings perpendicular to the direction of loading. If the footing is poured against forming, the ultimate passive resistance will be reduced by 30%. The upper 12 inches of soils should be ignored unless they are confined by pavement or slab. The passive resistance value can be linearly interpolated between at-rest pressure (equivalent to a fluid pressure of 50 pcf) at zero deflection and the ultimate at a deflection of 0.025*D, where D is the depth of the footing. The passive pressures against the footings and grade beams along the eastern and northern edge of the treatment Zone B should be ignored in foundation design due to potential for seismic soil displacements adjacent to the building in these areas.

6.2 Structural Mat Slab

When used, the structural mat slabs foundations should be supported on properly prepared subgrade that is proof rolled to provide a smooth, unyielding surface for slab support. Where the slab will be located at surface grade, we recommend at least 12 inches of imported, predominantly granular, "non-expansive" engineered fills that meet the requirements presented in the **Section 7.2** of the LLRC Geotechnical Report (Fugro, 2023) be provided below the slab. For slabs that support vehicular loads, the "non-expansive" engineered fill layer should consist of Caltrans Class 2 aggregate base.

For the portion of the slab that are supported on compacted soil, we recommend a modulus of subgrade reaction (k_1, 1 foot by 1 foot) of 125 pounds per square inch per inch (psi/in) be used for the design of the structural mat slab foundation for the static condition. This value can be modified to k as 125/B psi/in, where B is the equivalent foundation width measured in feet. However, for the seismic condition, the slabs should be able to span between the DMM grids due to liquefaction-induced settlements (i.e., subgrade modulus = 0.0).

For the portion of the slab that are supported on 100% area replacement ratio DMM, we recommend a modulus of subgrade reaction (k_1, 1 foot by 1 foot) of 4,000 psi/in be used for the design of the structural mat slab foundation. This value can be modified to k as 4,000/B psi/in, where B is the equivalent foundation width measured in feet. Recommended values of modulus of subgrade reaction for various footing aspect ratios are provided in **Table 6.1** below. For other aspect ratios, values of modulus of subgrade reaction can be linearly interpolated. Note that if the area replacement ratio is less than 100%, **rigid footing** bearing capacities provided above and **modulus of subgrade reaction** in table below shall be **reduced by** multiplying by **the** area replacement ratio.

Table 6.2: Values of Modulus of Subgrade Reaction for Various Rigid Footing Aspect Ratios

Footing Aspect Ratio Length (L) : Width (B)	Modulus of Subgrade Reaction k (psi/inch)
1	4,000/B
1.5	3,300/B
2	2,900/B
3	2,500/B
4	2,300/B
5	2,100/B
10	1,800/B
15	1,700/B

Note: B, equivalent footing width measured in feet

7. Conclusions and Recommendations

We recommend that approximately 30- to 55-foot deep DMM ground improvement be used to support shallow foundations of the proposed LLRC building to mitigate seismic hazards. We conclude that the LLRC building supported on DMM that are designed using the recommendations provided below will meet the project settlement criteria and allowable lateral spread displacement for shallow foundations per ASCE 7-16. We estimate total settlements less than ¾ inch (differential settlements up ½ inch over 30 feet or between adjacent structural columns) for DMM supported foundations and less than approximately 4 inches of average lateral spread displacement for the design-level ground motion.

A summary of our key recommendations for DMM follows:

- Ultimate DMM compressive capacities of 21,300 psf and 14,400 psf can be used for design of footings and slabs supported on DMM with 100% A_r for long term/seismic and short-term construction loads (after 28 days), respectively. For footings supported on DMM with A_r less than 100%, the bearing capacity should be reduced by multiplying by A_r.
- The allowable axial capacity should be calculated by dividing the ultimate axial capacities by the factors of safety provided in **Table 6.1**.
- Lateral resistance of footing bases and slabs supported on DMM can be calculated using an allowable frictional coefficient of 0.35.
- The DMM should be constructed using multi-shaft mixing equipment to create columns that are a minimum of 3 feet and a maximum of 6 feet in diameter.
- The overlapping between any two adjacent DMM columns should be at least 30 percent of the column diameter.

- The bottom of DMM columns should be extended to below elevation -15 feet for Zone A1 and -35 feet for Zones A2 and B.
- The design unconfined compressive strength ($q_{dm,spec}$) is 125 psi .
- The mixed-in-place soil cement grids and blocks should cover a minimum Area Replacement Ratio (A_r) of 50 percent for a specified unconfined compressive strength ($q_{dm,spec}$) of 125 psi.
- Center-to-center spacing of DMM grids should not exceed 4.0d Zones A1 and A2 and 3.2d for Zone B, for min A_r = 50% ($q_{dm,spec}$ = 125 psi)., where d is the diameter of the DMM columns.
- DMM within the building footprint should be arranged in an uninterrupted grid that follows the building column lines and underlies all footings and moment frame grade beams.
- The DMM should underlie the entire building footprint and extend laterally to include any attached structures which are deemed to be essential parts of the buildings.
- To achieve the full allowable bearing pressure, DMM should extend laterally beyond footing bases such that the area replacement ratio under the footing is 100%. Otherwise, the bearing capacity should be multiplied by the actual A_r under the foundation.
- Elevator shafts should also be supported by DMM grids.
- Ground floor slabs should be designed to structurally span between DMM walls. Vapor barrier recommendations for floor slabs are provided in Section 7.3.4 of the LLRC Geotechnical Report (Fugro, 2023).
- The top of the DMM elements should extend to the base of the ground floor slab section, the bottom of footings, and the bottom of moment frame grade beams. The bottom of DMM elements should extend at least to the minimum depth shown on **Plate 1**.
- The average unconfined compressive strength of the DMM core specimens should be at least 125 psi at 28 days for a minimum A_r of 50% as determined by ASTM D2166. Ninety percent (90%) of all unconfined compressive strength tests on core samples should exceed the specified unconfined compressive strength.
- Lumps of unimproved soils should not amount to more than 15 percent of the total volume of any core run from continuous full-depth core sample and all of the unrecovered core length should be assumed to be unimproved soil.
- Any individual or aggregation of lumps of unimproved soil should not be larger than 12 inches in greatest dimension.
- Detailed DMM acceptance criteria are provided in performance specifications in **Appendix A**.
- Before construction, a detailed utility locating report should be provided to the design team.
- The DMM Contractor should control and process all spoils created during the DMM construction and should coordinate with the project grading contractor for the spoils to be reused as fills at the project site. The DMM spoils can be used below all interior slabs-on-grade or structural mat slabs as non-expansive engineered fills provided, they meet the requirements provided in our Geotechnical Report (Fugro, March 31, 2023).

- Design parameters per ASCE 7-16 at the ground surface are tabulated in **Table 7.1** below:

Table 7.1: Design Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.95
S_{M1}	2.28
S_{DS}	1.30
S_{D1}	1.52
T_L	8 seconds

- DMM modulus of subgrade reaction (k_1, 1 foot by 1 foot) of 4,000 psi/in can be used for the structural design for portions of the slab that are supported by A_r = 100% DMM. The modulus of subgrade reaction for compacted soil between DMM grid is 125 psi/in which should be ignored when designing for seismic condition. These values can be modified to k aa k_1/B psi/in, where B is the equivalent foundation width measured in feet. DMM modulus of subgrade reaction (k) for footings with various aspect ratios are provided in Table 6.1.
- Design requirements for foundations in liquefiable sites specified in **Section 12.13.9** of **ASCE 7-16** should be used for when designing the structural members and foundation system.
- We judge that by using DMM to mitigate liquefaction effects, post-liquefaction reconsolidation settlements of up to 2 inches can develop between DMM grids beneath the floor slabs. Therefore, we recommend using structural slabs to span between deep mixed shear walls.
- We recommend DMM ground improvement be installed by a qualified specialty contractor with demonstrated experience in this type of ground improvement. Construction of uniformly mixed, high strength DMM columns requires proper equipment, trained and experienced personnel, the proper mix design for the soils encountered, careful attention to the construction procedures, continuous monitoring of the installation parameters, and sufficient quality control testing. The DMM contractor should develop construction procedures, mix design, and quality control required to achieve the desired results and meet the project design and specified acceptance criteria. Additional details regarding contractor's responsibility are included in the DMM specifications.
- Fugro should also be retained to provide geotechnical services during DMM contractor selection, construction document and drawing submittal review, and DMM implementation and testing, to observe compliance with the design concepts, specifications, and recommendations presented in this addendum and the project Geotechnical Report (Fugro, , March 31, 2023). Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered.

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List of Plates

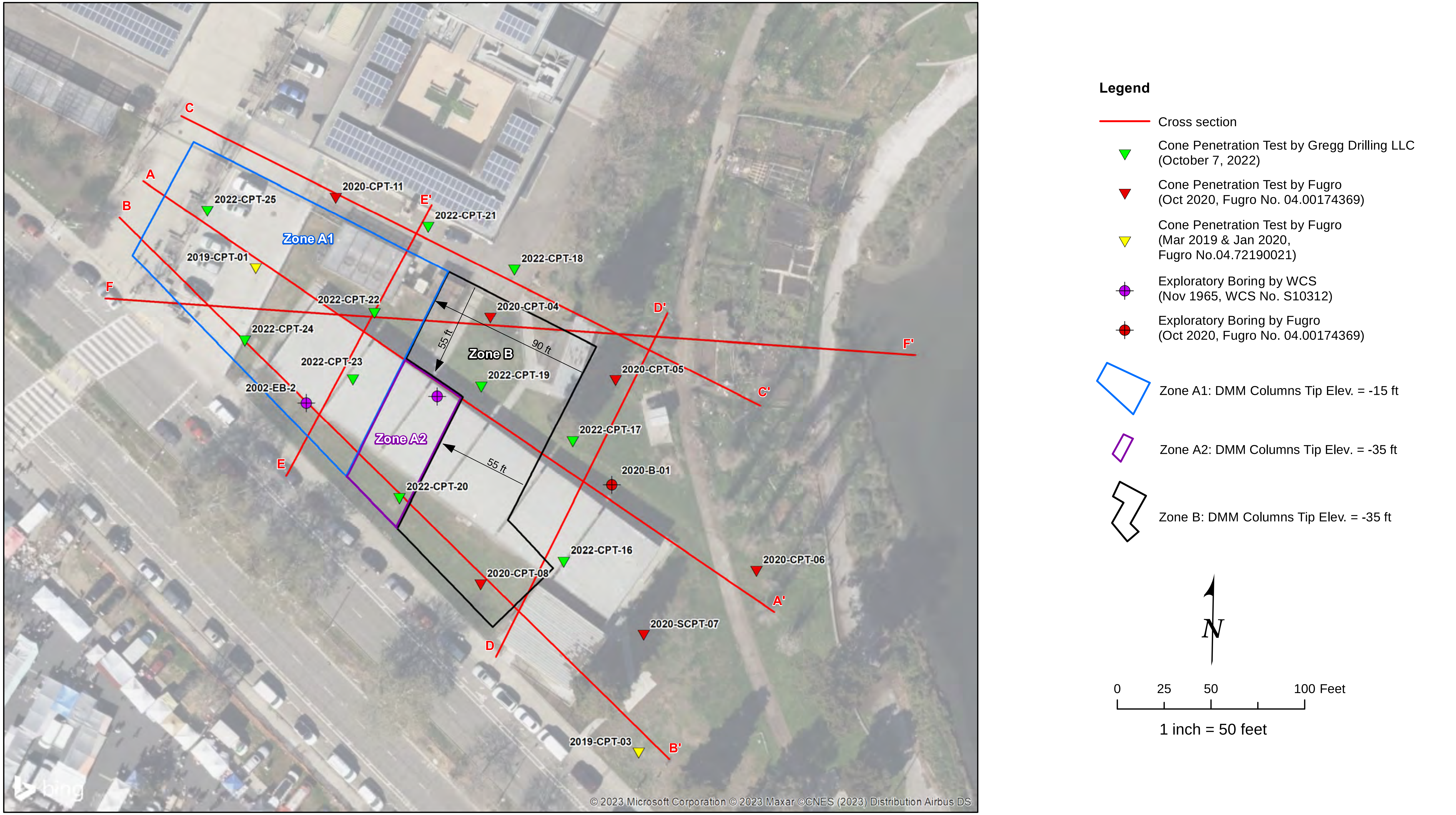

Plate 1: Ground Improvement Plan

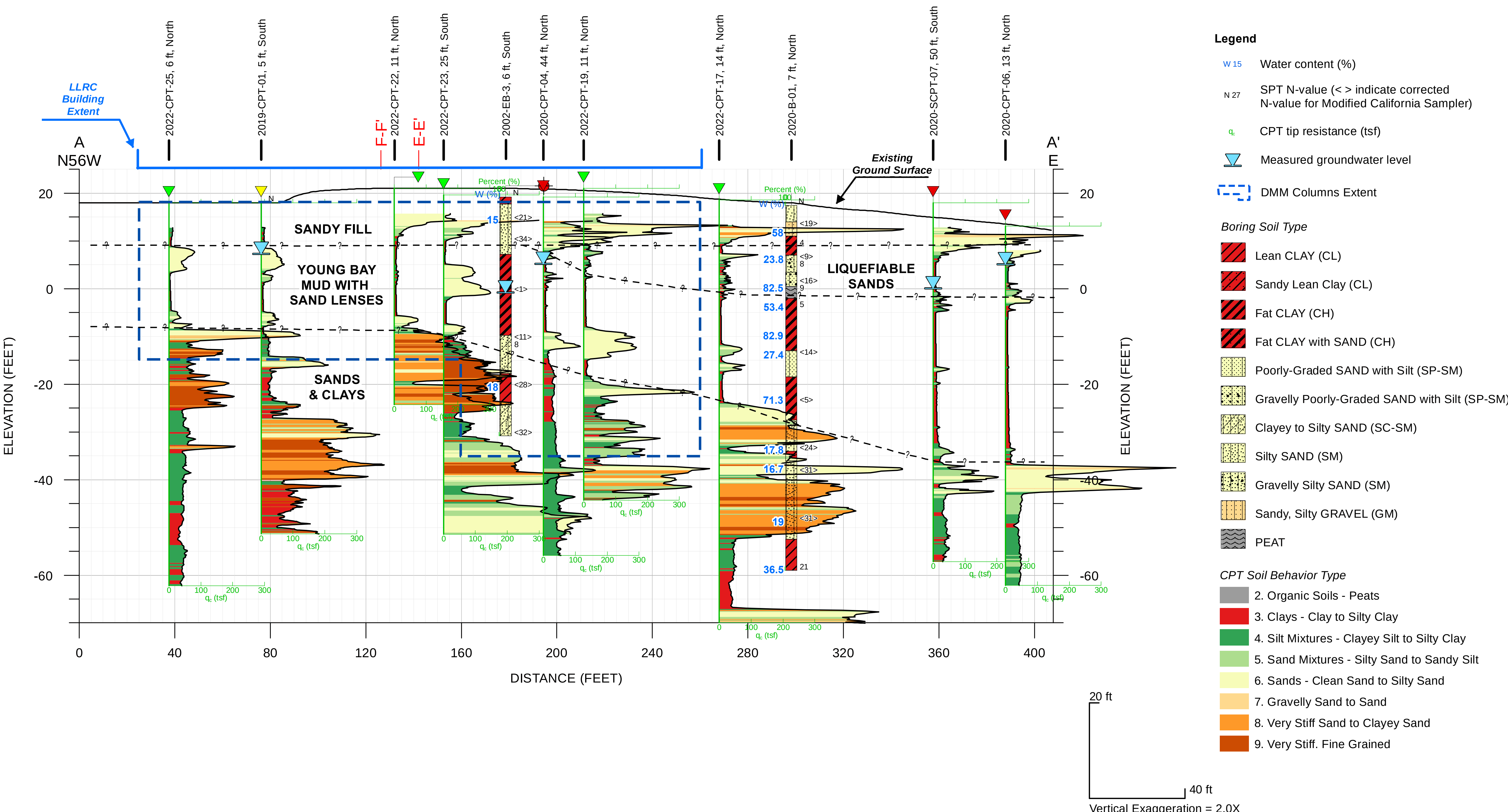

Plate 2a: Cross Section A-A'

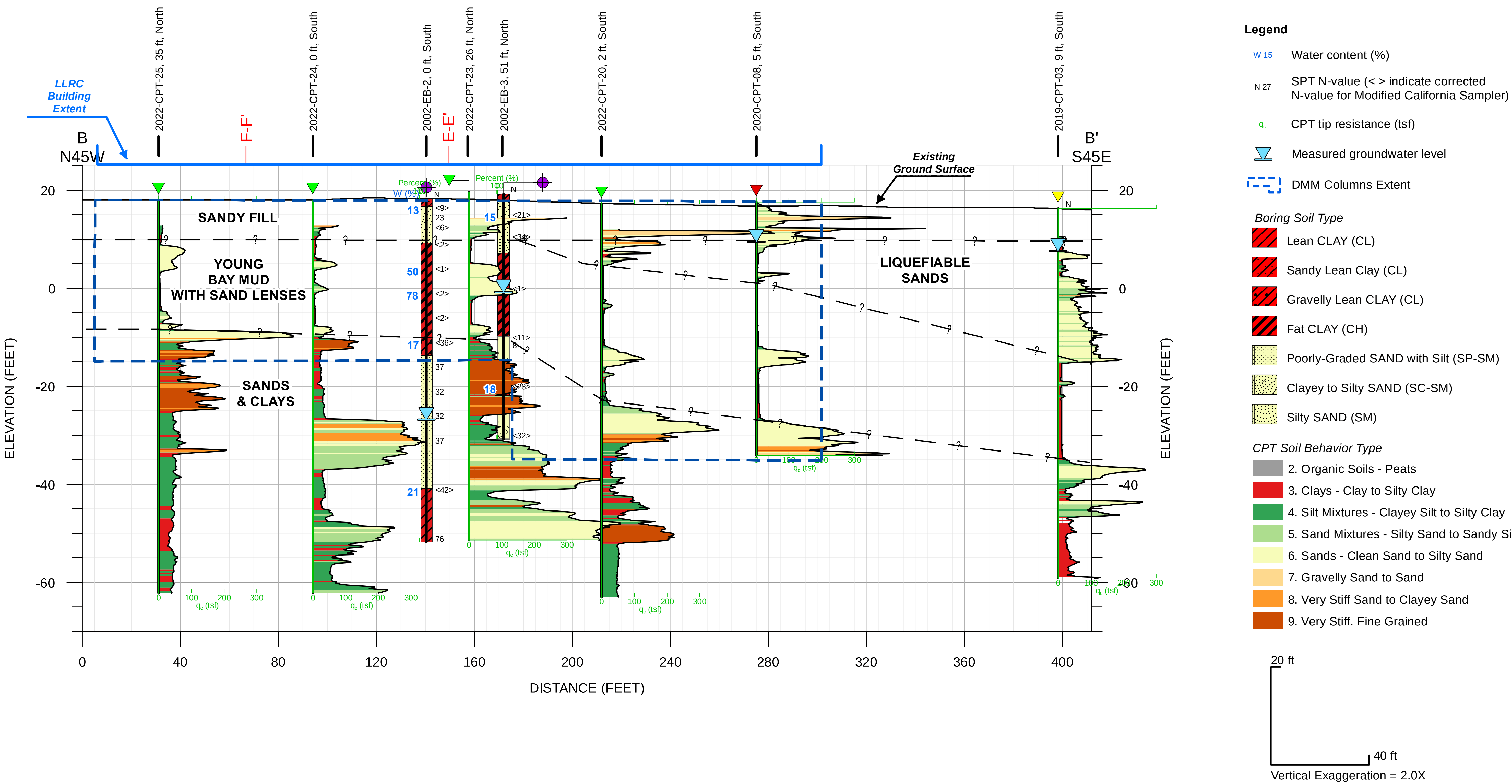

Plate 2b: Cross Section B-B'

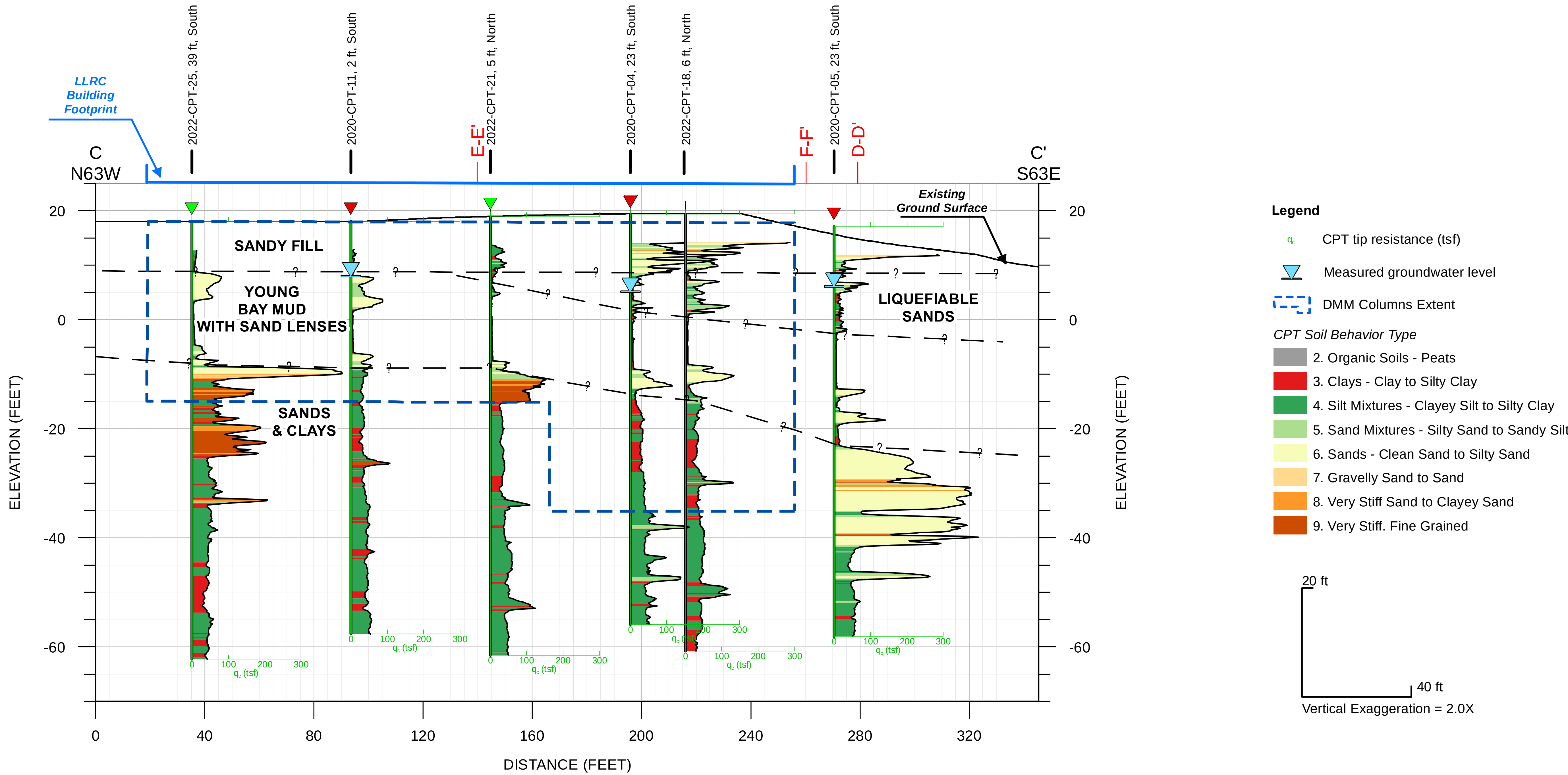

Plate 2c: Cross Section C-C'

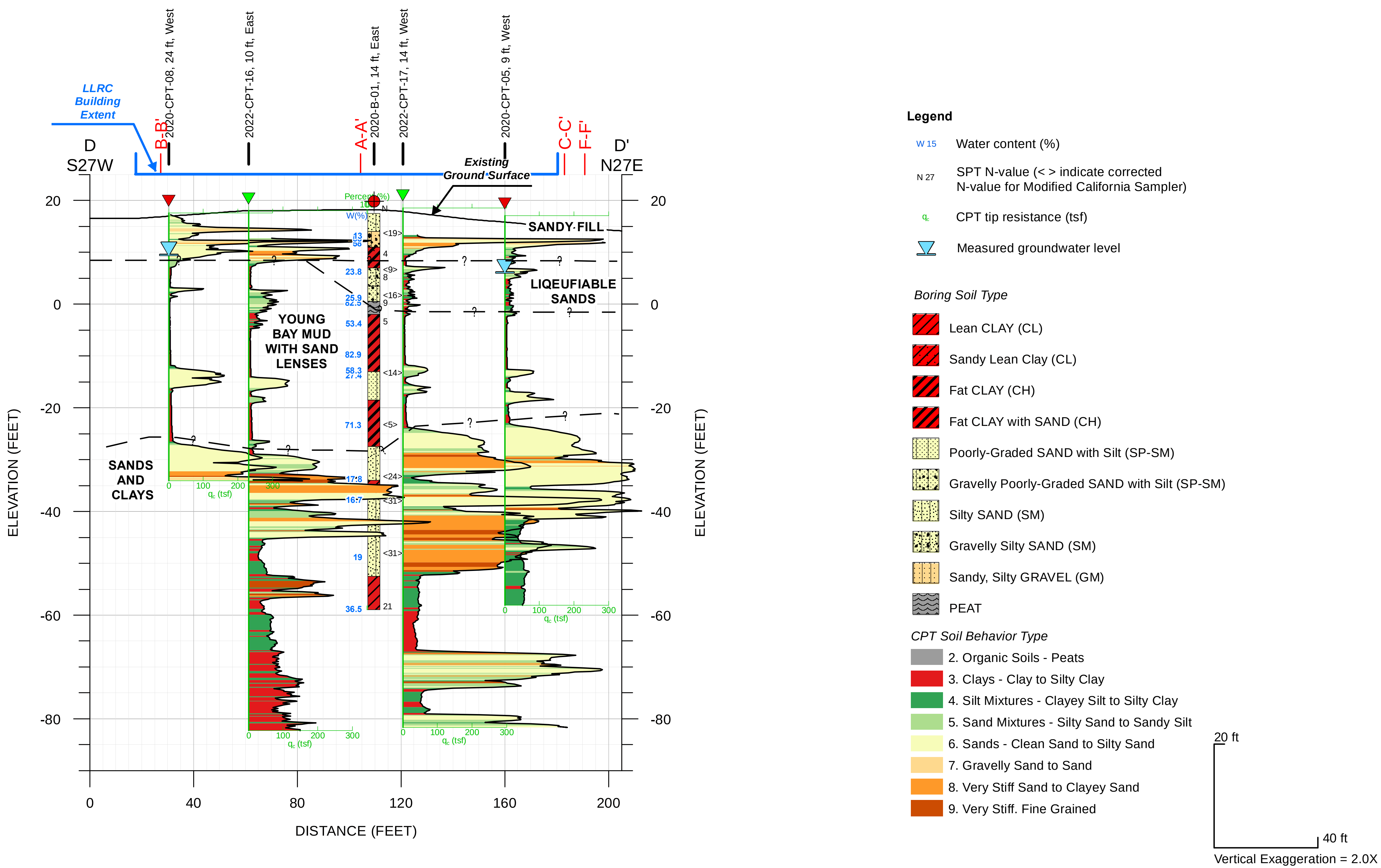

Plate 2d: Cross Section D-D'

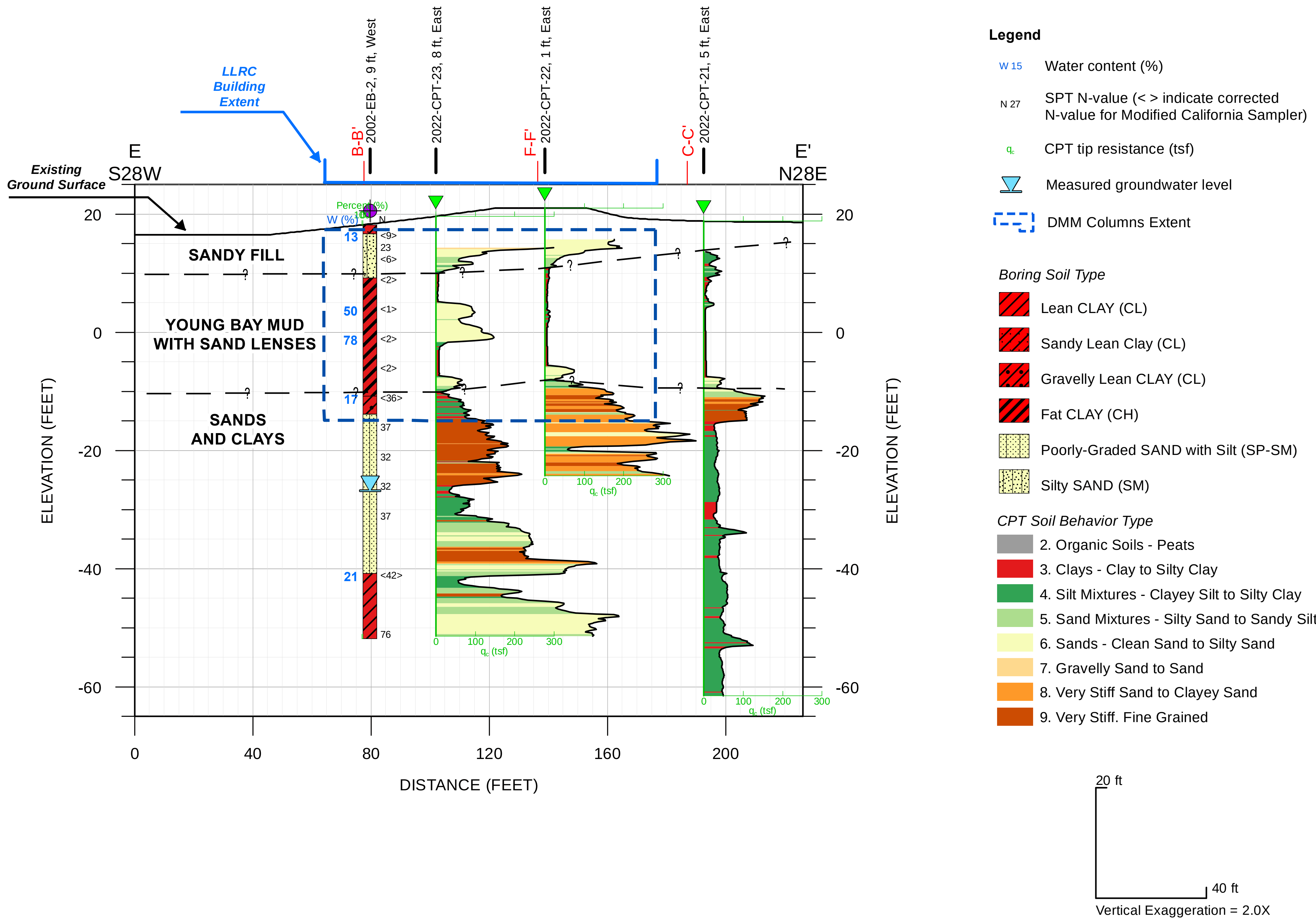

Plate 2e: Cross Section E-E'

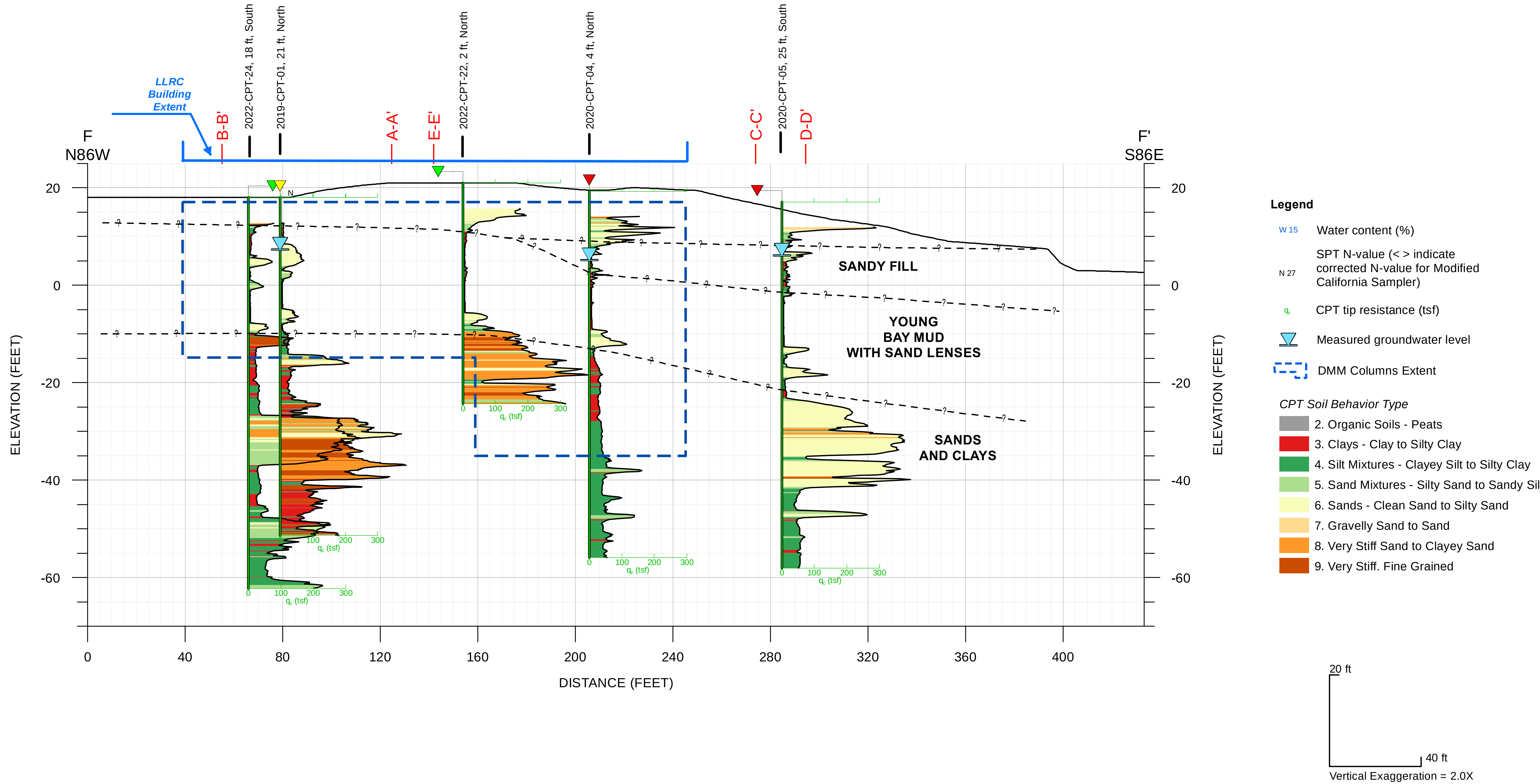

Plate 2f: Cross Section F-F'

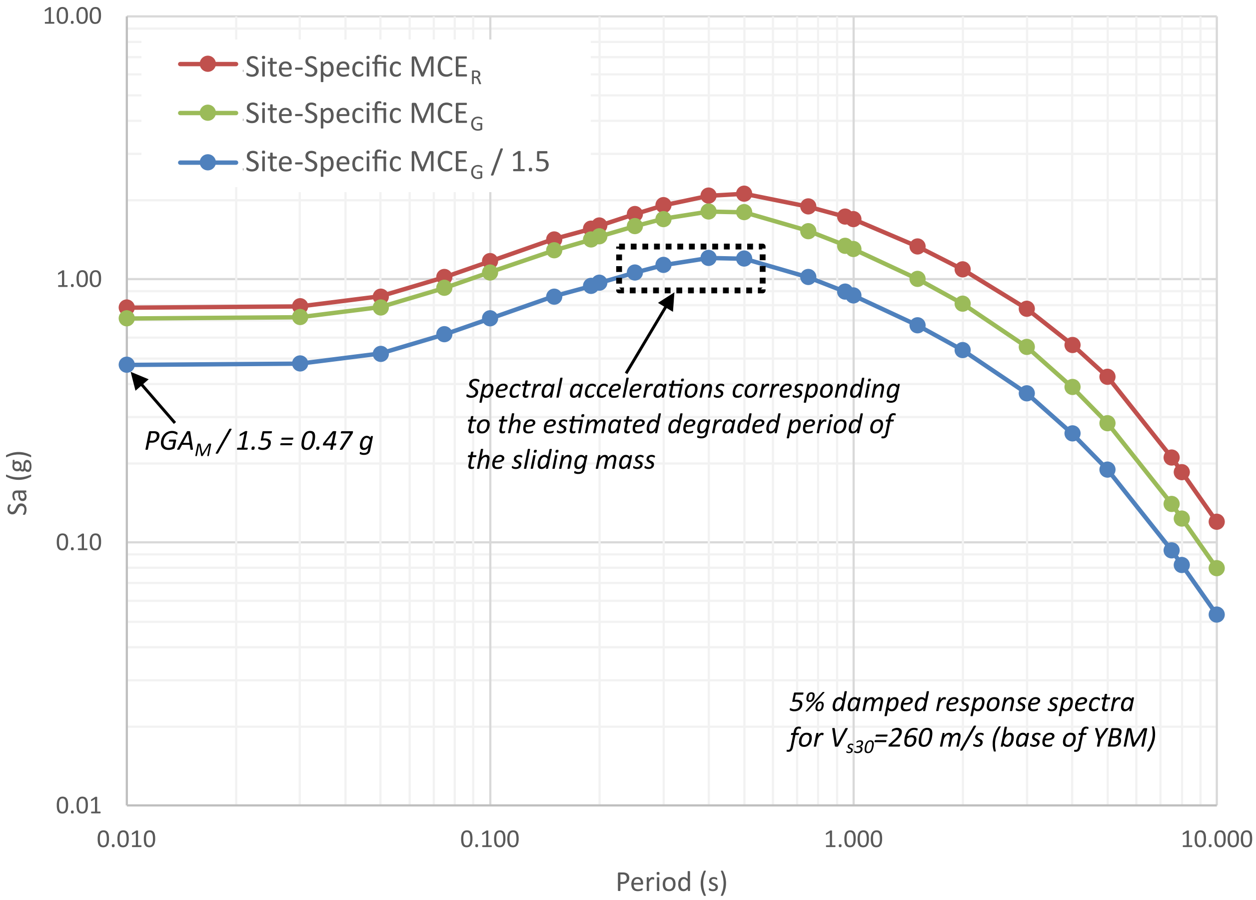

Plate 3: Acceleration Response Spectra at Base of YBM (V_{s30} = 260 m/s)

Title: Laney College Library Learning Resource Center
File Name: Section F_rev03.gsz
Name: DSM_I_b_mp
Horz Seismic Coef.: 0.35
Method: Morgenstern-Price
Date: 05/04/2022

Color	Name	Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Undrained Shear Strength vs Vertical Effective Stress Function	Cohesion' (psf)	Phi' (°)
	DSM Composite	Mohr-Coulomb	120				5,000	0
	Post-Liquefaction Sand (K&W)	SHANSEP	110	0		Kramer and Wang (2015) N160=15		
	Sand and Clay	Mohr-Coulomb	130				0	40
	Sandy Fill	Mohr-Coulomb	120				0	35
	Young Bay Mud (During Earthquake)	SHANSEP	90	200	0.22			

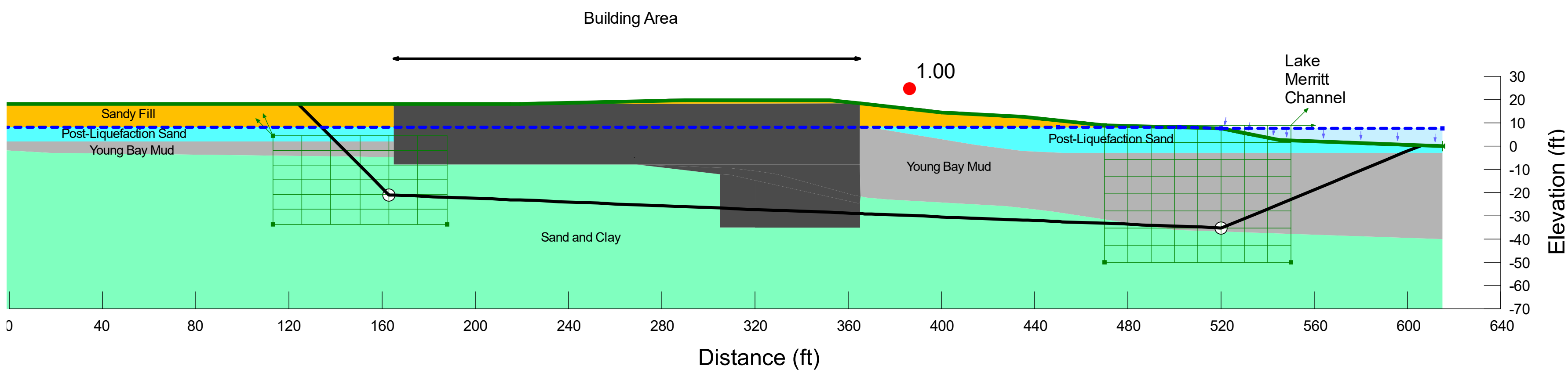

Plate 4a: Pseudostatic Slope Stability Analysis for DMM Ground Improvement for k_h = 0.35 and Block Slip Surface

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File Name: Section F_rev03.gsz
Name: DSM_I_c_mp
Horz Seismic Coef.: 0.35
Method: Morgenstern-Price
Date: 05/04/2022

Color	Name	Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Undrained Shear Strength vs Vertical Effective Stress Function	Cohesion' (psf)	Phi' (°)
	DSM Composite	Mohr-Coulomb	120				5,000	0
	Post-Liquefaction Sand (K&W)	SHANSEP	110	0		Kramer and Wang (2015) N160=15		
	Sand and Clay	Mohr-Coulomb	130				0	40
	Sandy Fill	Mohr-Coulomb	120				0	35
	Young Bay Mud (During Earthquake)	SHANSEP	90	200	0.22			

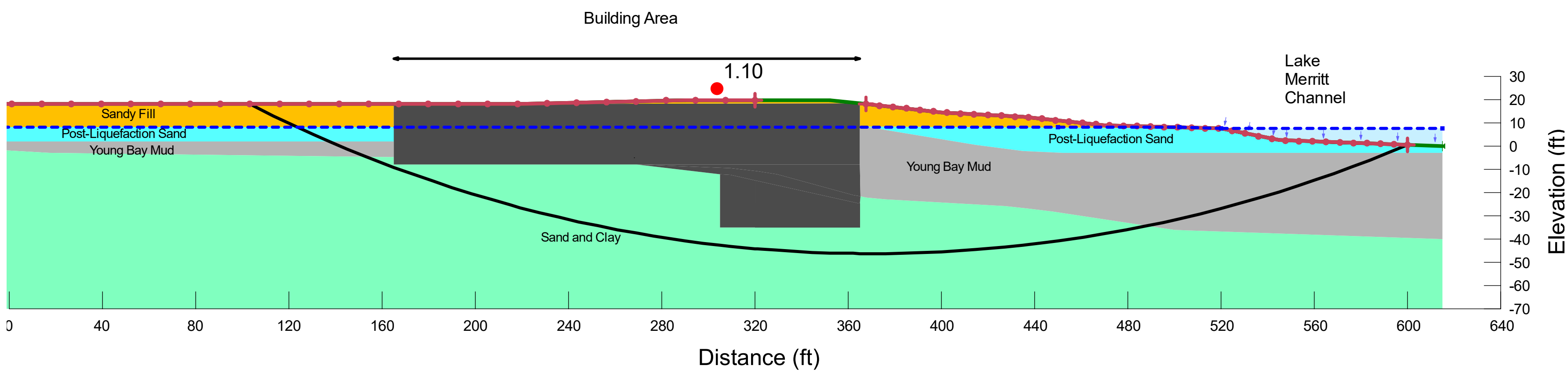


Plate 4b: Pseudostatic Slope Stability Analysis for DMM Ground Improvement for k_h = 0.35 and Circular Slip Surface

Title: Laney College Library Learning Resource Center
File Name: Section F_rev03.gsz
Name: DSM_I_c_mp
Horz Seismic Coef.: 0.35
Method: Morgenstern-Price
Date: 05/04/2022

Color	Name	Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Undrained Shear Strength vs Vertical Effective Stress Function	Cohesion' (psf)	Phi' (°)
	DSM Composite	Mohr-Coulomb	120				5,000	0
	Post-Liquefaction Sand (K&W)	SHANSEP	110	0		Kramer and Wang (2015) N160=15		
	Sand and Clay	Mohr-Coulomb	130				0	40
	Sandy Fill	Mohr-Coulomb	120				0	35
	Young Bay Mud (During Earthquake)	SHANSEP	90	200	0.22			

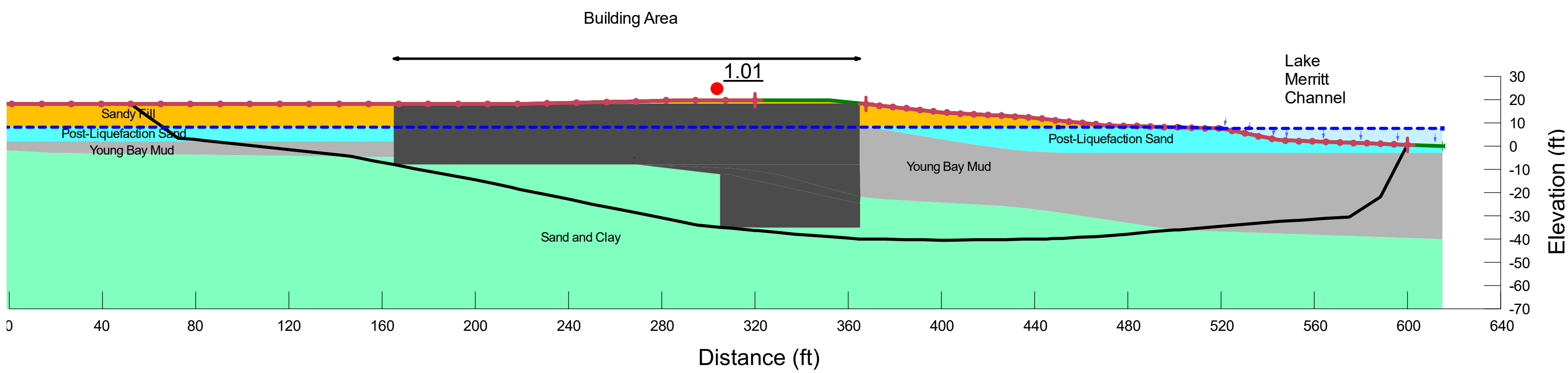


Plate 4c: Pseudostatic Slope Stability Analysis for DMM Ground Improvement for k_h = 0.35 and Optimized Circular Slip Surface

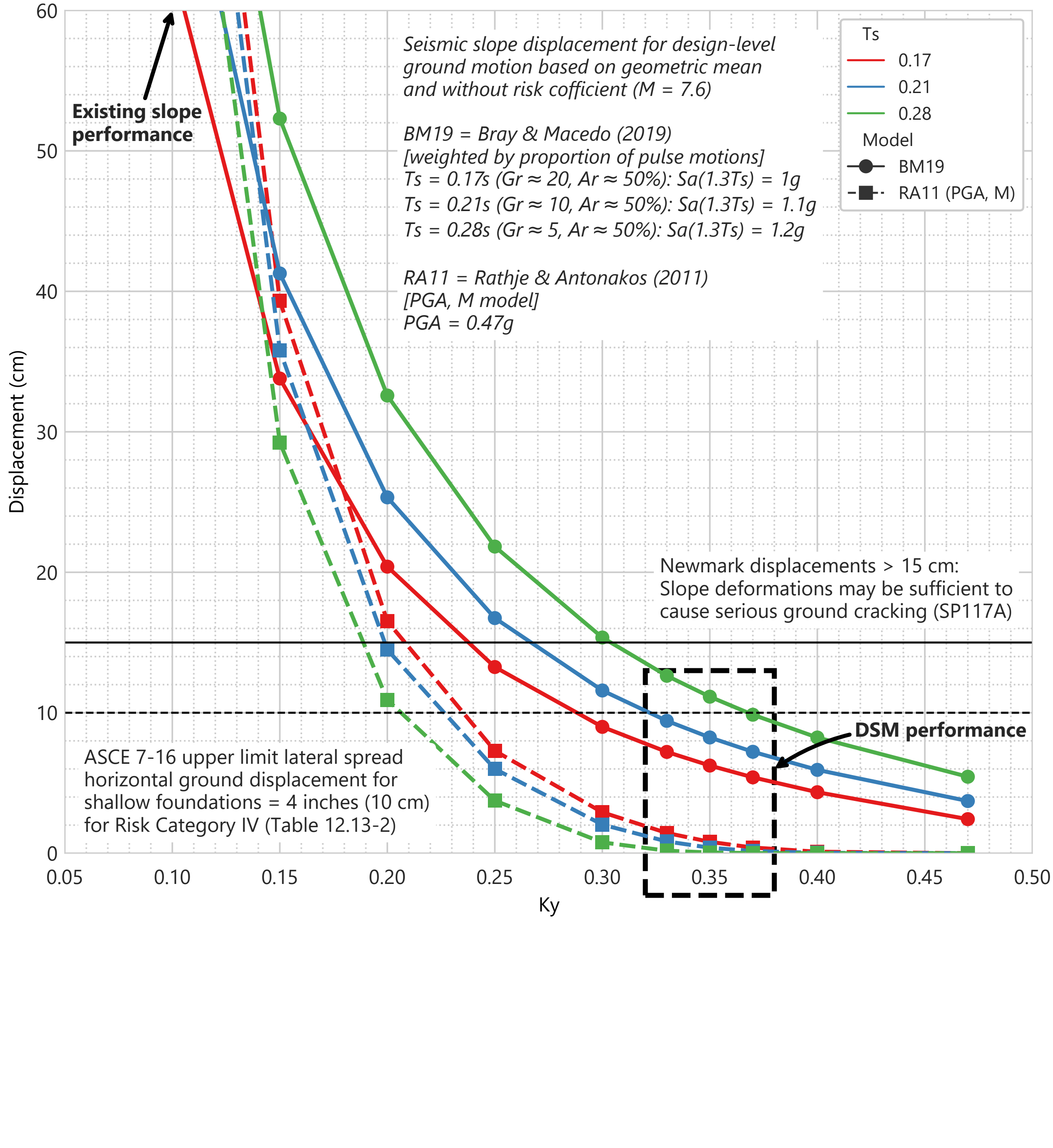


Plate 5: Seismic Slope Displacement vs. Yield Coefficient

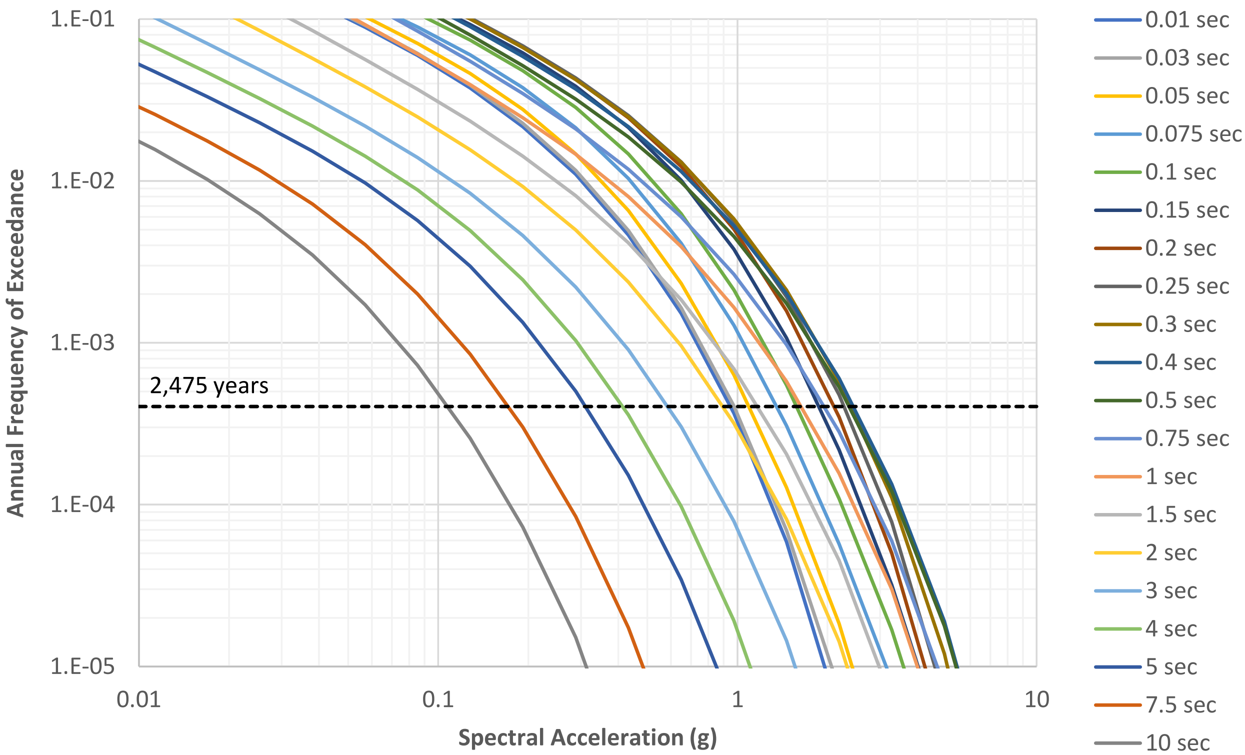


Plate 6: Mean Annual Seismic Hazard Curves for V_{s30} of 270 m/s

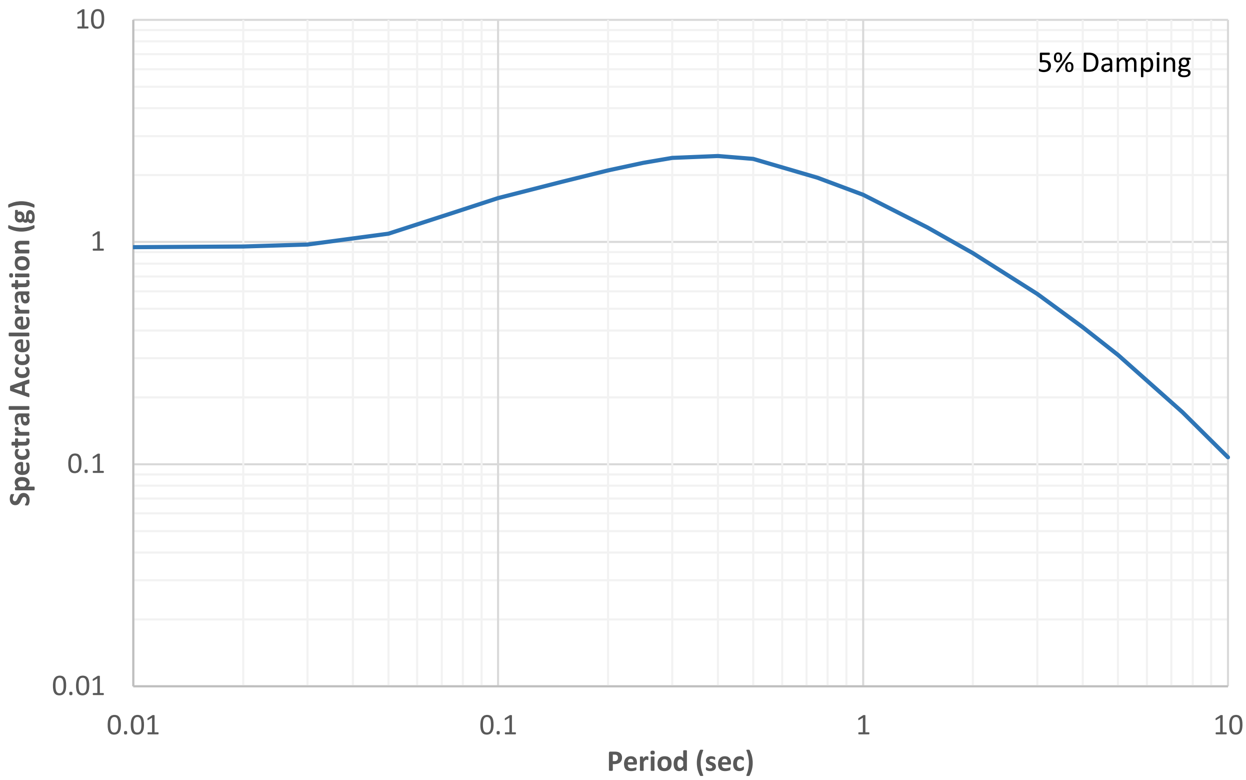


Plate 7: Mean Horizontal Uniform Hazard Response Spectrum for a Return Period of 2,475 Years and V_{s30} of 270 m/s

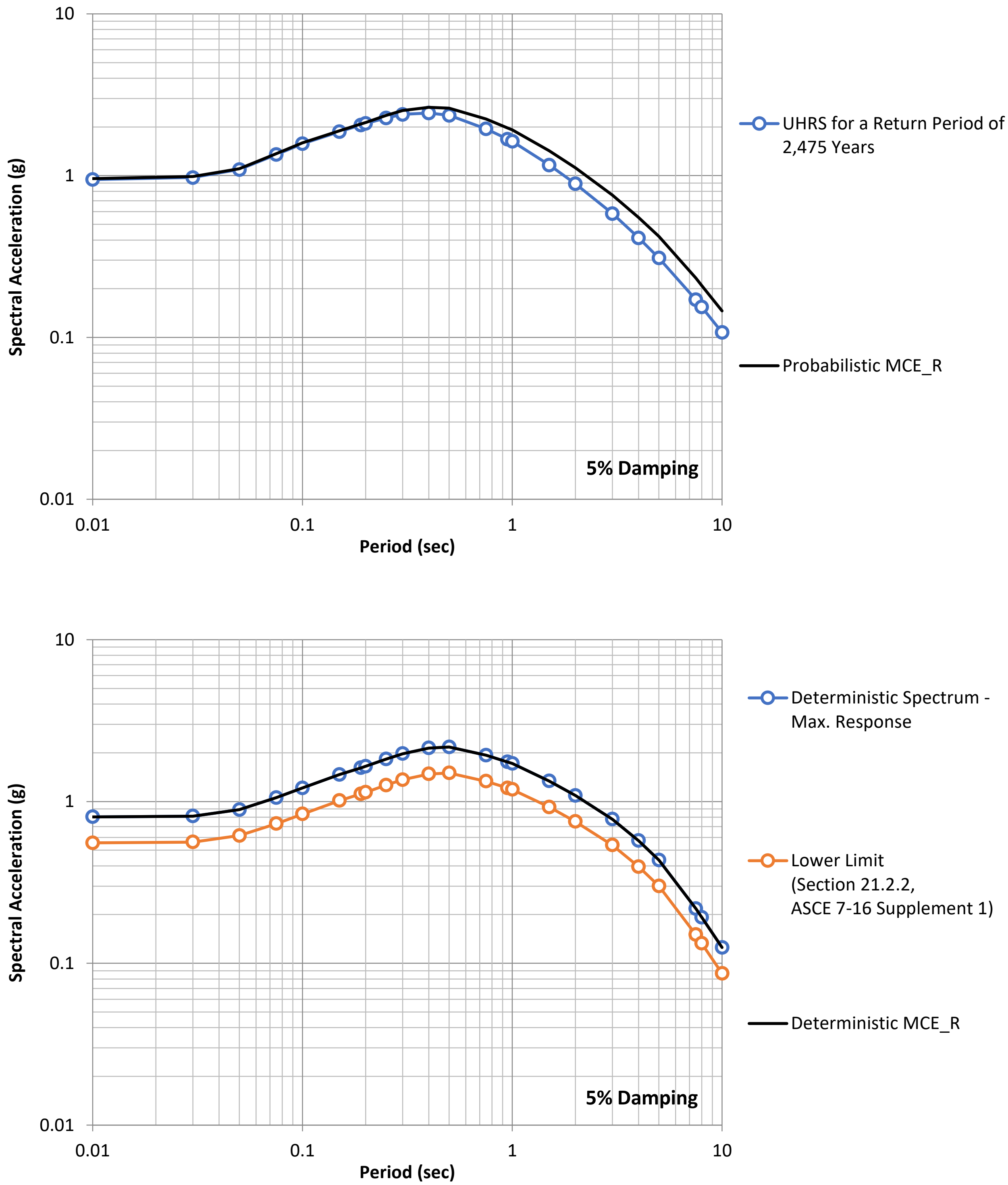
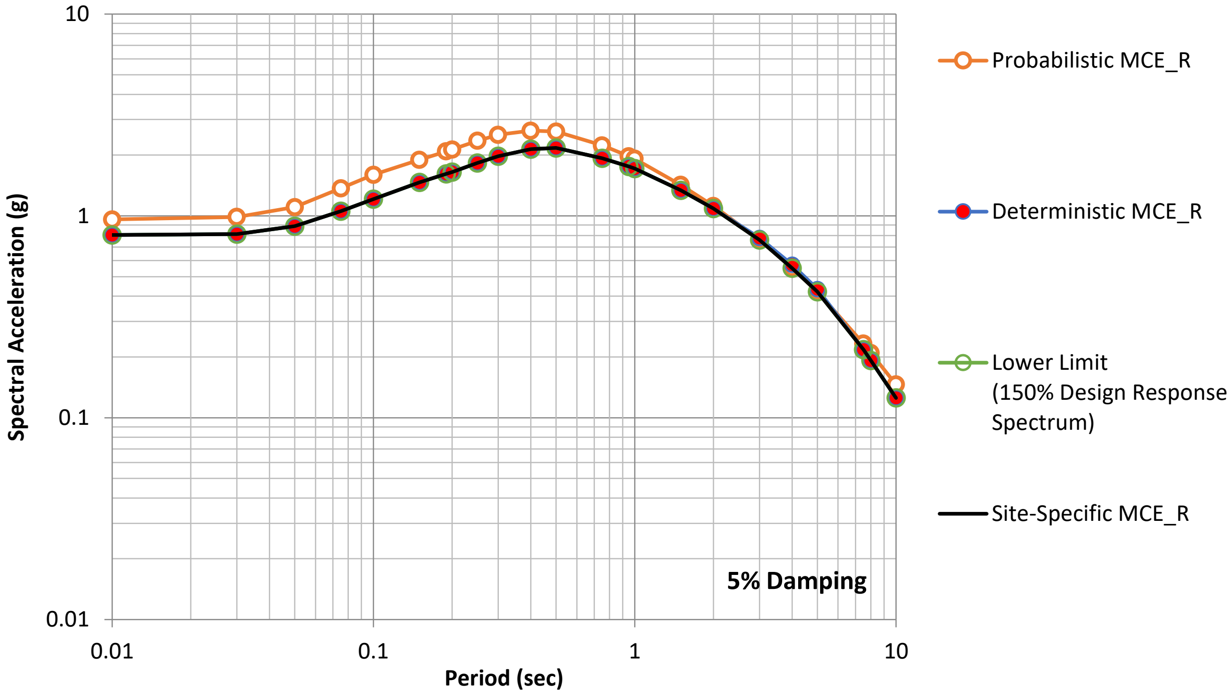


Plate 8: Calculation of the Probabilistic and Deterministic Horizontal MCE_R Response Spectra per ASCE 7-16 for V_{s30} of 270 m/s



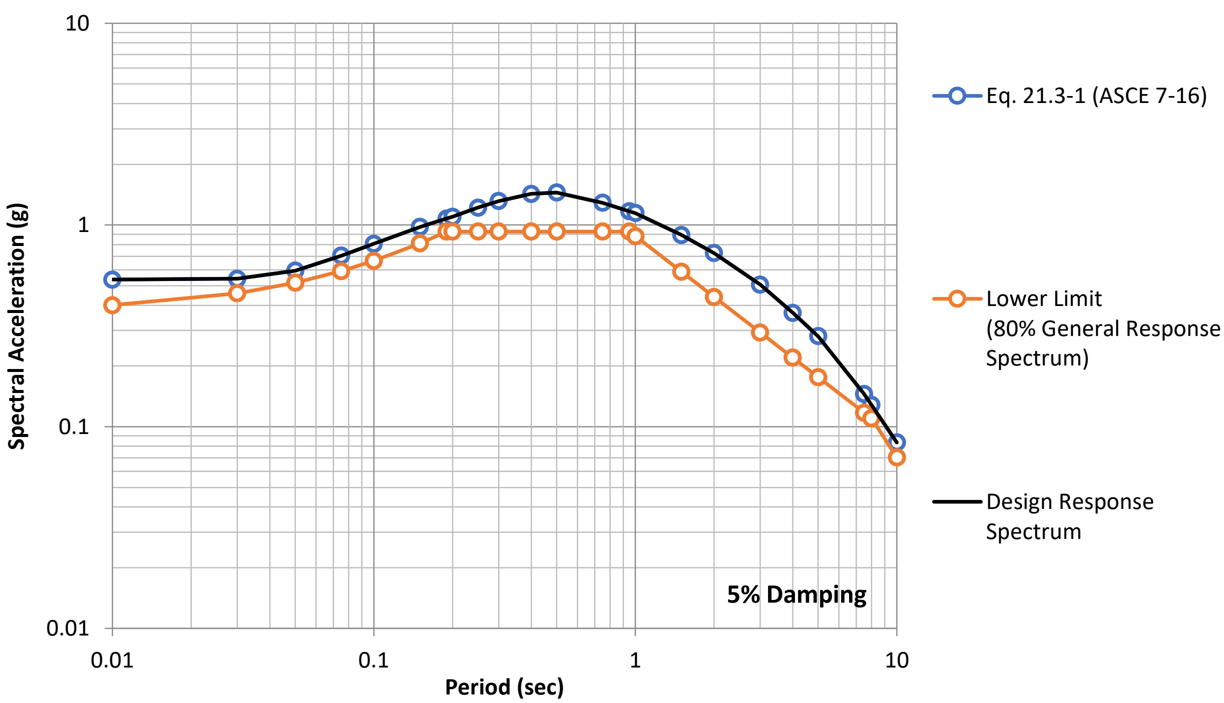


Plate 9: Calculation of the Site-Specific Horizontal MCE_R and Design Response Spectra per ASCE 7-16 for V_{s30} of 270 m/s

Appendix A

Construction Specification for

Deep Mixing

Deep Mixing Method (DMM)

A.1 Part 1 – General

A.1.1 Scope

1. The Deep Mixing Method (DMM) Contractor shall furnish all plant, equipment, labor, and materials required to construct and perform Quality Control of the DMM in accordance with the DMM Design Plans and Specifications.
2. The purpose of DMM ground improvement is to reduce seismically induced slope displacements and settlements to acceptable levels and to provide vertical and lateral support for shallow foundation systems for the Laney College Library & Learning Resource Center (LLRC) building for both static and seismic loadings. The DMM ground improvement consists of continuous underground overlapping deep mixed (DM) columns forming a series of DM walls that are arranged in a grid pattern to form a series of DM walls and blocks. The dimensions and layout of DM grids and blocks are shown on the DMM Design Plans and are described in **Section A.3.2** of this Specification.
3. This specification has been developed as a combination of performance and method specifications. The intent is that the DMM Contractor will select the means and methods for satisfying the acceptance criteria. The DMM Contractor will then demonstrate that the means and methods will satisfy the acceptance criteria using one or more test sections. Once the test section indicates satisfactory results, as determined by the Geotechnical Engineer, the DMM Contractor will follow the means and methods used to satisfy the acceptance criteria for all of the production DMM construction. If the DMM Contractor desires to change the means and methods during the course of production DMM, the changes need to first be approved by the Geotechnical Engineer and CGS. The Geotechnical Engineer may require additional test sections prior to approval of changes in means and methods.
4. The DMM Contractor shall be responsible for performing Quality Control (QC) during DMM construction, which includes QC documentation preparation and submittal, and sample collection, storage, and transportation. Sample testing shall be performed by a DSA approved testing laboratory hired by the Owner to verify that the acceptance criteria are satisfied. The Geotechnical Engineer shall make the determination as to whether the acceptance criteria have been met.
5. Upon completion of DMM installation, an as-built submittal package shall be prepared by the DMM Contractor and the Geotechnical Engineer to document that the installed DMM meets the project performance requirements. The as-built submittal package shall be submitted to CGS for approval. The submittal shall include test section results, daily quality control reports, DMM core and lab test results, as-built DMM record drawings, and any other information needed to document the work.

A.1.2 References

1. American Concrete Institute (ACI)
2. American Society of Testing and Materials (ASTM)
3. American Petroleum Institute (API)

A.1.3 Definitions

1. Area Replacement Ratio (Ar): A ratio of the surface area of soil-cement to the total surface area of ground to be improved within a given Treatment Zone. The total area of each Treatment Zone is measured to the outer tangent lines of the DMM columns along the entire Treatment Zone perimeter.
2. DMM: In situ ground treatment in which soil is blended with cementitious and/or other binder materials to improve strength, permeability, and/or compressibility characteristics (synonym terms include DSM, deep mixing, CDSM, and soil cement mixing).
    a. The DMM grids and blocks are formed by an arrangement of at least two soil mixing shafts with overlapping augers and blades (paddles), guided by a lead mounted on a crawler base machine.
    b. The mixing shafts shall be driven by a power source sufficient to provide torque for the wide range of expected drilling conditions, indicated by the available boring and CPT logs and other test data included in the Geotechnical Investigation Report (GIR) and planned future CPT logs prior to construction.
    c. As the mixing shafts are advanced into the soil, grout is pumped through the hollow stem of the shafts and injected into the soil at the shaft tips. Auger flights and mixing blades on the shafts blend the soil with grout in a pugmill fashion. When the design depth is reached, the mixing shafts are withdrawn while the mixing process is continued.
    d. The mixing shafts are positioned so as to overlap one another to form continuously mixed overlapping columns. After withdrawal, two (or more) overlapping soil-cement columns remain in the ground.
    e. The process is then repeated to form grids and blocks of overlapping DMM columns.
3. DMM Design Addendum (DA): DMM Design Addendum No. 1 prepared by Fugro USA Land, Inc. dated June 10, 2022, and subsequent addenda.
4. DMM Elements: DMM columns will be used to create DMM grids and DMM blocks of treated soil referred to as ground improvement. A DMM grid will consist of interconnected DMM walls formed by partially overlapping columns arranged in a grid pattern with a replacement ratio less than 100 percent. A DMM block used to support a building footing will consist of interconnected DMM walls formed by overlapping columns arranged in a parallel pattern with a replacement ratio of 100 percent or less as shown in project plans and drawings. For this project, individual DMM Element refers to the grouping of columns installed simultaneously during single penetration of the DMM rig.
5. DMM Contractor: The firm performing the DMM construction.

6. DMM Layout Plan: The alternate DMM construction layouts designed by the DMM Contractor, which satisfy the requirements of this Specification. The DMM Layout Plans shall be reviewed and approved by the Geotechnical Engineer and Structural Engineer.
7. Cement Dosage: The amount of cement (in terms of dry weight) used to treat a given initial volume of in-situ soil.
8. Cone Penetrometer Test (CPT): A geotechnical exploration tool, as defined in ASTM D 5778.
9. Core Run: The total length reported by the driller as the actual depth penetrated by coring, including both recovered and unrecovered lengths.
10. Geotechnical Investigation Report (GIR): Geotechnical Investigation Report prepared by Fugro USA Land, Inc. dated February 28, 2020, and subsequent addenda. Note the DMM Design Addendum No. 1 (DA) supersedes the GIR.
11. Geotechnical Engineer: The geotechnical engineer of record responsible for the DMM design, who is hired by the Owner.
12. Ground Improvement Area: The plan area contained within a single perimeter shown on the DMM Design Plans that surrounds:
    a. - All planned soil-cement grids/blocks.
    b. - Unmixed soil within the grids.
13. Grout: A stable colloidal mixture of water, Portland cement, and admixtures. The purpose of the grout is to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in-situ soil.
14. Grout-Soil Ratio: A volumetric ratio of grout to in-situ soil to be mixed.
15. Owner: Peralta Community College District and its representatives.
16. Structural Engineer: The structural engineer of record responsible for designs of structure foundations supported by DMM, who is hired by the Owner.
17. Testing Laboratory: The testing laboratory of record performing construction material testing, which is hired by the Owner and approved by the Geotechnical Engineer. The Testing Laboratory shall be selected from the DSA approved laboratory list.
    a. Treatment Zone: A spatial zone of soil targeted for ground improvement. The vertical and lateral (horizontal) extents of the Treatment Zones are defined on the DMM Design Plans.

A.1.4 Submittals

1. Evidence of conformance to the referenced standards and requirements shall be submitted by the DMM Contractor to the Geotechnical Engineer for the following, but not limited to, in accordance with the requirements in this Specification.
    a. Cement: Certificate of compliance for each truck load delivery.
    b. Admixtures: If used, certificate of compliance for each load or lot of material delivered.
    c. Preliminary Mix Design: Proposed mix designs including all materials and quantities and documentation of calibration of the grout mixing plant.

d. Proposed Test Section Program, Sampling Plan, and Laboratory Testing Program, conforming to the requirements described in this Section.
e. Construction Schedule: Submit a detailed schedule that identifies start dates and duration of each major task in the work. The schedule shall at a minimum include information regarding equipment mobilization, equipment setup, soil-cement mixing test section, production installation, and verification testing.
f. Site Work Plan: Submit a site plan showing staging area for all on-site equipment, including anticipated sections of the streets which may require blocking of parking spaces or traffic clearances.
g. DMM Layout Plans: Submit 1″=20′ scale drawings showing proposed layout of DMM Elements (including test section(s) and production DMM), including column diameters, column overlap, grid sizes, tip elevations, top elevations, coordinates of the corners, foundations, and proposed column and element numbering scheme prior to site mobilization in hard copy and electronic format using the project coordinate system at least 14 calendar days prior to beginning DMM construction. The DMM Contractor must obtain the Geotechnical Engineer's approval of the proposed column layout prior to beginning DMM construction.
h. Equipment and Procedures: Submit a detailed description of the equipment and procedures to be used during all DMM work including, but not limited to, construction of DMM test section(s), production DMM work, and collecting samples for laboratory confirmation testing. Procedures shall include methods for locating the DMM Elements in the field and confirming that the columns are plumb. In addition, while it is recognized that the specific responses to field difficulties are dependent on several factors, the DMM Contractor shall submit their anticipated responses to the following possible situations that could occur during construction and testing of the DMM columns including poor core sample recovery or inability to retrieve core samples, and failing production test results (e.g., repair and/or treatment of failed area and modification to approved procedures or mix design).
i. The DMM Contractor shall also submit the anticipated cement dosages (proportions) to achieve the acceptance criteria outlined under acceptance criteria in **Section A.3.15** of this Specification.
j. Quality Control Program, as outlined in the Execution Section of this Specification.
k. Daily Quality Control Reports: Prior to construction, submit a proposed Daily Quality Control Report format for approval by the Geotechnical Engineer. Submit the Daily Quality Control Report at the end of the next working day. The report should be in conformance with quality control in **Section A.3.14** of this Specification.
l. DMM Test Results: Submit all QC test results as outlined in quality control in **Section A.3.14** of this Specification.

   m. Calibrations: Submit all metering equipment calibration test results including mixing systems, delivery systems, alignment systems, and mixing tool rotational and vertical speed.
   n. Record Drawing: Submit record drawings prepared by the DMM Contractor indicating the as-built location and elevations of the DMM Elements in terms of project coordinates and vertical datum. The record drawings shall also indicate the above structure foundation designs and locations.
2. Upon completion of DMM installation, the as-built submittal by the DMM Contractor to the Geotechnical Engineer, Structural Engineer, and CGS shall include test section results, daily quality control reports, DMM core and lab test results, DMM record drawings, and any other information needed to document the work.

A.2 Part 2 – Products, Materials, and Equipment

A.2.1 Materials

1. Grout: The material added to the blended in situ soils shall be a water-based Portland cement grout. The purposes of the grout are to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in-situ soils. The grout shall be premixed in a mixing plant which combines dry materials and water in predetermined proportions.
2. Cement used in preparing the grout shall conform to ASTM C150 “Standard Specification for Portland Cement Type II”. The cement shall be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter shall not be used.
3. Water: Fresh water, free of deleterious substances that adversely affect the strength and mixing properties of the grout, shall be used to manufacture grout.
4. Admixtures: Admixtures are ingredients in the grout other than Portland cement, and water. Admixtures of softening agents, dispersions, pozzolans, retarders or plugging or bridging agents may be added to the water or the grout to permit efficient use of materials and proper workability of the grout. However, no admixtures shall be used except as approved by the Geotechnical Engineer.

A.2.2 Equipment

The DMM equipment shall meet the following requirements:

1. The mixing tools shall have mixing augers and blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and grout.
   a. Multi-shaft mixing equipment (machines with at least two soil mixing shafts with overlapping augers and blades) shall be used.

    i. The mixing augers and blades shall be minimum 3 feet and maximum of 6 feet in diameter.
    ii. Allowable wear to mixing augers and blades will be limited such that equipment produces a column no less than the design diameter listed on the DMM Layout Plans.
    iii. The overlapping between any two adjacent DMM columns shall be at least 30 percent of the column diameter.
  b. The power source for driving the mixing shafts shall:
    i. be sufficient to provide torque for the wide range of expected drilling conditions, indicated by available boring and CPT logs and other test data included in the Geotechnical Investigation Report (GIR) and DMM Design Addendum (DA).
    ii. be sufficient to maintain the required revolutions per minute (RPM) and penetration rate from a stopped position at the maximum depth required.
2. The DMM rig shall be equipped with electronic sensors built into the leads to determine vertical alignment in two directions: fore-aft and left-right.
  a. The sensors shall be calibrated at the beginning of the project and the calibration data shall be provided to the Geotechnical Engineer. The calibration shall be repeated at intervals not to exceed three months per rig.
  b. The output from the sensors shall be routed to a console that is visible to the operator and the Geotechnical Engineer during penetration and reported. The console shall be capable of indicating the alignment angle in each plane.
3. The DMM equipment shall be adequately marked to allow the Geotechnical Engineer to confirm the penetration depth to within 6 inches during construction.
4. The grout shall be premixed in an on-site mixing plant, using a batch process, which combines dry materials and water in predetermined proportions. The mixing plant shall consist of a grout mixer, grout agitator, grout pump, batching scales, and a computer control unit.
  a. Dry materials shall be stored in silos. The dry materials shall be transported to the project site and blown into the on-site storage tanks using a pneumatic system.
  b. The air evacuated from the storage tanks during the loading process shall be filtered before being discharged to the atmosphere.
  c. Automatic batch scales shall be used to accurately determine mix proportions for water and cement during grout preparation.
  d. The dry admixtures, if used for mixing with water and cement, can be delivered to the mixing plant by calibrated auger. However, the DMM Contractor shall demonstrate that the calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

    e. Calibration of mixing components shall be done at the beginning of the project and repeated at intervals not to exceed three months thereafter and after each move of the batch plant.
5. Positive displacement pumps shall be used to transfer the grout from the mixing plant to the mixing tool head. The grout shall be delivered to each slurry-injecting tool head by an individual positive displacement pump.
6. The DMM rig shall be equipped with sensors to continuously monitor and record the mixing tool penetration/withdrawal speed, mixing tool rotation speed, and injection rate.
    a. The output from these sensors shall be visible to the Operator and Geotechnical Engineer during penetration and withdrawal.
    b. The DMM Contractor may propose alternative display/monitoring systems; however, the systems shall first be reviewed and approved by the Geotechnical Engineer prior to use.
    c. Calibration of this equipment shall be performed at the beginning of the project and the calibration data shall be provided to the Geotechnical Engineer. The calibration shall be repeated at intervals not to exceed three months.

A.2.3 Products

1. DMM: The in-place grout mix together with the soils shall meet all of the acceptance criteria specified in **Section A.3.15** of this Specification, determined according to the quality control, sampling, and testing methods specified in **Section A.3.14** of this Specification.

A.3 Part 3 – Execution

A.3.1 Observation of Work

1. The work covered by these specifications shall be performed under the observation of the Geotechnical Engineer, who shall be retained and paid by the Owner. The Geotechnical Engineer will be present at the site during the conduct of work to observe the work, and to perform field and laboratory tests, as deemed necessary by the Owner. The DMM Contractor shall cooperate with the Geotechnical Engineer in performing the observations and tests. At the completion of their work, the Geotechnical Engineer shall submit a report to the Owner, including a tabulation of all tests performed. The Geotechnical Engineer's costs for observing the construction, testing, and the repair of unsatisfactory work performed by the DMM Contractor shall be billed to the Owner. The Owner shall pay them and then shall deduct the amount from monies due to the DMM Contractor.
2. This work falls under the jurisdiction of the California Division of State Architect (DSA) who will review submittals and may observe portions of the work.

A.3.2 General

1. The soil-mixing shall be constructed by the DMM Contractor to the lines, grades, and cross sections indicated on the DMM Design Plans example layouts or an alternate layout approved by the Geotechnical Engineer, Structural Engineer, and CGS. Revisions to the approved layouts shall be submitted to Geotechnical Engineer of Record (GEOR) for review and approval. DMM ground improvement within a single structure shall be arranged in an uninterrupted grid that follows the structure column lines and underlies all footings and moment frame grade beams, tie beams, and shear walls as shown on the DMM Design Plans.
2. As shown on the DMM Design Plans, the DMM shall underlie the entire structure footprints and extend laterally to include any attached structures which are deemed to be essential parts of the structures.
3. Grading after the site demolition may be required to provide suitable level ground for constructing the DMM. The DMM contractor is responsible for coordinating with the site grading operation to define the Drill-Through Zone.
4. The minimum Area Replacement Ratio (Ar) for DMM grids and blocks and the maximum spacing for DMM grids depends on the specified unconfined compressive strength ($q_{dm,spec}$) as shown in **Table A.1**. Additional DMM Elements may be added within the untreated area to meet or reduce the slab free span distance as instructed by the Geotechnical and Structural Engineer.

Table A.1: Minimum DMM A_r and Maximum DMM Grid Spacing

Specified Unconfined Compression Strength, $q_{dm,spec}$ (psi)	Minimum A_r (%)	Zone B Maximum DMM Center-to-Center Grid Spacing[1]	Zones A1 and A2 Maximum DMM Center-to-Center Grid Spacing[1]
125	50	3.2d	4.0d

[1]d = DMM column diameter

5. DMM elements shall extend to at least the elevations indicated on the DMM Design Plans based on the penetration of the shortest mixing shafts.
6. The top of the DMM shall extend to the base of the ground floor slab section, the bottom of footings, the bottom of moment frame grade beams, and the bottom of elevator pits, as indicated in the DMM Design Plans.
7. Any proposed plan and Area Replacement Ratio by the DMM contractor should be approved by the Geotechnical Engineer and CGS.
8. Elevator pits shall be supported entirely by DMM.

9. The DMM columns shall be essentially vertical columns as stated in this Specification, with a minimum diameter of 3 feet and a maximum diameter of 6 feet and shall extend from the top to the bottom of the Treatment Zone indicated on the DMM Design Plans.
10. The overlapping between any two adjacent columns at ground surface shall be a minimum of 30 percent of column diameter.
11. The completed DMM shall be a homogeneous mixture of grout and the in-situ soils. Mixing is to be controlled by shaft rotational speed, drilling speed, and grout injection rate.
12. Monitoring of construction parameters and confirmation testing will be used to verify that the acceptance criteria have been satisfied.
    a. The DMM Contractor shall establish consistent procedures to be employed during DMM construction to ensure a relatively uniform product is created.
    b. These procedures are to be defined in the equipment and procedures submittal as defined in **Section A.1.4** of this Specification and subsequently modified, if necessary, based on the results of the pre-production testing or quality control testing.
13. The DMM Contractor may request that the established grout mix/grout-soil ratio design, equipment, installation procedure, or test methods be modified. However, the Geotechnical Engineer may require additional testing, at no additional cost to the Owner, to verify that acceptable results can be achieved.
    a. The DMM Contractor shall not employ modified grout mix/grout-soil ratio design, equipment, installation procedures, or sampling or testing methods until approved by the Geotechnical Engineer in writing.
    b. The Geotechnical Engineer, at his sole discretion, may reject any modification proposed by the DMM Contractor.

A.3.3 Construction Site Survey

The location of both active and abandoned buried utilities at the site can have significant impact on the design and construction of deep mixing works. Careful consideration of the presence and location of all utilities is required.

1. Prior to bidding, the contractor should review the available subsurface information and visit the site to assess the site geometry, equipment access conditions, location of existing structures, and above-ground utilities and facilities.
2. The contractor should field locate and verify the locations of all utilities prior to starting work. The contractor should maintain uninterrupted service for those utilities designated to remain in service throughout the work. The contractor should notify the engineer of any utility locations different from those shown in the plans that may require relocation of deep mixed elements or structure design modification. Subject to owner’s geotechnical engineer’s approval, the contractor should be compensated for additional costs of element relocation and/or structure design modifications resulting from utility locations different from those shown in the plans.

A.3.4 Site Access for Soil Samples

1. After award of the Contract, the DMM Contractor will have the option of accessing the jobsite to collect additional soil samples for use in mix designs with the following requirements:
   a. Prior to commencing with field work, the DMM Contractor shall obtain all necessary permits for sampling activities, including drilling permits from Alameda County Public Work Agency, if applicable.
   b. The DMM Contractor shall submit to the Geotechnical Engineer a sampling plan indicating in detail the sampling activities proposed, and the proposed methods for backfilling boreholes or excavations and restoring the site.
   c. Cement grout backfill for boreholes per Alameda County Public Work Agency is required.
   d. The soil sampling and testing will be performed by DMM Contractor. The costs of additional soil sampling and testing (if performed) are to be included in the project DMM construction costs.

A.3.5 Test Sections

1. The DMM Contractor shall construct a minimum of one test section on site to demonstrate. that the proposed mix design, equipment, and procedures will meet the specified requirements. The location(s) of the test section(s) shall be determined by the DMM Contractor with the approval of the Geotechnical Engineer.
2. Additional test sections may be performed at the DMM Contractor's option to optimize the mix design and procedures.
3. Each test section must extend at least to the deepest DMM design depth as indicated by the DMM Design Plans.
4. The costs of the test section(s) are to be included in the project DMM construction costs.
5. Each test section shall consist of at least two full strokes of the DMM equipment. For example, if the DMM rig uses three augers, then the test section shall consist of 2 strokes times 3 columns equal 6 columns.
6. Test sections shall not be located directly below proposed footings, moment frame grade beams, and elevator pits. However, the test sections may be constructed in place of other production DMM columns, provided it is later demonstrated that the test sections meet all acceptance criteria. If the test sections are found to fail the acceptance criteria, the DMM Contractor shall make necessary repairs or replace the DMM Elements to the written satisfaction of the Geotechnical Engineer and CGS.
7. During the time interval between construction of the test section(s) and the completion of laboratory test results, the DMM Contractor may proceed with production DMM installation at their own risk. Any production DMM found to fail the acceptance criteria must be

repaired at the DMM Contractor's expense, to the written satisfaction of the Geotechnical Engineer and CGS.

8. A minimum of two (2) full-depth cores shall be obtained from each test section, according to the procedures detailed in this Specification.
9. Laboratory tests, as specified in this Specification, shall be performed on a minimum of ten samples per full-depth core or a minimum of one sample per core run, whichever is greater, from each test section, as selected by the Geotechnical Engineer. Additional cores may be performed to retrieve enough test samples.

A.3.6 Horizontal Alignment

1. The DMM Contractor shall accurately stake the location of DMM Elements using a surveyor before beginning installation. The main survey control for a given area shall be established by a California licensed surveyor; layout of individual DMM Elements does not require a licensed surveyor. Horizontal alignment of DMM columns shall conform to the geometric tolerances in the acceptance criteria of this Specification.
2. The DMM Contractor shall provide an adequate method to allow the Geotechnical Engineer to verify the as-built location of the DMM during construction.
3. Movement of the crawler base machine shall provide the preliminary alignment of the augers and the final alignment shall be adjusted by hydraulic manipulation of the leads.
4. One stroke of the machine shall construct a DMM Element consisting of at least two overlapping columns.
5. The DMM shall be advanced stepwise by overlapping the adjacent columns of the previous strokes.
6. Following DMM construction, the DMM Contractor shall submit as-built drawings indicating the location of the DMM elements in terms of project coordinates and elevation datum.
7. The DMM contractor should provide a construction plan at least two (2) weeks prior to the start of construction that includes the plan showing the numbering and location of the DMM columns, tip elevations or depths, and cut-off (top) elevations. The daily work plan should be provided to the Geotechnical Engineer at the beginning of workday and work progress should be checked and confirmed by the Geotechnical Engineer during and at the end of each day. The DMM contractor should provide a summary progress report to the Geotechnical Engineer at the end of each workday.
8. The location of known obstructions or utilities at or near the treatment area should be marked on the project drawings and on the ground before construction begins. Existing obstructions within the treatment zone area should be removed prior to construction. It is not anticipated that drilling obstructions will be encountered within the Treatment Zone during DMM construction unless further site investigation reveals otherwise.
    a. If an obstruction preventing drilling advancement is encountered, the DMM Contractor shall investigate the location and extent of the obstruction using methods approved by

the Geotechnical Engineer. The DMM Contractor shall propose remedial measures to clear the obstruction for approval by the Geotechnical Engineer.

b. While the investigation for an obstruction is underway, the DMM Contractor shall continue to install columns in areas away from the obstruction location. No stand-by delay will be allowed for equipment and operations during the investigation of an obstruction.

c. The DMM Contractor will be compensated for removal or clearing of obstructions as a Changed Condition, paid in accordance with the General Conditions.

d. The DMM Contractor will not be compensated for removal or clearing of obstructions without prior approval by the Geotechnical Engineer and the Owner.

9. The DMM Contractor will not be compensated for DMM Elements that are located outside of the tolerances specified in the acceptance criteria.

A.3.7 Vertical Alignment

1. The equipment operator shall control vertical alignment of the auger stroke. Verticality shall be monitored with respect to two orthogonal horizontal axes. Vertical alignment of DMM columns shall conform to the geometric tolerances in the acceptance criteria of this Specification.

A.3.8 DMM Depth

1. DMM depths shall extend to the line and grades shown on the DMM Design Plans.
2. The total depth of penetration shall be measured either by observing the length of the mixing shaft inserted below a reference point on the mast, or by subtraction of the exposed length of shaft above the reference point from the total shaft length.
   a. For each stroke, the elevation of the reference point on the mast must be established within one inch using measurements from a surveyed control point.
   b. The final depth and bottom elevation of the stroke shall be noted and recorded on the Daily Quality Control Report by the DMM Contractor. The equipment shall be adequately marked to allow the Geotechnical Engineer to confirm the penetration depth during construction.
3. If rigs with varying mixing shaft lengths are used, the shortest shafts shall extend to the minimum DMM depths indicated on the DMM Design Plans.

A.3.9 Grout Preparation

1. Dry material shall be stored in silos and fed to mixers for agitation and shearing. In order to accurately control the mixing ratio of grout, the addition of water and cement shall be determined by weight using the automatic batch scales in the mixing plant.
   a. The admixtures, if used, for mixing with water and cement, can be delivered to the mixing plant by calibrated auger. However, the DMM Contractor shall prove that the

calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

2. A minimum mixing time of one minute and a maximum holding time of four hours will be enforced for the grout.
   a. The grout hold time shall be calculated from the beginning of the initial mixing.
3. The specific gravity of the grout shall be determined during the design mix program for double checking grout proportions.
   a. The specific gravity of the grout shall be checked by the DMM Contractor at least twice per shift per rig using the methods outlined in ASTM D4380.
   b. The specific gravity of the grout measured in the field should not deviate by more than 3 percent of the calculated specific gravity for the design cement ratio.
   c. If the specific gravity is lower than that required by the design mix, the DMM Contractor shall add additional cement and remix and retest the grout at no cost or schedule impact to the Owner.
   d. The specific gravity measurements shall be indicated on the Daily Quality Control Report.

A.3.10 Soil-Grout Mixing

1. Installation of each column shall be continuous without interruption.
   a. If an interruption of more than one hour occurs, the column shall be remixed (while injecting grout at the design grout ratio) for the entire height of the element at no additional cost to the Owner.
   b. If an interruption of more than ten minutes occurs, the DMM Contractor shall inject a volume of grout equal to that required for three feet of auger penetration, while maintaining constant auger elevation. Once the specified volume of grout has been injected, auger penetration may continue.
2. The completed CSDM shall be a uniform mixture of cement grout and the in-situ soils.
   a. Soil and grout shall be mixed together in place by the specially designed overlapping augers or blades on the mixing shafts.
   b. The grout shall be pumped through the mixing shafts and injected from the tip of the shafts. The shafts shall break up the soil and blend it with cement grout.
   c. The mixing action of the shafts shall blend, circulate, and knead the soil over the length of the column while mixing it in place with the grout.

A.3.11 Shaft Rotational Speed and Penetration/Withdrawal Rate

1. The mixing shaft rotational speed (measured in RPMs) and penetration/withdrawal rates shall be established before beginning work. It may be adjusted with the approval of the Geotechnical Engineer to achieve adequate mixing.

2. The contractor shall obtain the suitable shaft rotational speed during the installation of test section. The rotational speeds and penetration/ withdrawal rates shall be recorded on the Daily Quality Control Report.
3. The established rotational speeds and penetration/withdrawal rates shall be used during the work. If these parameters are varied more than ten (10) percent from those determined during the test section(s), the Geotechnical Engineer may require additional testing, at no additional cost to the Owner, to verify that the acceptance criteria are met.
4. The DMM Contractor may request that the established mixing parameters be modified during the production DMM installation. To verify acceptable results for the modified parameters, the Geotechnical Engineer may require additional testing at no additional cost to the Owner.

A.3.12 Grout Injection Rate

1. The grout injection rate per no more than three vertical feet of column shall be in accordance with the requirements of the design mix.
   a. The required mix design and grout-soil ratio shall be determined during the test section(s).
   b. The grout injection rate shall be constantly monitored and controlled.
   c. The DMM Contractor shall record the volume of grout injected continuously for each column on the Daily Quality Control Report.
2. If the volume of grout injected per three vertical feet of column is less than the amount required to meet the grout-soil ratio established during the test section, the DMM columns shall be remixed and additional grout injected (at the design grout-soil ratio) to a depth at least 3 feet below the deficient zone or until design depth is met, at no additional cost to the Owner.
3. The DMM Contractor may request that the established grout-soil ratio be modified during the production DMM installation.
   a. To verify acceptable results for the modified grout-soil ratio, the Geotechnical Engineer may require additional testing or a new test section at no additional cost to the Owner.

A.3.13 Control of Spoils

1. The DMM Contractor shall control and process all spoils created during the DMM construction.
   a. Prior to stockpiling materials greater than 10 feet in height, stockpile locations and heights shall be submitted for review and approval to Geotechnical Engineer. The DMM Contractor shall consider the locations of and avoid damage to existing utilities, structures, and other improvements, as well as recently constructed DMM Elements when stockpiling material. Lesser stockpile heights may be necessary in some areas.

   b. The spoils shall be processed until they have cured to a sufficient level to allow them to be stockpiled such that they will not reform a cemented mass in the stockpile. The DMM Contractor shall dispose of spoils in accordance with all local laws, codes, and ordinances in a manner acceptable to the Owner or coordinate with the project grading contractor for the spoils be reused as fills at the project site.

A.3.14 Quality Control Program

1. The DMM Quality Control Program shall be the responsibility of the DMM Contractor and shall include, as a minimum, the following components:
   a. An approved pre-construction test program on soils obtained from the project site, to establish appropriate design parameters such as cement dosage and water content.
   b. Field monitoring by the DMM Contractor of construction parameters during DMM construction.
   c. Sample collection including full depth continuous coring, sample storage, and sample transportation to the Testing Laboratory.
   d. Reporting of the field monitoring and sampling performed by the DMM Contractor.
   e. Reporting of the core strength testing performed by the Testing Laboratory.
2. Prior to site mobilization, the DMM Contractor shall submit a detailed work plan for the Quality Control Program for review and approval by the Geotechnical Engineer. The work plan shall include, as a minimum:
   a. A description of all installation, monitoring, sampling, and testing procedures to be implemented. The proposed auger penetration and withdrawal rates shall be proposed by the DMM Contractor at this time.
   b. Descriptions of all sampling equipment.
   c. A list of parameters to be monitored.
   d. Tolerances for the parameters monitored.
   e. Names of any subcontractors.
3. The DMM Contractor shall provide all the personnel and equipment necessary to implement the Quality Control Program. Contractor to provide the number of years/projects, project descriptions, and reference list for all cases below:
   a. The DMM Contractor must have at least 7 (seven) years of previous successful experience with at least 5 (five) DMM projects for soil conditions and project scope similar to that of the project being bid.
   b. The DMM contractor must have a registered California Professional Engineer (PE) who have had at least 5 (five) years of experience with at least 3 (three) DMM projects.
   c. The DMM Contractor must have assign a project manager who have had at least 5 (five) years of experience on at least 3 (three) DMM projects.
   d. The DMM Contractor must have assign a project engineer/ supervisor who have had at least 3 (three) years of experience with at least 2 (two) DMM projects.

e. The DMM Contractor must assign a full-time project superintendent with at-least 3 (three) DMM projects with at least 150,000 cubic yard of total treatment volume in DMM construction.
f. The DMM equipment operator must have at least three years of experience with the equipment and DMM construction.
g. Written requests for substitution of these key personnel must be submitted prior to personnel changes. Documentation must be submitted to the owner that demonstrates that the substitute meets the requirements listed. Substitution may not be made until written approval is provided by the owner.

4. The Geotechnical Engineer will continuously observe the DMM construction. The Geotechnical Engineer will review DMM Contractor submittals to check that the Quality Control Program is being properly implemented.
5. The established quality control procedures shall be maintained throughout the production DMM installation to ensure consistency in the installation and to verify that the work complies with all requirements indicated in the DMM Design Plans and Specifications, unless modifications to the procedures are approved in writing by the Geotechnical Engineer.
6. DMM Contractor shall perform sample collection, storage, and transportation.
   a. DMM Contractor shall collect one full-depth continuous coring should be made for every 3% of the total DMM elements or for every 900 square feet of treated ground, whichever produces the greater number of cores at locations specified by the Geotechnical Engineer.
      i. The coring rig shall be a triple-barrel rig approved by the Geotechnical Engineer and capable of achieving the required recovery. The ability to achieve the recovery criteria is solely the Contractor’s responsibility.
      ii. Full-depth samples obtained by the DMM Contractor shall have a diameter of at least 3 inches.
      iii. The continuous core sample shall extend from the top through the bottom of the Treatment Zone, and to at least 5 feet below the Treatment Zone to sample the foundation soil directly below the Treatment Zone.
      iv. Unless otherwise directed by the Geotechnical Engineer, the full-depth samples shall be obtained along an essentially vertical alignment located one-fourth of a column diameter from the column center and not within column overlaps.
      v. The DMM Contractor shall perform all full-depth sampling in the presence of the Geotechnical Engineer.
      vi. Full-depth core samples shall be retrieved using triple tube continuous coring techniques after the soil-grout mixture has hardened sufficiently.
      vii. Each core run shall be a minimum 4 feet in length.

viii. Following logging, the engineer will select at least five specimens from each full-depth continuous core for strength testing. Each test specimen should have a length-to-diameter ratio of 2 or greater.

ix. A minimum recovery of 85 percent for each 4-foot core run shall be achieved for cores from within the Treatment Zones. During coring, the elevation of the bottom of the holes shall be measured after each core run in order that the core recovery for each run can be calculated.

x. The DMM Contractor shall determine the time interval between column installation and coring except that the interval shall be no longer than required to conduct 28-day strength testing.

xi. The DMM contractor should photograph each core run and submit to the Geotechnical Engineer for test sample selection.

xii. Upon retrieval, the core runs shall be provided to the Geotechnical Engineer for logging, uniformity inspection, and test specimen selection.

xiii. Following logging and test specimen selection by the Geotechnical Engineer, the entire full-depth core, including the designated test specimens, shall be immediately sealed in plastic wrap to prevent drying and transported to the laboratory by the DMM Contractor. Alternatively, the DMM Contractor may transport only the selected test specimens to the laboratory and store the remaining core on-site in a humidity and temperature-controlled storage facility as described in this Specification.

xiv. All core holes shall be filled with cement grout that will obtain a 28-day strength equal to or greater than the strength of the DMM. However, the Contractor shall not grout the core holes until after acceptable core recover and uniformity has been confirmed by the Geotechnical Engineer.

xv. The DMM Contractor shall notify the Geotechnical Engineer at least one business day (24 hours) in advance of beginning core sampling operations.

b. In addition to coring, the DMM Contractor should obtain wet grab samples from the DMM elements at the presence of Geotechnical engineer.

i. 3 (three) wet samples from each mixed design used in each test section as directed by the geotechnical engineer.

ii. One wet sample (i.e., one selected depth at one location) should be retrieved every 2 (two) production days or for every 2,500 cubic yards of treated soil, whichever produces the higher sampling frequency.

iii. The contractor proposes locations for wet sampling as outlined in the QC program, considering input from the geotechnical engineer based on subsurface conditions, DMM layout, review of the QC results, and observation of the soil mixing operation.

iv. The contractor should report all attempts, successful and unsuccessful, to obtain wet samples. Some deep mixed material may not be able to be sampled readily because either the mixture is too stiff or the material may not flow back into the void left after the sampler is extracted, possibly leaving a damaged element.

v. The sampling tool is inserted into the DMM column to a designated depth, filled with treated soil, and lifted to the ground surface. The treated soil material is then poured into a container, screened for oversized lumps (gravel versus unmixed soil), and placed in 3-inch (76-mm)- diameter, 6-inch (152 mm)- long molds. Eight test specimens should be prepared from each wet sample.

vi. The wet treated material should be placed into the mold in three to five layers. After the placement of each layer, the specimens must be tapped or vibrated to remove trapped air bubbles. The specimens should be sealed to prevent moisture from entering or leaving the specimens, and the sealed specimens should be stored in a humid environment in accordance ASTM C192.

vii. For field validation testing, unconfined compressive strength testing may be performed on specimens at 3, 7, 28, and 56 or more days. For full production work, unconfined compressive strength testing may be performed at 3, 7 and 28 days.

viii. The DMM contractor should deliver the samples for testing to a local lab as directed by the geotechnical engineer.

ix. If wet samples produce results that are consistently acceptable, the frequency of wet sampling can be reduced as the project progresses.

x. The engineer may request additional test specimens for QA testing.

c. Untested portions of the full-depth samples shall be retained at the laboratory until completion and acceptance of all DMM, for possible inspection and confirmation testing by the Geotechnical Engineer.

7. The DMM Contractor shall be responsible for handling of test specimens, including storing of untested specimens and transporting test specimens to the Testing Laboratory.
   a. The laboratory testing shall be performed by the DSA accepted Testing Laboratory hired by the Owner and approved by the Geotechnical Engineer.
   b. The samples shall be stored in a moist room as specified in ASTM C 192 until the test date.
   c. Testing for 28-day unconfined compressive strength shall be conducted in accordance with ASTM D2166.
8. In addition to confirmation tests performed by the Testing Laboratory, additional tests may be requested by the Geotechnical Engineer on samples collected by the DMM Contractor. Both the Testing Laboratory's testing and the Geotechnical Engineer's requested additional testing (if performed) shall demonstrate that the acceptance criteria are met prior to acceptance of the work.
9. Daily Quality Control Report
   a. The DMM Contractor shall submit Daily Quality Control Reports to the Geotechnical Engineer at the end of the next working day. The Daily Quality Control Report shall

document the progress of the DMM construction, present the results of the QC parameter monitoring, present the results of the strength testing, and clearly indicate if the columns have met the acceptance criteria. The DMM Contractor shall make all Daily Quality Control Reports available to the Geotechnical Engineer

b. The Daily Quality Control Report shall include as a minimum the results of the following QC parameter monitoring for each column:
    i. Rig number,
    ii. Type of mixing tool,
    iii. Date and time (start and finish) of column construction,
    iv. Column number and reference drawing number,
    v. Column diameter,
    vi. Column top and bottom elevations,
    vii. Grout mix design designation,
    viii. Slurry specific gravity measurements (refer to Section A3.9 for number of tests and tolerance), and
    ix. Description of obstructions, interruptions, or other difficulties during installation and how they were resolved.
c. The Daily Quality Control Reports shall also include the following parameters recorded automatically for each column continuously and submitted in the form of either tables or figures (as agreed to by the Geotechnical Engineer):
    i. Elevation in feet vs. real time,
    ii. Shaft rotation speed in RPMs vs. depth,
    iii. Penetration and withdrawal rates in feet per minute vs. depth,
    iv. Grout injection rate in gpm vs. depth, and
    v. The average quantity of grout in gallons per foot injected per 3-foot (or less) vertical increment of column vs. depth.

A.3.15 Acceptance Criteria

1. The Geotechnical Engineer and CGS shall make the sole determination as to whether the acceptance criteria have been satisfied. The in-place grout-soil mixture comprising the DMM Elements shall meet the following acceptance criteria:
    a. The DMM within the Treatment Zone shall be installed within the following geometric tolerances:
        i. The horizontal alignment of the DMM blocks shall be within 4 inches of the location shown on the approved DMM Layout Plans.
        ii. The vertical inclination of the DMM columns shall be no more than 1: 100 (horizontal to vertical).

   iii. Overlap between any two adjacent columns shall be a minimum of 30 percent of column diameter for the entire depth, as calculated based on depth of column embedment and measured auger lateral and longitudinal inclination.
   iv. The tops of the columns shall be at or higher than the elevations indicated on the DMM Design Plans.
   v. The bottoms of the columns shall extend to or lower than the levels indicated on the DMM Design Plans.

b. Two alternative specified unconfined compressive strengths are provided with corresponding minimum A_r and maximum grid spacing in **Table A.1**. The DMM Contractor shall select one of these options for the entire project.
c. The unconfined compressive strength shall be determined by ASTM D2166 at 28 days on samples taken by coring of the constructed DMM.
d. 80 percent of all unconfined compressive strength testing on core samples determined by ASTM D2166 from each tested deep mixed element shall equal or exceeds the specified strength. If a strength specimen falls below the specified strengths due to an obviously unrepresentative lump of unmixed soil in the specimen, the Geotechnical Engineer has the option to select another specimen from the same core run and allow the Testing Laboratory to test the replacement specimen and substitute the strength from the replacement specimen for the strength from the unrepresentative specimen that failed to satisfy the strength requirement. Only one such retest will be allowed per core run.
e. 90 percent of all the test results on core samples across the site should equal or exceed the specified strength.
f. To prevent a weak layer at one elevation in the DMM foundation system, strengths below the specified strength are not permitted within 10 feet of the same elevation in more than 2 nearby cored elements.
g. Uniformity of mixing within the target zone shall be evaluated by the Geotechnical Engineer based on the full-depth samples recovered by the DMM Contractor from the columns.
   i. Lumps of unimproved soils shall not amount to more than 15 percent of the total volume of any core run from a continuous full-depth core sample. For evaluating the volume of unimproved lumps of soil, all of the unrecovered core length shall be assumed to be unimproved soil.
   ii. Any individual or aggregation of lumps of unimproved soil shall not be larger than 12 inches in greatest dimension.
   iii. Continuous core recovery shall be at least 85 percent over any full-length core.

2. If the acceptance criteria specified in this Specification are not achieved for production DMM, the failed section of DMM shall be rejected.

a. Unless otherwise determined by the Geotechnical Engineer, the failed section of DMM shall be considered to include all DMM columns constructed during all rig shifts that occurred between the times of construction when passing tests were achieved.
b. The DMM Contractor may conduct additional sampling and testing to better define the limits of the failed area at no additional cost to the Owner.
c. The DMM Contractor shall submit a proposed plan for remixing or repair of failed sections for review and approval by the Geotechnical Engineer and CGS.
d. If the treated soil that failed to meet the uniformity criteria is concentrated in a narrow elevation range forming weak planes or zones, the contractor could propose redrilling and remixing to 3 feet below and above the deficient zone. IF redrilling and remixing cannot be done efficiently, the contractor must replace the elements to the full depth. If the treated zone in the narrow elevation meets the uniformity criteria but fails to meet the strength criteria, the contractor could propose to redrill and remix the deficient zone or to assign a lower strength level to the deficient zone and install additional elements to compensate for the strength deficiency.
e. If the treated soil that failed to pass cannot be isolated in a specific zone, the contractor must provide remedial measures for all elements constructed during all rig shifts that occurred between passing elements.
f. Remedial measures are subject to coring and application of the specification acceptance criteria.

Appendix C

Construction Specification for

Deep Mixing

Deep Mixing Method (DMM)

A.1 Part 1 – General

A.1.1 Scope

1. The Deep Mixing Method (DMM) Contractor shall furnish all plant, equipment, labor, and materials required to construct and perform Quality Control of the DMM in accordance with the DMM Design Plans and Specifications.
2. The purpose of DMM ground improvement is to reduce seismically induced slope displacements and settlements to acceptable levels and to provide vertical and lateral support for shallow foundation systems for the Laney College Library & Learning Resource Center (LLRC) building for both static and seismic loadings. The DMM ground improvement consists of continuous underground overlapping deep mixed (DM) columns forming a series of DM walls that are arranged in a grid pattern to form a series of DM walls and blocks. The dimensions and layout of DM grids and blocks are shown on the DMM Design Plans and are described in **Section A.3.2** of this Specification.
3. This specification has been developed as a combination of performance and method specifications. The intent is that the DMM Contractor will select the means and methods for satisfying the acceptance criteria. The DMM Contractor will then demonstrate that the means and methods will satisfy the acceptance criteria using one or more test sections. Once the test section indicates satisfactory results, as determined by the Geotechnical Engineer, the DMM Contractor will follow the means and methods used to satisfy the acceptance criteria for all of the production DMM construction. If the DMM Contractor desires to change the means and methods during the course of production DMM, the changes need to first be approved by the Geotechnical Engineer and CGS. The Geotechnical Engineer may require additional test sections prior to approval of changes in means and methods.
4. The DMM Contractor shall be responsible for performing Quality Control (QC) during DMM construction, which includes QC documentation preparation and submittal, and sample collection, storage, and transportation. Sample testing shall be performed by a DSA approved testing laboratory hired by the Owner to verify that the acceptance criteria are satisfied. The Geotechnical Engineer shall make the determination as to whether the acceptance criteria have been met.
5. Upon completion of DMM installation, an as-built submittal package shall be prepared by the DMM Contractor and the Geotechnical Engineer to document that the installed DMM meets the project performance requirements. The as-built submittal package shall be submitted to CGS for approval. The submittal shall include test section results, daily quality control reports, DMM core and lab test results, as-built DMM record drawings, and any other information needed to document the work.

A.1.2 References

1. American Concrete Institute (ACI)
2. American Society of Testing and Materials (ASTM)
3. American Petroleum Institute (API)

A.1.3 Definitions

1. Area Replacement Ratio (Ar): A ratio of the surface area of soil-cement to the total surface area of ground to be improved within a given Treatment Zone. The total area of each Treatment Zone is measured to the outer tangent lines of the DMM columns along the entire Treatment Zone perimeter.
2. DMM: In situ ground treatment in which soil is blended with cementitious and/or other binder materials to improve strength, permeability, and/or compressibility characteristics (synonym terms include DSM, deep mixing, CDSM, and soil cement mixing).
   a. The DMM grids and blocks are formed by an arrangement of at least two soil mixing shafts with overlapping augers and blades (paddles), guided by a lead mounted on a crawler base machine.
   b. The mixing shafts shall be driven by a power source sufficient to provide torque for the wide range of expected drilling conditions, indicated by the available boring and CPT logs and other test data included in the Geotechnical Investigation Report (GIR) and planned future CPT logs prior to construction.
   c. As the mixing shafts are advanced into the soil, grout is pumped through the hollow stem of the shafts and injected into the soil at the shaft tips. Auger flights and mixing blades on the shafts blend the soil with grout in a pugmill fashion. When the design depth is reached, the mixing shafts are withdrawn while the mixing process is continued.
   d. The mixing shafts are positioned so as to overlap one another to form continuously mixed overlapping columns. After withdrawal, two (or more) overlapping soil-cement columns remain in the ground.
   e. The process is then repeated to form grids and blocks of overlapping DMM columns.
3. DMM Design Addendum (DA): DMM Design Addendum No. 1 prepared by Fugro USA Land, Inc. dated June 10, 2022, and subsequent addenda.
4. DMM Elements: DMM columns will be used to create DMM grids and DMM blocks of treated soil referred to as ground improvement. A DMM grid will consist of interconnected DMM walls formed by partially overlapping columns arranged in a grid pattern with a replacement ratio less than 100 percent. A DMM block used to support a building footing will consist of interconnected DMM walls formed by overlapping columns arranged in a parallel pattern with a replacement ratio of 100 percent or less as shown in project plans and drawings. For this project, individual DMM Element refers to the grouping of columns installed simultaneously during single penetration of the DMM rig.
5. DMM Contractor: The firm performing the DMM construction.

6. DMM Layout Plan: The alternate DMM construction layouts designed by the DMM Contractor, which satisfy the requirements of this Specification. The DMM Layout Plans shall be reviewed and approved by the Geotechnical Engineer and Structural Engineer.
7. Cement Dosage: The amount of cement (in terms of dry weight) used to treat a given initial volume of in-situ soil.
8. Cone Penetrometer Test (CPT): A geotechnical exploration tool, as defined in ASTM D 5778.
9. Core Run: The total length reported by the driller as the actual depth penetrated by coring, including both recovered and unrecovered lengths.
10. Geotechnical Investigation Report (GIR): Geotechnical Investigation Report prepared by Fugro USA Land, Inc. dated February 28, 2020, and subsequent addenda. Note the DMM Design Addendum No. 1 (DA) supersedes the GIR.
11. Geotechnical Engineer: The geotechnical engineer of record responsible for the DMM design, who is hired by the Owner.
12. Ground Improvement Area: The plan area contained within a single perimeter shown on the DMM Design Plans that surrounds:
    a. - All planned soil-cement grids/blocks.
    b. - Unmixed soil within the grids.
13. Grout: A stable colloidal mixture of water, Portland cement, and admixtures. The purpose of the grout is to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in-situ soil.
14. Grout-Soil Ratio: A volumetric ratio of grout to in-situ soil to be mixed.
15. Owner: Peralta Community College District and its representatives.
16. Structural Engineer: The structural engineer of record responsible for designs of structure foundations supported by DMM, who is hired by the Owner.
17. Testing Laboratory: The testing laboratory of record performing construction material testing, which is hired by the Owner and approved by the Geotechnical Engineer. The Testing Laboratory shall be selected from the DSA approved laboratory list.
    a. Treatment Zone: A spatial zone of soil targeted for ground improvement. The vertical and lateral (horizontal) extents of the Treatment Zones are defined on the DMM Design Plans.

A.1.4 Submittals

1. Evidence of conformance to the referenced standards and requirements shall be submitted by the DMM Contractor to the Geotechnical Engineer for the following, but not limited to, in accordance with the requirements in this Specification.
    a. Cement: Certificate of compliance for each truck load delivery.
    b. Admixtures: If used, certificate of compliance for each load or lot of material delivered.
    c. Preliminary Mix Design: Proposed mix designs including all materials and quantities and documentation of calibration of the grout mixing plant.

d. Proposed Test Section Program, Sampling Plan, and Laboratory Testing Program, conforming to the requirements described in this Section.
e. Construction Schedule: Submit a detailed schedule that identifies start dates and duration of each major task in the work. The schedule shall at a minimum include information regarding equipment mobilization, equipment setup, soil-cement mixing test section, production installation, and verification testing.
f. Site Work Plan: Submit a site plan showing staging area for all on-site equipment, including anticipated sections of the streets which may require blocking of parking spaces or traffic clearances.
g. DMM Layout Plans: Submit 1”=20’ scale drawings showing proposed layout of DMM Elements (including test section(s) and production DMM), including column diameters, column overlap, grid sizes, tip elevations, top elevations, coordinates of the corners, foundations, and proposed column and element numbering scheme prior to site mobilization in hard copy and electronic format using the project coordinate system at least 14 calendar days prior to beginning DMM construction. The DMM Contractor must obtain the Geotechnical Engineer’s approval of the proposed column layout prior to beginning DMM construction.
h. Equipment and Procedures: Submit a detailed description of the equipment and procedures to be used during all DMM work including, but not limited to, construction of DMM test section(s), production DMM work, and collecting samples for laboratory confirmation testing. Procedures shall include methods for locating the DMM Elements in the field and confirming that the columns are plumb. In addition, while it is recognized that the specific responses to field difficulties are dependent on several factors, the DMM Contractor shall submit their anticipated responses to the following possible situations that could occur during construction and testing of the DMM columns including poor core sample recovery or inability to retrieve core samples, and failing production test results (e.g., repair and/or treatment of failed area and modification to approved procedures or mix design).
i. The DMM Contractor shall also submit the anticipated cement dosages (proportions) to achieve the acceptance criteria outlined under acceptance criteria in **Section A.3.15** of this Specification.
j. Quality Control Program, as outlined in the Execution Section of this Specification.
k. Daily Quality Control Reports: Prior to construction, submit a proposed Daily Quality Control Report format for approval by the Geotechnical Engineer. Submit the Daily Quality Control Report at the end of the next working day. The report should be in conformance with quality control in **Section A.3.14** of this Specification.
l. DMM Test Results: Submit all QC test results as outlined in quality control in **Section A.3.14** of this Specification.

m. Calibrations: Submit all metering equipment calibration test results including mixing systems, delivery systems, alignment systems, and mixing tool rotational and vertical speed.
n. Record Drawing: Submit record drawings prepared by the DMM Contractor indicating the as-built location and elevations of the DMM Elements in terms of project coordinates and vertical datum. The record drawings shall also indicate the above structure foundation designs and locations.

2. Upon completion of DMM installation, the as-built submittal by the DMM Contractor to the Geotechnical Engineer, Structural Engineer, and CGS shall include test section results, daily quality control reports, DMM core and lab test results, DMM record drawings, and any other information needed to document the work.

A.2 Part 2 – Products, Materials, and Equipment

A.2.1 Materials

1. Grout: The material added to the blended in situ soils shall be a water-based Portland cement grout. The purposes of the grout are to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in-situ soils. The grout shall be premixed in a mixing plant which combines dry materials and water in predetermined proportions.
2. Cement used in preparing the grout shall conform to ASTM C150 "Standard Specification for Portland Cement Type II". The cement shall be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter shall not be used.
3. Water: Fresh water, free of deleterious substances that adversely affect the strength and mixing properties of the grout, shall be used to manufacture grout.
4. Admixtures: Admixtures are ingredients in the grout other than Portland cement, and water. Admixtures of softening agents, dispersions, pozzolans, retarders or plugging or bridging agents may be added to the water or the grout to permit efficient use of materials and proper workability of the grout. However, no admixtures shall be used except as approved by the Geotechnical Engineer.

A.2.2 Equipment

The DMM equipment shall meet the following requirements:

1. The mixing tools shall have mixing augers and blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and grout.
   a. Multi-shaft mixing equipment (machines with at least two soil mixing shafts with overlapping augers and blades) shall be used.

  i. The mixing augers and blades shall be minimum 3 feet and maximum of 6 feet in diameter.
  ii. Allowable wear to mixing augers and blades will be limited such that equipment produces a column no less than the design diameter listed on the DMM Layout Plans.
  iii. The overlapping between any two adjacent DMM columns shall be at least 30 percent of the column diameter.
 b. The power source for driving the mixing shafts shall:
  i. be sufficient to provide torque for the wide range of expected drilling conditions, indicated by available boring and CPT logs and other test data included in the Geotechnical Investigation Report (GIR) and DMM Design Addendum (DA).
  ii. be sufficient to maintain the required revolutions per minute (RPM) and penetration rate from a stopped position at the maximum depth required.

2. The DMM rig shall be equipped with electronic sensors built into the leads to determine vertical alignment in two directions: fore-aft and left-right.
   a. The sensors shall be calibrated at the beginning of the project and the calibration data shall be provided to the Geotechnical Engineer. The calibration shall be repeated at intervals not to exceed three months per rig.
   b. The output from the sensors shall be routed to a console that is visible to the operator and the Geotechnical Engineer during penetration and reported. The console shall be capable of indicating the alignment angle in each plane.
3. The DMM equipment shall be adequately marked to allow the Geotechnical Engineer to confirm the penetration depth to within 6 inches during construction.
4. The grout shall be premixed in an on-site mixing plant, using a batch process, which combines dry materials and water in predetermined proportions. The mixing plant shall consist of a grout mixer, grout agitator, grout pump, batching scales, and a computer control unit.
   a. Dry materials shall be stored in silos. The dry materials shall be transported to the project site and blown into the on-site storage tanks using a pneumatic system.
   b. The air evacuated from the storage tanks during the loading process shall be filtered before being discharged to the atmosphere.
   c. Automatic batch scales shall be used to accurately determine mix proportions for water and cement during grout preparation.
   d. The dry admixtures, if used for mixing with water and cement, can be delivered to the mixing plant by calibrated auger. However, the DMM Contractor shall demonstrate that the calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

   e. Calibration of mixing components shall be done at the beginning of the project and repeated at intervals not to exceed three months thereafter and after each move of the batch plant.
5. Positive displacement pumps shall be used to transfer the grout from the mixing plant to the mixing tool head. The grout shall be delivered to each slurry-injecting tool head by an individual positive displacement pump.
6. The DMM rig shall be equipped with sensors to continuously monitor and record the mixing tool penetration/withdrawal speed, mixing tool rotation speed, and injection rate.
   a. The output from these sensors shall be visible to the Operator and Geotechnical Engineer during penetration and withdrawal.
   b. The DMM Contractor may propose alternative display/monitoring systems; however, the systems shall first be reviewed and approved by the Geotechnical Engineer prior to use.
   c. Calibration of this equipment shall be performed at the beginning of the project and the calibration data shall be provided to the Geotechnical Engineer. The calibration shall be repeated at intervals not to exceed three months.

A.2.3 Products

1. DMM: The in-place grout mix together with the soils shall meet all of the acceptance criteria specified in **Section A.3.15** of this Specification, determined according to the quality control, sampling, and testing methods specified in **Section A.3.14** of this Specification.

A.3 Part 3 – Execution

A.3.1 Observation of Work

1. The work covered by these specifications shall be performed under the observation of the Geotechnical Engineer, who shall be retained and paid by the Owner. The Geotechnical Engineer will be present at the site during the conduct of work to observe the work, and to perform field and laboratory tests, as deemed necessary by the Owner. The DMM Contractor shall cooperate with the Geotechnical Engineer in performing the observations and tests. At the completion of their work, the Geotechnical Engineer shall submit a report to the Owner, including a tabulation of all tests performed. The Geotechnical Engineer's costs for observing the construction, testing, and the repair of unsatisfactory work performed by the DMM Contractor shall be billed to the Owner. The Owner shall pay them and then shall deduct the amount from monies due to the DMM Contractor.
2. This work falls under the jurisdiction of the California Division of State Architect (DSA) who will review submittals and may observe portions of the work.

A.3.2 General

1. The soil-mixing shall be constructed by the DMM Contractor to the lines, grades, and cross sections indicated on the DMM Design Plans example layouts or an alternate layout approved by the Geotechnical Engineer, Structural Engineer, and CGS. Revisions to the approved layouts shall be submitted to Geotechnical Engineer of Record (GEOR) for review and approval. DMM ground improvement within a single structure shall be arranged in an uninterrupted grid that follows the structure column lines and underlies all footings and moment frame grade beams, tie beams, and shear walls as shown on the DMM Design Plans.
2. As shown on the DMM Design Plans, the DMM shall underlie the entire structure footprints and extend laterally to include any attached structures which are deemed to be essential parts of the structures.
3. Grading after the site demolition may be required to provide suitable level ground for constructing the DMM. The DMM contractor is responsible for coordinating with the site grading operation to define the Drill-Through Zone.
4. The minimum Area Replacement Ratio (Ar) for DMM grids and blocks and the maximum spacing for DMM grids depends on the specified unconfined compressive strength ($q_{dm,spec}$) as shown in **Table A.1**. Additional DMM Elements may be added within the untreated area to meet or reduce the slab free span distance as instructed by the Geotechnical and Structural Engineer.

Table A.1: Minimum DMM A_r and Maximum DMM Grid Spacing

Specified Unconfined Compression Strength, $q_{dm,spec}$ (psi)	Minimum A_r (%)	Zone B Maximum DMM Center-to-Center Grid Spacing[1]	Zones A1 and A2 Maximum DMM Center-to-Center Grid Spacing[1]
125	50	3.2d	4.0d

[1]d = DMM column diameter

5. DMM elements shall extend to at least the elevations indicated on the DMM Design Plans based on the penetration of the shortest mixing shafts.
6. The top of the DMM shall extend to the base of the ground floor slab section, the bottom of footings, the bottom of moment frame grade beams, and the bottom of elevator pits, as indicated in the DMM Design Plans.
7. Any proposed plan and Area Replacement Ratio by the DMM contractor should be approved by the Geotechnical Engineer and CGS.
8. Elevator pits shall be supported entirely by DMM.

9. The DMM columns shall be essentially vertical columns as stated in this Specification, with a minimum diameter of 3 feet and a maximum diameter of 6 feet and shall extend from the top to the bottom of the Treatment Zone indicated on the DMM Design Plans.
10. The overlapping between any two adjacent columns at ground surface shall be a minimum of 30 percent of column diameter.
11. The completed DMM shall be a homogeneous mixture of grout and the in-situ soils. Mixing is to be controlled by shaft rotational speed, drilling speed, and grout injection rate.
12. Monitoring of construction parameters and confirmation testing will be used to verify that the acceptance criteria have been satisfied.
    a. The DMM Contractor shall establish consistent procedures to be employed during DMM construction to ensure a relatively uniform product is created.
    b. These procedures are to be defined in the equipment and procedures submittal as defined in **Section A.1.4** of this Specification and subsequently modified, if necessary, based on the results of the pre-production testing or quality control testing.
13. The DMM Contractor may request that the established grout mix/grout-soil ratio design, equipment, installation procedure, or test methods be modified. However, the Geotechnical Engineer may require additional testing, at no additional cost to the Owner, to verify that acceptable results can be achieved.
    a. The DMM Contractor shall not employ modified grout mix/grout-soil ratio design, equipment, installation procedures, or sampling or testing methods until approved by the Geotechnical Engineer in writing.
    b. The Geotechnical Engineer, at his sole discretion, may reject any modification proposed by the DMM Contractor.

A.3.3 Construction Site Survey

The location of both active and abandoned buried utilities at the site can have significant impact on the design and construction of deep mixing works. Careful consideration of the presence and location of all utilities is required.

1. Prior to bidding, the contractor should review the available subsurface information and visit the site to assess the site geometry, equipment access conditions, location of existing structures, and above-ground utilities and facilities.
2. The contractor should field locate and verify the locations of all utilities prior to starting work. The contractor should maintain uninterrupted service for those utilities designated to remain in service throughout the work. The contractor should notify the engineer of any utility locations different from those shown in the plans that may require relocation of deep mixed elements or structure design modification. Subject to owner's geotechnical engineer's approval, the contractor should be compensated for additional costs of element relocation and/or structure design modifications resulting from utility locations different from those shown in the plans.

A.3.4 Site Access for Soil Samples

1. After award of the Contract, the DMM Contractor will have the option of accessing the jobsite to collect additional soil samples for use in mix designs with the following requirements:
   a. Prior to commencing with field work, the DMM Contractor shall obtain all necessary permits for sampling activities, including drilling permits from Alameda County Public Work Agency, if applicable.
   b. The DMM Contractor shall submit to the Geotechnical Engineer a sampling plan indicating in detail the sampling activities proposed, and the proposed methods for backfilling boreholes or excavations and restoring the site.
   c. Cement grout backfill for boreholes per Alameda County Public Work Agency is required.
   d. The soil sampling and testing will be performed by DMM Contractor. The costs of additional soil sampling and testing (if performed) are to be included in the project DMM construction costs.

A.3.5 Test Sections

1. The DMM Contractor shall construct a minimum of one test section on site to demonstrate. that the proposed mix design, equipment, and procedures will meet the specified requirements. The location(s) of the test section(s) shall be determined by the DMM Contractor with the approval of the Geotechnical Engineer.
2. Additional test sections may be performed at the DMM Contractor's option to optimize the mix design and procedures.
3. Each test section must extend at least to the deepest DMM design depth as indicated by the DMM Design Plans.
4. The costs of the test section(s) are to be included in the project DMM construction costs.
5. Each test section shall consist of at least two full strokes of the DMM equipment. For example, if the DMM rig uses three augers, then the test section shall consist of 2 strokes times 3 columns equal 6 columns.
6. Test sections shall not be located directly below proposed footings, moment frame grade beams, and elevator pits. However, the test sections may be constructed in place of other production DMM columns, provided it is later demonstrated that the test sections meet all acceptance criteria. If the test sections are found to fail the acceptance criteria, the DMM Contractor shall make necessary repairs or replace the DMM Elements to the written satisfaction of the Geotechnical Engineer and CGS.
7. During the time interval between construction of the test section(s) and the completion of laboratory test results, the DMM Contractor may proceed with production DMM installation at their own risk. Any production DMM found to fail the acceptance criteria must be

repaired at the DMM Contractor's expense, to the written satisfaction of the Geotechnical Engineer and CGS.

8. A minimum of two (2) full-depth cores shall be obtained from each test section, according to the procedures detailed in this Specification.
9. Laboratory tests, as specified in this Specification, shall be performed on a minimum of ten samples per full-depth core or a minimum of one sample per core run, whichever is greater, from each test section, as selected by the Geotechnical Engineer. Additional cores may be performed to retrieve enough test samples.

A.3.6 Horizontal Alignment

1. The DMM Contractor shall accurately stake the location of DMM Elements using a surveyor before beginning installation. The main survey control for a given area shall be established by a California licensed surveyor; layout of individual DMM Elements does not require a licensed surveyor. Horizontal alignment of DMM columns shall conform to the geometric tolerances in the acceptance criteria of this Specification.
2. The DMM Contractor shall provide an adequate method to allow the Geotechnical Engineer to verify the as-built location of the DMM during construction.
3. Movement of the crawler base machine shall provide the preliminary alignment of the augers and the final alignment shall be adjusted by hydraulic manipulation of the leads.
4. One stroke of the machine shall construct a DMM Element consisting of at least two overlapping columns.
5. The DMM shall be advanced stepwise by overlapping the adjacent columns of the previous strokes.
6. Following DMM construction, the DMM Contractor shall submit as-built drawings indicating the location of the DMM elements in terms of project coordinates and elevation datum.
7. The DMM contractor should provide a construction plan at least two (2) weeks prior to the start of construction that includes the plan showing the numbering and location of the DMM columns, tip elevations or depths, and cut-off (top) elevations. The daily work plan should be provided to the Geotechnical Engineer at the beginning of workday and work progress should be checked and confirmed by the Geotechnical Engineer during and at the end of each day. The DMM contractor should provide a summary progress report to the Geotechnical Engineer at the end of each workday.
8. The location of known obstructions or utilities at or near the treatment area should be marked on the project drawings and on the ground before construction begins. Existing obstructions within the treatment zone area should be removed prior to construction. It is not anticipated that drilling obstructions will be encountered within the Treatment Zone during DMM construction unless further site investigation reveals otherwise.
    a. If an obstruction preventing drilling advancement is encountered, the DMM Contractor shall investigate the location and extent of the obstruction using methods approved by

the Geotechnical Engineer. The DMM Contractor shall propose remedial measures to clear the obstruction for approval by the Geotechnical Engineer.

b. While the investigation for an obstruction is underway, the DMM Contractor shall continue to install columns in areas away from the obstruction location. No stand-by delay will be allowed for equipment and operations during the investigation of an obstruction.
c. The DMM Contractor will be compensated for removal or clearing of obstructions as a Changed Condition, paid in accordance with the General Conditions.
d. The DMM Contractor will not be compensated for removal or clearing of obstructions without prior approval by the Geotechnical Engineer and the Owner.

9. The DMM Contractor will not be compensated for DMM Elements that are located outside of the tolerances specified in the acceptance criteria.

A.3.7 Vertical Alignment

1. The equipment operator shall control vertical alignment of the auger stroke. Verticality shall be monitored with respect to two orthogonal horizontal axes. Vertical alignment of DMM columns shall conform to the geometric tolerances in the acceptance criteria of this Specification.

A.3.8 DMM Depth

1. DMM depths shall extend to the line and grades shown on the DMM Design Plans.
2. The total depth of penetration shall be measured either by observing the length of the mixing shaft inserted below a reference point on the mast, or by subtraction of the exposed length of shaft above the reference point from the total shaft length.
   a. For each stroke, the elevation of the reference point on the mast must be established within one inch using measurements from a surveyed control point.
   b. The final depth and bottom elevation of the stroke shall be noted and recorded on the Daily Quality Control Report by the DMM Contractor. The equipment shall be adequately marked to allow the Geotechnical Engineer to confirm the penetration depth during construction.
3. If rigs with varying mixing shaft lengths are used, the shortest shafts shall extend to the minimum DMM depths indicated on the DMM Design Plans.

A.3.9 Grout Preparation

1. Dry material shall be stored in silos and fed to mixers for agitation and shearing. In order to accurately control the mixing ratio of grout, the addition of water and cement shall be determined by weight using the automatic batch scales in the mixing plant.
   a. The admixtures, if used, for mixing with water and cement, can be delivered to the mixing plant by calibrated auger. However, the DMM Contractor shall prove that the

calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

2. A minimum mixing time of one minute and a maximum holding time of four hours will be enforced for the grout.
    a. The grout hold time shall be calculated from the beginning of the initial mixing.
3. The specific gravity of the grout shall be determined during the design mix program for double checking grout proportions.
    a. The specific gravity of the grout shall be checked by the DMM Contractor at least twice per shift per rig using the methods outlined in ASTM D4380.
    b. The specific gravity of the grout measured in the field should not deviate by more than 3 percent of the calculated specific gravity for the design cement ratio.
    c. If the specific gravity is lower than that required by the design mix, the DMM Contractor shall add additional cement and remix and retest the grout at no cost or schedule impact to the Owner.
    d. The specific gravity measurements shall be indicated on the Daily Quality Control Report.

A.3.10 Soil-Grout Mixing

1. Installation of each column shall be continuous without interruption.
    a. If an interruption of more than one hour occurs, the column shall be remixed (while injecting grout at the design grout ratio) for the entire height of the element at no additional cost to the Owner.
    b. If an interruption of more than ten minutes occurs, the DMM Contractor shall inject a volume of grout equal to that required for three feet of auger penetration, while maintaining constant auger elevation. Once the specified volume of grout has been injected, auger penetration may continue.
2. The completed CSDM shall be a uniform mixture of cement grout and the in-situ soils.
    a. Soil and grout shall be mixed together in place by the specially designed overlapping augers or blades on the mixing shafts.
    b. The grout shall be pumped through the mixing shafts and injected from the tip of the shafts. The shafts shall break up the soil and blend it with cement grout.
    c. The mixing action of the shafts shall blend, circulate, and knead the soil over the length of the column while mixing it in place with the grout.

A.3.11 Shaft Rotational Speed and Penetration/Withdrawal Rate

1. The mixing shaft rotational speed (measured in RPMs) and penetration/withdrawal rates shall be established before beginning work. It may be adjusted with the approval of the Geotechnical Engineer to achieve adequate mixing.

2. The contractor shall obtain the suitable shaft rotational speed during the installation of test section. The rotational speeds and penetration/ withdrawal rates shall be recorded on the Daily Quality Control Report.
3. The established rotational speeds and penetration/withdrawal rates shall be used during the work. If these parameters are varied more than ten (10) percent from those determined during the test section(s), the Geotechnical Engineer may require additional testing, at no additional cost to the Owner, to verify that the acceptance criteria are met.
4. The DMM Contractor may request that the established mixing parameters be modified during the production DMM installation. To verify acceptable results for the modified parameters, the Geotechnical Engineer may require additional testing at no additional cost to the Owner.

A.3.12 Grout Injection Rate

1. The grout injection rate per no more than three vertical feet of column shall be in accordance with the requirements of the design mix.
    a. The required mix design and grout-soil ratio shall be determined during the test section(s).
    b. The grout injection rate shall be constantly monitored and controlled.
    c. The DMM Contractor shall record the volume of grout injected continuously for each column on the Daily Quality Control Report.
2. If the volume of grout injected per three vertical feet of column is less than the amount required to meet the grout-soil ratio established during the test section, the DMM columns shall be remixed and additional grout injected (at the design grout-soil ratio) to a depth at least 3 feet below the deficient zone or until design depth is met, at no additional cost to the Owner.
3. The DMM Contractor may request that the established grout-soil ratio be modified during the production DMM installation.
    a. To verify acceptable results for the modified grout-soil ratio, the Geotechnical Engineer may require additional testing or a new test section at no additional cost to the Owner.

A.3.13 Control of Spoils

1. The DMM Contractor shall control and process all spoils created during the DMM construction.
    a. Prior to stockpiling materials greater than 10 feet in height, stockpile locations and heights shall be submitted for review and approval to Geotechnical Engineer. The DMM Contractor shall consider the locations of and avoid damage to existing utilities, structures, and other improvements, as well as recently constructed DMM Elements when stockpiling material. Lesser stockpile heights may be necessary in some areas.

b. The spoils shall be processed until they have cured to a sufficient level to allow them to be stockpiled such that they will not reform a cemented mass in the stockpile. The DMM Contractor shall dispose of spoils in accordance with all local laws, codes, and ordinances in a manner acceptable to the Owner or coordinate with the project grading contractor for the spoils be reused as fills at the project site.

A.3.14 Quality Control Program

1. The DMM Quality Control Program shall be the responsibility of the DMM Contractor and shall include, as a minimum, the following components:
   a. An approved pre-construction test program on soils obtained from the project site, to establish appropriate design parameters such as cement dosage and water content.
   b. Field monitoring by the DMM Contractor of construction parameters during DMM construction.
   c. Sample collection including full depth continuous coring, sample storage, and sample transportation to the Testing Laboratory.
   d. Reporting of the field monitoring and sampling performed by the DMM Contractor.
   e. Reporting of the core strength testing performed by the Testing Laboratory.
2. Prior to site mobilization, the DMM Contractor shall submit a detailed work plan for the Quality Control Program for review and approval by the Geotechnical Engineer. The work plan shall include, as a minimum:
   a. A description of all installation, monitoring, sampling, and testing procedures to be implemented. The proposed auger penetration and withdrawal rates shall be proposed by the DMM Contractor at this time.
   b. Descriptions of all sampling equipment.
   c. A list of parameters to be monitored.
   d. Tolerances for the parameters monitored.
   e. Names of any subcontractors.
3. The DMM Contractor shall provide all the personnel and equipment necessary to implement the Quality Control Program. Contractor to provide the number of years/projects, project descriptions, and reference list for all cases below:
   a. The DMM Contractor must have at least 7 (seven) years of previous successful experience with at least 5 (five) DMM projects for soil conditions and project scope similar to that of the project being bid.
   b. The DMM contractor must have a registered California Professional Engineer (PE) who have had at least 5 (five) years of experience with at least 3 (three) DMM projects.
   c. The DMM Contractor must have assign a project manager who have had at least 5 (five) years of experience on at least 3 (three) DMM projects.
   d. The DMM Contractor must have assign a project engineer/ supervisor who have had at least 3 (three) years of experience with at least 2 (two) DMM projects.

e. The DMM Contractor must assign a full-time project superintendent with at-least 3 (three) DMM projects with at least 150,000 cubic yard of total treatment volume in DMM construction.
f. The DMM equipment operator must have at least three years of experience with the equipment and DMM construction.
g. Written requests for substitution of these key personnel must be submitted prior to personnel changes. Documentation must be submitted to the owner that demonstrates that the substitute meets the requirements listed. Substitution may not be made until written approval is provided by the owner.

4. The Geotechnical Engineer will continuously observe the DMM construction. The Geotechnical Engineer will review DMM Contractor submittals to check that the Quality Control Program is being properly implemented.
5. The established quality control procedures shall be maintained throughout the production DMM installation to ensure consistency in the installation and to verify that the work complies with all requirements indicated in the DMM Design Plans and Specifications, unless modifications to the procedures are approved in writing by the Geotechnical Engineer.
6. DMM Contractor shall perform sample collection, storage, and transportation.
   a. DMM Contractor shall collect one full-depth continuous coring should be made for every 3% of the total DMM elements or for every 900 square feet of treated ground, whichever produces the greater number of cores at locations specified by the Geotechnical Engineer.
      i. The coring rig shall be a triple-barrel rig approved by the Geotechnical Engineer and capable of achieving the required recovery. The ability to achieve the recovery criteria is solely the Contractor's responsibility.
      ii. Full-depth samples obtained by the DMM Contractor shall have a diameter of at least 3 inches.
      iii. The continuous core sample shall extend from the top through the bottom of the Treatment Zone, and to at least 5 feet below the Treatment Zone to sample the foundation soil directly below the Treatment Zone.
      iv. Unless otherwise directed by the Geotechnical Engineer, the full-depth samples shall be obtained along an essentially vertical alignment located one-fourth of a column diameter from the column center and not within column overlaps.
      v. The DMM Contractor shall perform all full-depth sampling in the presence of the Geotechnical Engineer.
      vi. Full-depth core samples shall be retrieved using triple tube continuous coring techniques after the soil-grout mixture has hardened sufficiently.
      vii. Each core run shall be a minimum 4 feet in length.

viii. Following logging, the engineer will select at least five specimens from each full-depth continuous core for strength testing. Each test specimen should have a length-to-diameter ratio of 2 or greater.
ix. A minimum recovery of 85 percent for each 4-foot core run shall be achieved for cores from within the Treatment Zones. During coring, the elevation of the bottom of the holes shall be measured after each core run in order that the core recovery for each run can be calculated.
x. The DMM Contractor shall determine the time interval between column installation and coring except that the interval shall be no longer than required to conduct 28-day strength testing.
xi. The DMM contractor should photograph each core run and submit to the Geotechnical Engineer for test sample selection.
xii. Upon retrieval, the core runs shall be provided to the Geotechnical Engineer for logging, uniformity inspection, and test specimen selection.
xiii. Following logging and test specimen selection by the Geotechnical Engineer, the entire full-depth core, including the designated test specimens, shall be immediately sealed in plastic wrap to prevent drying and transported to the laboratory by the DMM Contractor. Alternatively, the DMM Contractor may transport only the selected test specimens to the laboratory and store the remaining core on-site in a humidity and temperature-controlled storage facility as described in this Specification.
xiv. All core holes shall be filled with cement grout that will obtain a 28-day strength equal to or greater than the strength of the DMM. However, the Contractor shall not grout the core holes until after acceptable core recover and uniformity has been confirmed by the Geotechnical Engineer.
xv. The DMM Contractor shall notify the Geotechnical Engineer at least one business day (24 hours) in advance of beginning core sampling operations.

b. In addition to coring, the DMM Contractor should obtain wet grab samples from the DMM elements at the presence of Geotechnical engineer.
   i. 3 (three) wet samples from each mixed design used in each test section as directed by the geotechnical engineer.
   ii. One wet sample (i.e., one selected depth at one location) should be retrieved every 2 (two) production days or for every 2,500 cubic yards of treated soil, whichever produces the higher sampling frequency.
   iii. The contractor proposes locations for wet sampling as outlined in the QC program, considering input from the geotechnical engineer based on subsurface conditions, DMM layout, review of the QC results, and observation of the soil mixing operation.
   iv. The contractor should report all attempts, successful and unsuccessful, to obtain wet samples. Some deep mixed material may not be able to be sampled readily

because either the mixture is too stiff or the material may not flow back into the void left after the sampler is extracted, possibly leaving a damaged element.

v. The sampling tool is inserted into the DMM column to a designated depth, filled with treated soil, and lifted to the ground surface. The treated soil material is then poured into a container, screened for oversized lumps (gravel versus unmixed soil), and placed in 3-inch (76-mm)- diameter, 6-inch (152 mm)- long molds. Eight test specimens should be prepared from each wet sample.

vi. The wet treated material should be placed into the mold in three to five layers. After the placement of each layer, the specimens must be tapped or vibrated to remove trapped air bubbles. The specimens should be sealed to prevent moisture from entering or leaving the specimens, and the sealed specimens should be stored in a humid environment in accordance ASTM C192.

vii. For field validation testing, unconfined compressive strength testing may be performed on specimens at 3, 7, 28, and 56 or more days. For full production work, unconfined compressive strength testing may be performed at 3, 7 and 28 days.

viii. The DMM contractor should deliver the samples for testing to a local lab as directed by the geotechnical engineer.

ix. If wet samples produce results that are consistently acceptable, the frequency of wet sampling can be reduced as the project progresses.

x. The engineer may request additional test specimens for QA testing.

c. Untested portions of the full-depth samples shall be retained at the laboratory until completion and acceptance of all DMM, for possible inspection and confirmation testing by the Geotechnical Engineer.

7. The DMM Contractor shall be responsible for handling of test specimens, including storing of untested specimens and transporting test specimens to the Testing Laboratory.
   a. The laboratory testing shall be performed by the DSA accepted Testing Laboratory hired by the Owner and approved by the Geotechnical Engineer.
   b. The samples shall be stored in a moist room as specified in ASTM C 192 until the test date.
   c. Testing for 28-day unconfined compressive strength shall be conducted in accordance with ASTM D2166.
8. In addition to confirmation tests performed by the Testing Laboratory, additional tests may be requested by the Geotechnical Engineer on samples collected by the DMM Contractor. Both the Testing Laboratory's testing and the Geotechnical Engineer's requested additional testing (if performed) shall demonstrate that the acceptance criteria are met prior to acceptance of the work.
9. Daily Quality Control Report
   a. The DMM Contractor shall submit Daily Quality Control Reports to the Geotechnical Engineer at the end of the next working day. The Daily Quality Control Report shall

document the progress of the DMM construction, present the results of the QC parameter monitoring, present the results of the strength testing, and clearly indicate if the columns have met the acceptance criteria. The DMM Contractor shall make all Daily Quality Control Reports available to the Geotechnical Engineer

b. The Daily Quality Control Report shall include as a minimum the results of the following QC parameter monitoring for each column:
   i. Rig number,
   ii. Type of mixing tool,
   iii. Date and time (start and finish) of column construction,
   iv. Column number and reference drawing number,
   v. Column diameter,
   vi. Column top and bottom elevations,
   vii. Grout mix design designation,
   viii. Slurry specific gravity measurements (refer to Section A3.9 for number of tests and tolerance), and
   ix. Description of obstructions, interruptions, or other difficulties during installation and how they were resolved.

c. The Daily Quality Control Reports shall also include the following parameters recorded automatically for each column continuously and submitted in the form of either tables or figures (as agreed to by the Geotechnical Engineer):
   i. Elevation in feet vs. real time,
   ii. Shaft rotation speed in RPMs vs. depth,
   iii. Penetration and withdrawal rates in feet per minute vs. depth,
   iv. Grout injection rate in gpm vs. depth, and
   v. The average quantity of grout in gallons per foot injected per 3-foot (or less) vertical increment of column vs. depth.

A.3.15 Acceptance Criteria

1. The Geotechnical Engineer and CGS shall make the sole determination as to whether the acceptance criteria have been satisfied. The in-place grout-soil mixture comprising the DMM Elements shall meet the following acceptance criteria:
   a. The DMM within the Treatment Zone shall be installed within the following geometric tolerances:
      i. The horizontal alignment of the DMM blocks shall be within 4 inches of the location shown on the approved DMM Layout Plans.
      ii. The vertical inclination of the DMM columns shall be no more than 1: 100 (horizontal to vertical).

    iii. Overlap between any two adjacent columns shall be a minimum of 30 percent of column diameter for the entire depth, as calculated based on depth of column embedment and measured auger lateral and longitudinal inclination.
    iv. The tops of the columns shall be at or higher than the elevations indicated on the DMM Design Plans.
    v. The bottoms of the columns shall extend to or lower than the levels indicated on the DMM Design Plans.
  b. Two alternative specified unconfined compressive strengths are provided with corresponding minimum A_r and maximum grid spacing in **Table A.1**. The DMM Contractor shall select one of these options for the entire project.
  c. The unconfined compressive strength shall be determined by ASTM D2166 at 28 days on samples taken by coring of the constructed DMM.
  d. 80 percent of all unconfined compressive strength testing on core samples determined by ASTM D2166 from each tested deep mixed element shall equal or exceeds the specified strength. If a strength specimen falls below the specified strengths due to an obviously unrepresentative lump of unmixed soil in the specimen, the Geotechnical Engineer has the option to select another specimen from the same core run and allow the Testing Laboratory to test the replacement specimen and substitute the strength from the replacement specimen for the strength from the unrepresentative specimen that failed to satisfy the strength requirement. Only one such retest will be allowed per core run.
  e. 90 percent of all the test results on core samples across the site should equal or exceed the specified strength.
  f. To prevent a weak layer at one elevation in the DMM foundation system, strengths below the specified strength are not permitted within 10 feet of the same elevation in more than 2 nearby cored elements.
  g. Uniformity of mixing within the target zone shall be evaluated by the Geotechnical Engineer based on the full-depth samples recovered by the DMM Contractor from the columns.
    i. Lumps of unimproved soils shall not amount to more than 15 percent of the total volume of any core run from a continuous full-depth core sample. For evaluating the volume of unimproved lumps of soil, all of the unrecovered core length shall be assumed to be unimproved soil.
    ii. Any individual or aggregation of lumps of unimproved soil shall not be larger than 12 inches in greatest dimension.
    iii. Continuous core recovery shall be at least 85 percent over any full-length core.
2. If the acceptance criteria specified in this Specification are not achieved for production DMM, the failed section of DMM shall be rejected.

a. Unless otherwise determined by the Geotechnical Engineer, the failed section of DMM shall be considered to include all DMM columns constructed during all rig shifts that occurred between the times of construction when passing tests were achieved.
b. The DMM Contractor may conduct additional sampling and testing to better define the limits of the failed area at no additional cost to the Owner.
c. The DMM Contractor shall submit a proposed plan for remixing or repair of failed sections for review and approval by the Geotechnical Engineer and CGS.
d. If the treated soil that failed to meet the uniformity criteria is concentrated in a narrow elevation range forming weak planes or zones, the contractor could propose redrilling and remixing to 3 feet below and above the deficient zone. IF redrilling and remixing cannot be done efficiently, the contractor must replace the elements to the full depth. If the treated zone in the narrow elevation meets the uniformity criteria but fails to meet the strength criteria, the contractor could propose to redrill and remix the deficient zone or to assign a lower strength level to the deficient zone and install additional elements to compensate for the strength deficiency.
e. If the treated soil that failed to pass cannot be isolated in a specific zone, the contractor must provide remedial measures for all elements constructed during all rig shifts that occurred between passing elements.
f. Remedial measures are subject to coring and application of the specification acceptance criteria.