CITY OF PETALUMA

POST OFFICE BOX 61
PETALUMA, CA 94953-0061

Public Works & Utilities

Administration and Engineering
1318 Redwood Way, Ste. 120
Petaluma, CA 94954

City Engineer
11 English Street
Petaluma, CA 94952
Phone (707) 778-4303

Environmental Services
Ellis Creek Water Recycling Facility
3890 Cypress Drive
Petaluma, CA 94954
Phone (707) 776-3777
Fax (707) 656-4067

Petaluma Airport and Marina
601 Sky Ranch Dr.
Petaluma, CA 94954
Phone (707) 778-4404

Streets Maintenance
840 Hopper St.
Petaluma, CA 94952
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Fax (707) 206-6065

Transit Division
555 N. McDowell Blvd.
Petaluma, CA 94954
Phone (707) 778-4421

Utilities & Field Operations
202 N. McDowell Blvd.
Petaluma, CA 94954
Phone (707) 778-4546
Fax (707) 206-6034

E-Mail:
publicworks@
cityofpetaluma.org

ADDENDUM NO. 2

ELLIS CREEK CHEMICAL SYSTEM UPGRADE PROJECT PHASE I
C66501840

July 3, 2024

This Addendum No. 2 modifies the bidding documents for the Ellis Creek Chemical System Upgrade Project Phase I. This Addendum shall become part of the Contract and all provisions of the Contract shall apply thereto. Bidders shall acknowledge all Addendums in the Bid Schedule.

GENERAL

Bidders shall find responses to Contractor questions received thus far, revised drawings (Attachment A) and specifications (Attachment B). Any drawings or specifications not included in this Addendum have no revisions and shall remain as originally posted.

Bidders shall also find the Geotechnical Reports in Attachment C.

Finally, the list of pre-bid attendees is included as Attachment D.

NOTICE INVITING BIDS CHANGE

Notice Inviting Bids – Receipt of Bids – Page 1 – **CHANGE** "2:00 PM on Thursday, July 11, 2024" to "**2:00 PM on Thursday, July 18, 2024**."

Notice Inviting Bids – Opening of Bids – Page 1 – **CHANGE** "2:00 PM on Thursday July 11, 2024" to "**2:00 PM on Thursday, July 18, 2024**."

CLARIFICATION TO CONTRACTORS – QUESTION AND RESPONSE

Question #1: Does Section 13140 pertain to the Analyzer enclosure on Sheet M-7?

Response #1: Yes. Section 13140 pertains to the Analyzer enclosure shown on M-6 and M-7.

Question #2: Is a 25-Year Warranty on materials and workmanship appropriate? Or would the owner prefer a 20-Year Warranty instead?

Response #2: 20-Year Warranty is acceptable.

Question #3: 1.02.A.2 calls for waterproof and watertight. Flood waters and rain driven at 99 mph will get into any manufacturer's building. Is water-resistant acceptable?

Response #3: Water Resistant is acceptable. Section 1.02A.2 has been revised to change 'completely waterproof' to 'water resistant'.

Question #4: 1.02.A.2 calls for airtight. No manufacturer's building is perfectly airtight, and many applications call for airflow, either with a fan or with one or more natural air vents. Please confirm no airflow is needed in/out of this building.

Response #4: Analyzer enclosure building shall feature filtered vents and small vent fan. Section 1.02A.2 has been revised to remove 'air tight' from requirements.

Question #5: 2.03.B calls for permanently fused building. Is a building that could be taken apart, if needed, also be acceptable?

Response #5: Analyzer enclosure building shall be shop fabricated and assembled per specification section 13140, 1.01A1.

Question #6: 2.04.A calls for high gloss molds. Are matte molds (that mute the sun's reflection) also acceptable?

Response #6: Matte molds are acceptable.

Question #7: 3.02.A calls for field erected panels. This seems to conflict with the 2.03.B requirement for a permanently fused building. Is a fully assembled building also acceptable?

Response #7: Analyzer enclosure building shall be shop fabricated and assembled. Specification section 3.02A has been removed.

Question #8: 3.02.B calls for neoprene base gasket. Is ConSeal (cut sheet attached) also acceptable?

Response #8: Yes. ConSeal would be acceptable.

Question #9: Will the floor be a concrete slab by others?

Response #9: Floor will be existing concrete.

Question #10: May we assume the door is at least 3' wide x 6'-8" high?

Response #10: No. Door shall be 2'-6" wide and 6'-8" tall.

Question #11: Should the door hardware be 3-point pad lockable? Or should it be panic touchbar key-lockable?

Response #11: Both 3-point pad lockable and panic touchbar key lockable are acceptable. It is acceptable to assume 3-point pad lockable alternative for biding purposes with the expectation that final decision will be made during review of shop drawings.

Question #12: Should the door threshold be 2.75" high FRP step-over? Or should it be low-profile aluminum ½" high?

Response #12: Door threshold shall be low-profile, ½" high.

Question #13: Does the door require a window (nominal 15" x 15")?

Response #13: Window is not required. Section 1.1A.2 has been revised to remove 'windows' for the requirements.

Question #14: Will there be any field penetrations through the FRP larger than 2" diameter?

Response #14: It is not anticipated that there will be any field penetrations larger than 2" diameter at the Analyzer enclosure building.

Question #15: Will anything weighing more than 10 lbs. be field attached to the FRP? If so, how many 4' x 4' areas of reinforcement are needed?

Response #15: No. No single instrument or device weighs more than 10 lbs.

Question #16: Are white, green, tan, or gray sufficient exterior color options from which to choose?

Response #16: The proposed color options are sufficient.

Question #17: Is any electrical needed when the building arrives on site?

Response #17: No.

Question #18: Electrical terminations in junction box or breaker panel? If breaker panel; 120/240V, single-phase, 100A main breaker, NEMA 1?

Response #18: Termination box.

Question #19: Schedule 40 PVC conduit?

Response #19: Yes, for wiring and signal inside the analyzer enclosure building.

Question #20: Does the shelter need any duplex GFCI receptacles that are weatherproof when not-in-use?

Response #20: Yes, one is required.

Question #21: Does the shelter need LED interior lights providing at least 50 lumens per sf on average?

Response #21: Yes, one ceiling mounted LED light and external mounted switch is required.

Question #22: Does the shelter need any exterior LED floodlight or downlight controlled by photocell?

Response #22: Exterior LED floodlight is not required.

Question #23: Petaluma, CA has gotten as low as 16°F. Is a heater needed for freeze protection (40°F minimum)?

Response #23: No, if the staff requires a heater during periods of freezing temperatures, they will have the ability to install a plug-in portable heater.

Question #24: Are there any other unique requirements of this building that you would like to discuss?

Response #24: No unique requirements of the analyzer enclosure have been identified outside of the requirements described in specification section 13140 and responses to pre-bid questions provided herein. See photo below of an existing analyzer enclosure building currently being used on site:

Question #25: Detail 1 on Drawing C-15 shows a section of 13" of AB and 4" of AC paving on top of 12" of scarified and compacted subgrade. The Tilt and Cross Section on Drawing C-13 shows grading the band 3:1 on each side of the paving. Please clarify if is the intent to scarify the existing grade and install the AB and paving on top or if it is the intent to over excavate and remove the top 17" of material before proceeding with the subgrade prep, AB, and paving. If over excavation is required, there will be several thousand yards of material to dispose of.
Response #25: The intent is to scarify the existing grade and install the ab and paving on top and match existing grade with 3:1 max side slopes. Over excavation or additional scarifying outside of the ac limits is not intended.

Question #26: The Bid Schedule Items 16, 17, 18, 19, and 21 do not seem to apply to the project and no specifications cover these items. Are they part of the contract?

Response #26: The Bid Schedule has been revised accordingly.

Question #27: Are the areas of non-disturbance marked out? (Question received during the Pre-Bid Site Walk).

Response #27: Contractor shall coordinate with operations staff for confirmation of non-disturbance areas. Tree removal to be done in accordance with specification sections 01205 and 02050. Additionally, information for protection of trees has been identified in specification section 02050, 3.07.

Question #28: Site walk Question -2: Can the bid due date be extended 1 week to account for 4th of July? (Question received during the Pre-Bid Site Walk).

Response #28: Bid date has been extended until July 18th.

Question #29: For dewatering performed during construction, where should the water be disposed into? (Question received during the Pre-Bid Site Walk).

Response #29: Dewatering to be performed in accordance with specification sections 01500 – Temporary Controls, and 02140 - Dewatering. Water from dewatering operations shall be disposed of in conformance with the NPDES permit and as approved by the RWQCB. Specification section 02140, 1.04B4 requires dewatering submittal to include proposed disposal locations. The disposal location can be any of the nearby located oxidation ponds (Ponds 1 – 10 and the Aerated Lagoon).

REVISIONS TO DRAWINGS (*See Attachment A)*

1. Sheet E-2: See attached for clouded items indicating change.
2. Sheet E-10: See attached for clouded items indicating change.

REVISIONS TO SPECIFICATIONS (*See Attachment B)*

1. Multiple revision to Specification 13140 – Fiberglass Reinforced Plastic Building per Pre-Bid questions and responses.

2. Specification 09 Bids – Bid Schedule has been revised to remove prior Bid Schedule Items 16, 17, 18, 19, and 21.

This Addendum No. 2 shall become part of the Contract and all provisions of the Contract shall apply thereto.

City of Petaluma

Steve Worrell Digitally signed by Steve Worrell DN: C=US, E=sworrell@cityofpetaluma.org, O=Petaluma Public Works & Utilities, CN=Steve Worrell Reason: I am approving this document Date: 2024.07.03 11:22:18-07'00'

Steve Worrell, MS, PE
Sr. Civil Engineer
Public Works & Utilities

Attachments:
A – Revised Drawings
B – Revised Specifications
C – Geotechnical Report
D – Pre-Bid Attendees

A signed copy of this Addendum and the attached acknowledgement form shall be attached to the bid proposal. Failure to do so may cause rejection of your bid as being non-responsive.

ADDENDUM NO. 2

ACKNOWLEDGEMENT

Receipt of Addendum No. 2 is hereby acknowledged by ____
(Contractor's Name)

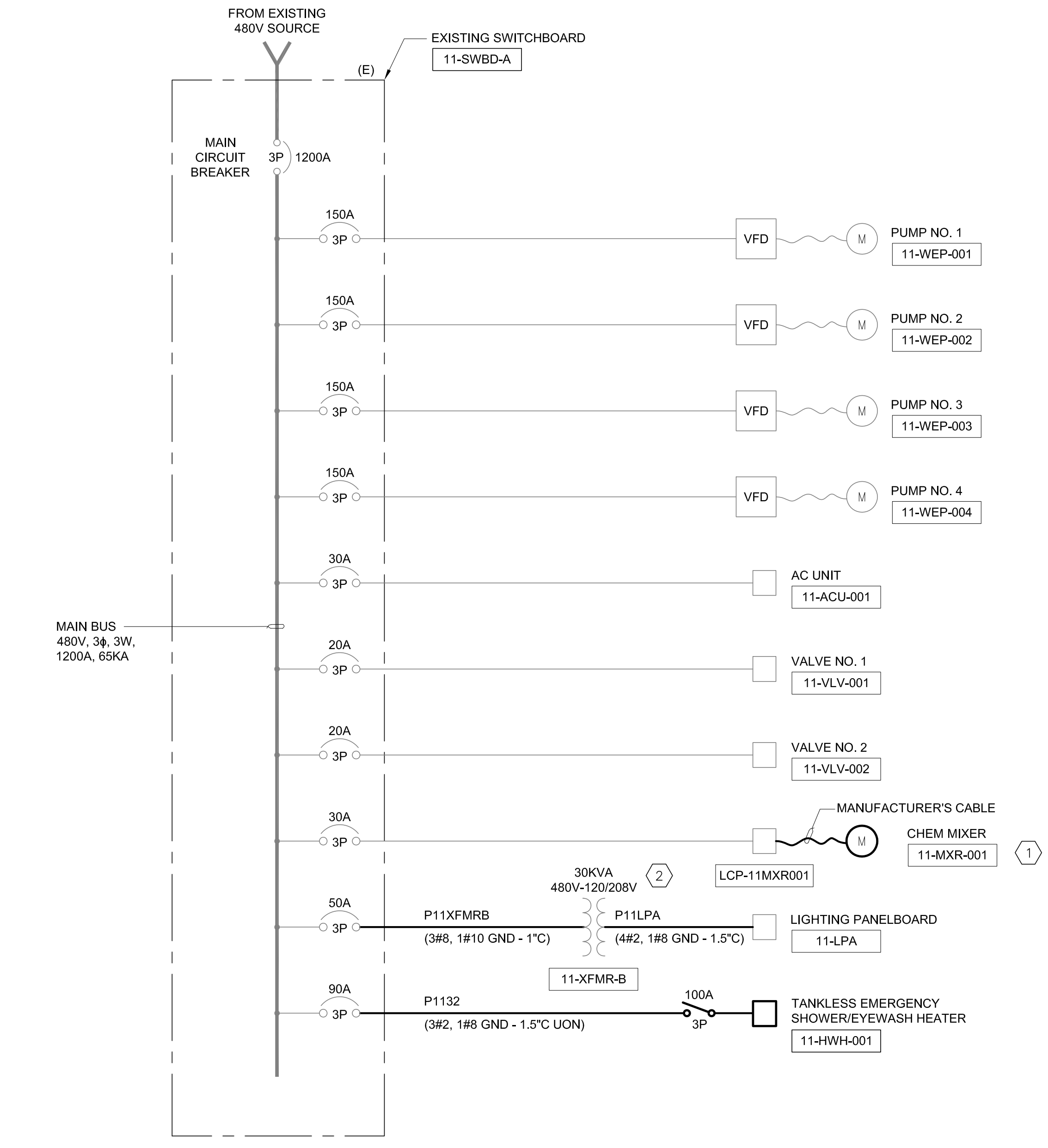
on the ___________ day of __________________, 2024.

By: ______________________________________
Signature

____
Title

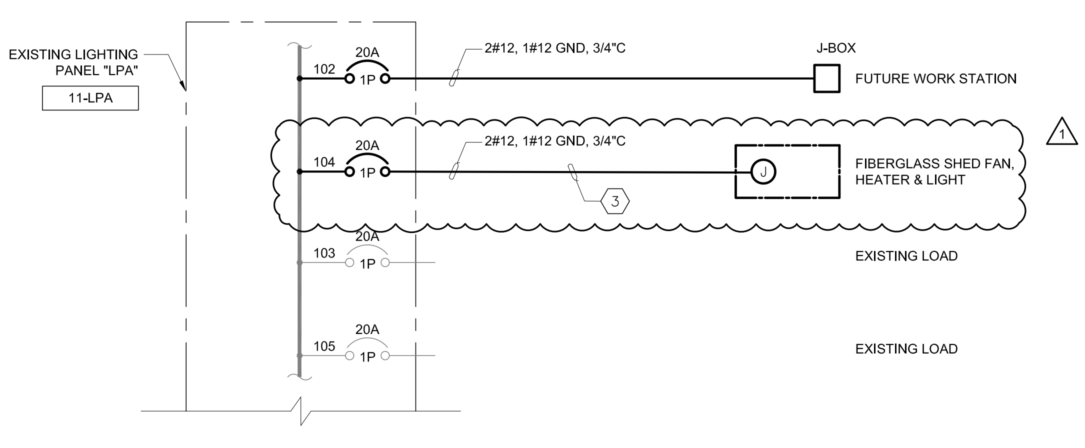
____
Company

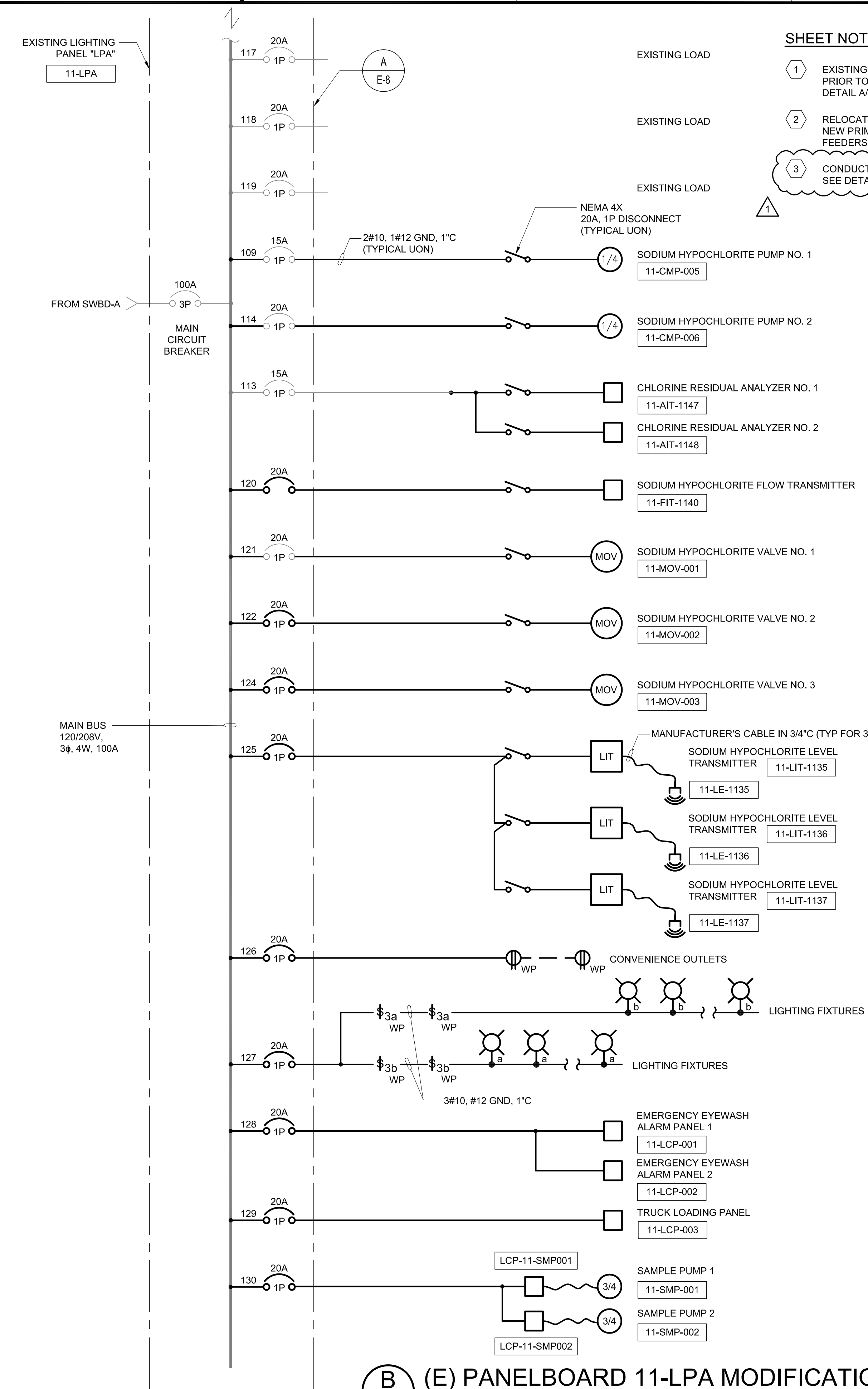
ATTACHMENT A



A WEPS MAIN SINGLE-LINE DIAGRAM

SCALE: NTS





B (E) PANELBOARD 11-LPA MODIFICATIONS

SCALE: NTS

SHEET NOTES:

1. EXISTING SUBMERSIBLE MIXER SHALL BE DISCONNECTED AND REMOVED PRIOR TO INSTALLATION AND CONNECTION OF THE NEW MIXER. SEE DETAIL A/E-10 FOR LOCATION. SEE SPECS FOR PHASING REQUIREMENTS.
2. RELOCATE TRANSFORMER AS PER DETAIL F/E-7. FURNISH AND INSTALL NEW PRIMARY AND SECONDARY FEEDERS AFTER REMOVAL OF EXISTING FEEDERS.
3. CONDUCTORS ARE INSTALLED IN NEW AND EXISTING CONDUIT SEGMENTS. SEE DETAIL A/E-10.

1 ADDENDUM 1 7/2/2024

ENGINEERS, INC.

DATE: 04/30/2024
DESIGNED BY: DTN
DRAWN BY: TP
CHECKED BY: DN

PROJECT NO. C66501840

CITY OF PETALUMA
PUBLIC WORKS & UTILITIES
202 N. McDowell Blvd., PETALUMA, CALIFORNIA, 94954
PH. 707-778-4546 FAX. 707-778-4508

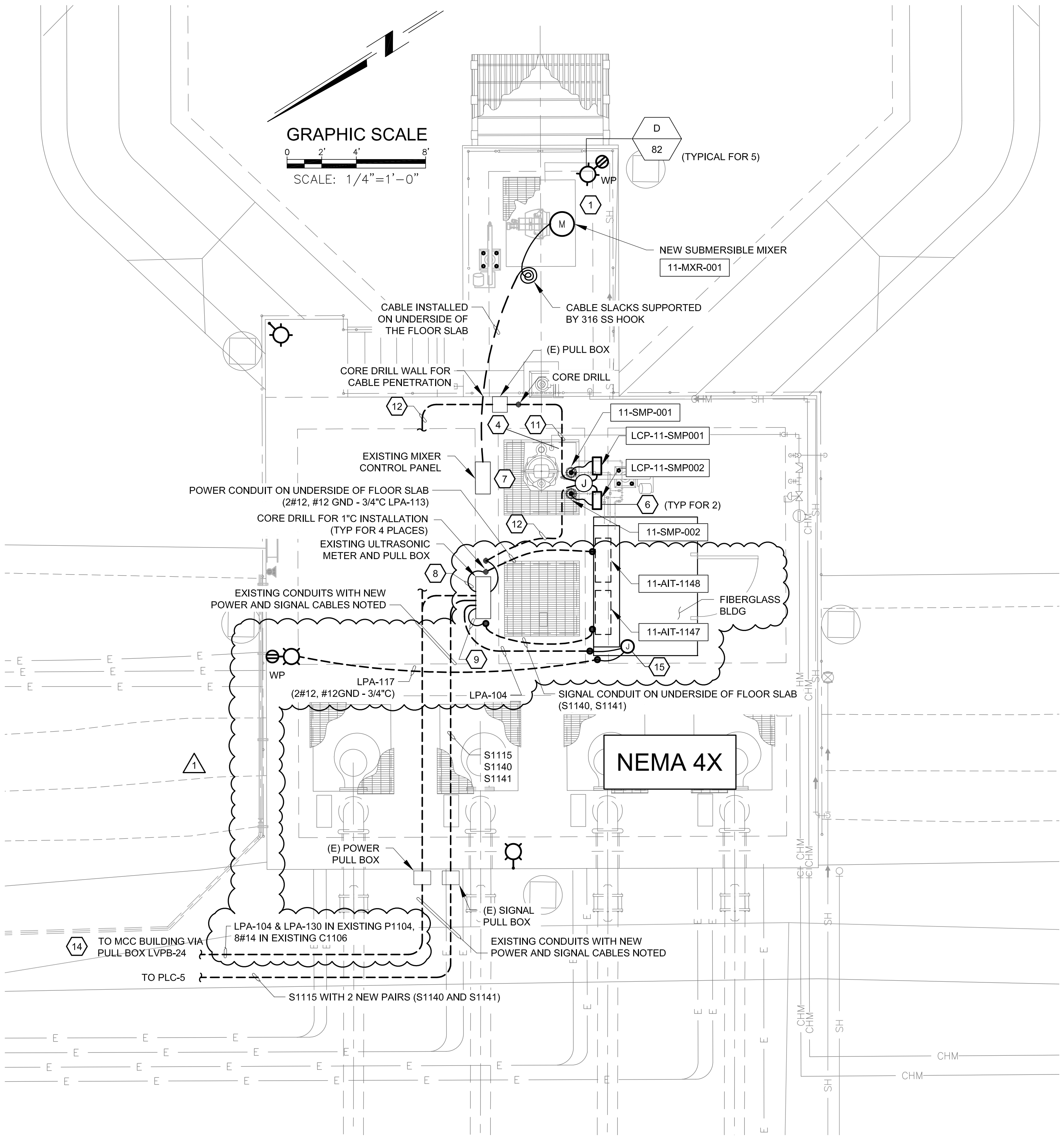
SODIUM HYPOCHLORITE SYSTEM REPLACEMENT AND RELOCATION

ELECTRICAL SINGLE-LINE DIAGRAMS

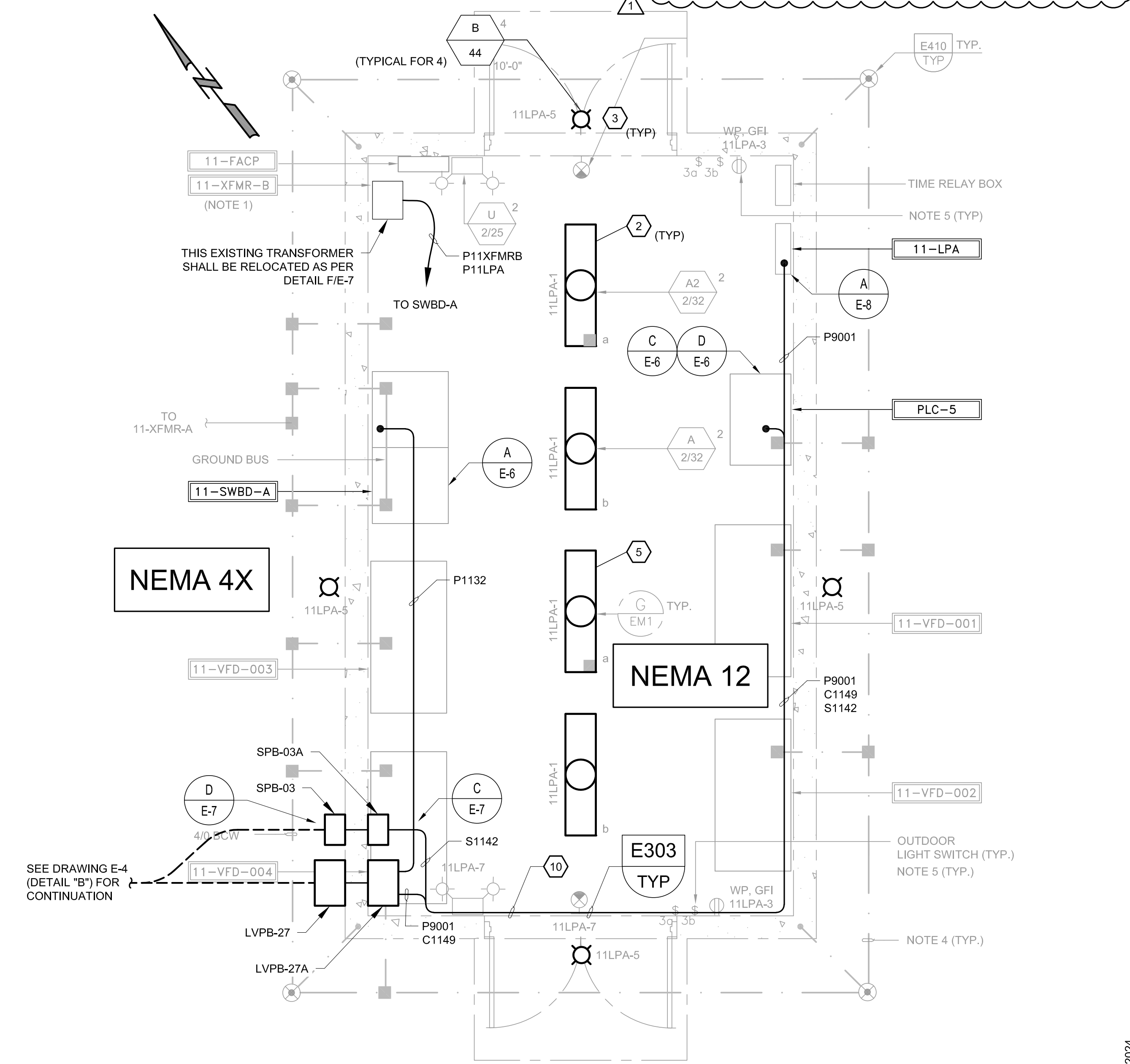
SHEET E-2

SHEET NOTES:

1. SEE NOTE 3 ON DRAWING E-11.
2. SEE NOTE 2 ON DRAWING E-11.
3. SEE NOTE 1 ON DRAWING E-11.
4. DISCONNECT AND REMOVE EXISTING SUBMERSIBLE MIXER FROM EXISTING CONTROL PANEL.
5. FOR THIS SPECIFIC LIGHT FIXTURE, REPLACE EXISTING T8 TUBE WITH STANDARD LED TUBE. DO NOT BYPASS NOR REWIRE EXISTING BALLAST WITH BATTERY BACKUP.
6. SEE TYPICAL DETAIL E331 AND DETAIL B/E-12.
7. DISCONNECT AND REMOVE EXISTING SUBMERSIBLE MIXER CABLE AND MIXER PRIOR TO INSTALLATION OF NEW FEEDER CABLE FOR NEW MIXER.
8. CONNECT TO EXISTING 120V POWER CIRCUIT TO FEED NEW ANALYZERS.
9. ROUTE 2 PAIRS IN EXISTING SIGNAL CONDUIT S1115 TO PLC-5.
10. ROUTE CONDUITS ALONG WALL AT 11 FT ABOVE FINISHED FLOOR TO AVOID CONFLICT WITH EXISTING DOORS AND EXIT SIGN.
11. LPA-130 (2#12, #12 GND - 3/4"C) MOUNTED ON UNDERSIDE OF FLOOR SLAB.
12. LPA-130 IN EXISTING P1104
13. 8#14 - 3/4"C (C1106C)
14. THE DISTANCE FROM THIS LOCATION TO MCC BUILDING VIA LOW VOLTAGE PULL BOX LVPB-24 IS APPROXIMATELY 120 FEET. SEE DETAIL B/E-4 FOR LOCATIONS OF PANEL LPA AND PLC5.
15. J-BOX FOR:
    a) LIGHT, FAN, HEATER LPA-104
    b) GFI OUTLET LPA-117



A — POWER, SIGNAL AND LIGHTING PLAN (NEMA 4X)
SCALE: 1/4" = 1'-0"



B — LIGHTING, SIGNAL AND POWER PLAN - "11-SWBD-A" BLDG
SCALE: 3/8" = 1'-0"

1 ADDENDUM 1 7/2/2024

DTN ENGINEERS, INC.

DATE: 04/30/2024
DESIGNED BY: DTN
DRAWN BY: TP
CHECKED BY: DN

PROJECT NO. C66501840

CITY OF PETALUMA
PUBLIC WORKS & UTILITIES
202 N. McDowell Blvd., PETALUMA, CALIFORNIA, 94954
PH. 707-778-4546 FAX. 707-778-4508

SODIUM HYPOCHLORITE SYSTEM REPLACEMENT AND RELOCATION
ELECTRICAL
LIGHTING IMPROVEMENT SHEET 1

SHEET E-10

3/4"x22" ORIGINAL SCALE IN INCHES

P:\482_Petaluma Chemical System Upgrade_Dude\DRAWINGS\482_E10 Addendum 1.dwg 07/02/2024 09:33

ATTACHMENT B

BID SCHEDULE

Item No.	Description	Estimated Quantity	Unit	Unit Price	Total Price
1	Mobilization/Demobilization	1	LS		
2	Starup and Testing	1	LS		
3	Tree Removal	1	LS		
4	Dewatering	1	LS		
5	Concrete Pavement	1	LS		
6	AC Pavement				
7	Culvert Pipe		LS		
8	Earthwork Excavation				
9	Pipe Bedding Materials				
10	Concrete Mat Slab				
11	Concrete Containment Wall				
12	Concrete Column Pedestal				
13	Chemical Tank Pad, Sump, and Pump Pads				
14	CIDH Piles				
15	Concrete Stairs				
16	FRP Stairs				
17	Blue-White Skid System				
18	Blue-White Peristaltic Pump				
19	Chemical Storage Tank				
20	Tankless Water Heater				
21	Shower/Eyewash				
22	Xylem (Flygt) Mixer, Mixer Mounting System				
23	3” PVC Pipe & Fitting				
24	4” DI Pipe & Fitting				
25	Chemical Piping				
26	Piping Supports				
27	Electrical Components and Installation		LS		
28	Instrumentation and Controls Components and Installation		LS		
Total Base Bid	**$**	**$**			

Item No.	Description	Estimated Quantity	Unit	Unit Price	Total Price

<u>OPTIONAL BID ITEMS</u>

Item No.	Description	Estimated Quantity	Unit	Unit Price	Total Price

***Note:** In case of error in extension of price into the total price column, the unit price will govern.

Total Amount of Bid (written in words) is:____

____ Dollars and

____ Cents.

In the event of discrepancy between words and figures, the words shall prevail.

$________________________

Figures

The award of the contract shall be awarded to the lowest price of the total of Base Bid items 1 through 6. Options Bid items should NOT be included in the Base Bid Price.

____________________________________ Address of Bidder

____________________________________ Signature of Bidder

____________________________________ City

____________________________________ Name of Bidder (Print)

____________________________________ Telephone Number of Bidder

____________________________________ Fax Number of Bidder

____________________________________ Contractor's License Number

____________________________________ License's Expiration Date

Addendum Acknowledgement

Addendum No. 1 Signature Acknowledging Receipt:______________________ Date:______________________

Addendum No. 2 Signature Acknowledging Receipt:______________________ Date:______________________

Addendum No. 3 Signature Acknowledging Receipt:______________________ Date:______________________

Addendum No. 4 Signature Acknowledging Receipt:______________________ Date:______________________

Addendum No. 5 Signature Acknowledging Receipt:______________________ Date:______________________

Addendum No. 6 Signature Acknowledging Receipt:______________________ Date:______________________

Addendum No. 7 Signature Acknowledging Receipt:______________________ Date:______________________

Addendum No. 8 Signature Acknowledging Receipt:______________________ Date:______________________

SECTION 13140

FIBERGLASS REINFORCED PLASTIC BUILDINGS

PART 1 - GENERAL

1.01 SUMMARY

A. Section Includes:

1. Freestanding, shop fabricated and assembled fiberglass reinforced plastic (FRP) insulated composite buildings/sheds/huts.
2. Include fasteners, anchors, doors and frames, vents, gasketing, lighting, and ventilation fan.

1.02 SYSTEM DESCRIPTION

A. Design Requirements:

1. Building shall conform to dimensions shown on Drawings or Buildings shall have outside dimensions of 8 ft wide by 6 ft deep with a height of 8 ft.
2. Building shall be water resistant, corrosion and chemical resistant, lightweight, and environmentally aesthetic. Filtered louvers shall be provided
3. Building shall be equipped with adequate interior lighting to allow for operator's to work on sampling and controls equipment in the fully-enclosed building. Light switch shall be mounted on same j-box with exhaust fan ON/OFF switch (NEMA 4X) installed on the exterior.
4. Building shall be equipped with one 120VAC GFI convenience receptacle.
5. Building shall be equipped with a single, standard sized door for access that allows for locking and unlocking with standard treatment facility keys.
6. Design to sustain superimposed loads for load combinations in accordance with ASCE 7-98.
   a. Design loads:
      (1) Dead load of building, live (snow) load, 35 psf, wind load, 25 psf, mechanical equipment.
   b. During installation of the composite FRP structure a concentrated load not exceeding 250 pounds may be placed on any portion of the roof. The concentrated load shall not be applied to the roof if other loads are present.
7. Stresses produced by specified load conditions shall be determined consistent with recognized methods of analysis.
8. Average R-value of assembled building shall be minimum of R-7.
9. Provide 800W 120VAC electric heater with adjustable thermostat type settings.

1.03 SUBMITTALS

A. Product Data:

1. Resin and glass manufacturers material specifications.

B. Shop Drawings:

1. Include plans and elevations, fabrication details indicating laminate thickness and section depths and widths, location of openings and equipment supports, size and location of anchor bolts, and gasketing details.

C. Submit in accordance with Section 01330.

1.04 QUALITY ASSURANCE

A. Buildings provided shall be end product of one manufacturer to achieve standardization for appearance. Manufacturer Qualifications: Building shall be manufactured and erected by firm with minimum of 5 yrs experience in structures of size and character specified. Provide 20-year warranty on materials and workmanship for the building.

1.05 DELIVERY, STORAGE, AND HANDLING

A. Store and protect on manufacturer's site, project site and during shipment and installation to prevent warping and fracturing.

PART 2 - PRODUCTS

2.01 MANUFACTURER

A. Mekco, Shelter Works, or equal

2.02 LAMINATE MATERIALS

A. Resins, Gel Coat, Glass Reinforcing, Insulation.

2.03 MISCELLANEOUS MATERIALS

A. Concrete Anchors, Doors, Gasketing

B. Permanently fused building assembly yielding a watertight one-piece structure.

2.04 FABRICATION

A. Form individual segments on high gloss or matte molds ensuring consistent dimensions of finished parts. Cast each segment in one piece. Laminate shall consist of chopped roving impregnated with resin. Form panel flanges and perimeter anchoring flanges to the interior of the building.

B. Exterior color of the building shall be light gray. Interior color shall be off-white.

2.05 ASSEMBLY

A. Shop assemble complete building. Flanges between adjacent panels shall be factory bonded together with structural adhesive. Seal exterior edges of adjacent panels with color matched silicon sealant. Fit and bond appurtenances, formed separately, into openings cut in finished panel or integrally mold to panel. Bond attachments with glass fibers and resin from interior of panel. Resin seal cut or drilled edges.

PART 3 - EXECUTION

3.01 EXAMINATION

A. Examine surface to receive building for acceptable installation conditions. Do not start installation unless acceptable conditions are provided.

3.02 INSTALLATION

A. Install in accordance with manufacturer’s instruction and approved submittals.

B. Install continuous neoprene gasket or ConSeal between perimeter anchoring flange and where panels rest on supporting structure. Resin seal cut or drilled edges. Repair damaged panels. Minimum spacing and edge distances of concrete anchors shall conform to requirements of Section 05500.

C. All wirings within the enclosure shall be in PVC schedule 40 conduits. Electrical wirings shall be terminated in a common NEMA 4X fiberglass J-box for connections to power 120V circuits by the Contractor.

ATTACHMENT C

GEOTECHNICAL INVESTIGATION REPORT
CHEMICAL SYSTEM UPGRADE
ELLIS CREEK WATER RECYCLING FACILITY
PETALUMA, CALIFORNIA

BSK PROJECT NO.: G00000357

PREPARED FOR:

DUDEK
1630 SAN PABLO AVENUE, SUITE 300
OAKLAND, CALIFORNIA 94612

June 27, 2023

DUDEK PETALUMA SONOMA COUNTY CALIFORNIA-USA

399 Lindbergh Avenue
Livermore CA 94551
P 925.315.3151
www.bskassociates.com

Sent via email: pgiori@dudek.com

June 27, 2023 BSK Proposal No. G00000357

Mr. Phillip Giori, PE
Dudek
1630 San Pablo Avenue, Suite 300
Oakland, California 94612

SUBJECT: **Geotechnical Investigation Report**
Chemical System Upgrade
Ellis Creek Water Recycling Facility
Petaluma, California

Dear Mr. Giori:

BSK Associates (BSK) is pleased to submit our geotechnical investigation report for the above-referenced project at the City of Petaluma (City) Ellis Creek Water Recycling Facility located at 3890 Cypress Drive in Petaluma, California. The enclosed report presents our recent geotechnical investigation performed within the limits of the Chemical System Upgrade project, and our conclusions and geotechnical design recommendations for the project. Note that our recent geotechnical investigation was also performed for the separate Oxidation Pond Transfer Structure Rehabilitation and Oxidation Pond Storage Expansion project which is presented in Appendix A of this report.

In summary, it is our opinion that the project sites (Sites) for the Chemical System Upgrade project are suitable for the proposed improvements provided the recommendations presented in this report are incorporated in the design and construction of the project. The primary geotechnical concerns at the Sites are the potential for strong ground shaking to affect the Sites during a future significant seismic event (typical of California), the presence of shallow groundwater and associated hydrostatic and buoyancy pressures, the presence of highly compressible Bay Mud and high organic content soils containing peat, and the presence of highly expansive surficial soils. The impact of these concerns on the project and ways to design for and/or mitigate them are discussed in the report.

The conclusions and recommendations presented in the enclosed report are based on limited subsurface investigation and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil conditions may be found in localized areas during construction. If significant variation in the subsurface conditions is encountered during construction, BSK should review the recommendations presented herein and provide supplemental recommendations if necessary.

Additionally, design plans should be reviewed by our office prior to their issuance for conformance with the general intent of our recommendations presented in the enclosed report.

We appreciate the opportunity of providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please contact us at (925) 315-3151.

Sincerely,
BSK Associates, Inc.

REGISTERED PROFESSIONAL ENGINEER
CRISTIANO B.C. DEMELO
No. 2756
Exp. 09/30/24
GEOTECHNICAL
STATE OF CALIFORNIA

Cristiano Melo, PE, GE #2756
Livermore Branch Manager

Carrie L. Foulk, PE, GE #3016
Geotechnical Group Manager

Table of Contents

Figures and Appendices

FIGURES

1. INTRODUCTION

This report presents the results of our recent geotechnical investigation for the planned Chemical System Upgrade (CSU) project at the Ellis Creek Water Recycling Facility (ECWRF) located at 3890 Cypress Drive in Petaluma, California. Our recent geotechnical investigation was also performed for the separate Oxidation Pond Transfer Structure Rehabilitation and Oxidation Pond Storage Expansion project, which is presented in Appendix A of this report. A Vicinity Map showing the location of the project sites (Sites) for the CSU project is presented on Figure 1. This report contains a brief description of our site investigation methods and findings for the Sites, including field and laboratory data. Based on these findings, this report presents conclusions regarding the geotechnical concerns for the planned improvements.

1.1 Site Description

The ECWRF is located at the southern end of Petaluma along the southwest side of Lakeville Highway (Highway 116) and Browns Lane within the floodplain of the Petaluma River. As shown on the Site Plans, Figures 2 through 4, the main facilities for the CSU project are located within the oxidation ponds situated at the southeastern area of the ECWRF. These facilities consist of the Wetlands Effluent Pump Station (WEPS) adjacent to the northwest side of Pond No. 9 and the chemical processing area adjacent to the south corner of Pond No. 10. The WEPS facility and the chemical processing area sit atop levee embankments that are about 30 to 50 years old. The levee embankment for the WEPS is about 100 to 200 feet long, by about 65 feet wide, by about 5 feet high, and has slope gradients of about 5H:1V (horizontal to vertical) to 3H:1V. The levee embankment for the chemical processing area is about 190 feet long, by about 140 feet wide, by about 7 feet high, and has slope gradients of about 7H:1V to 3H:1V. The top of the levees acts as vehicular pathways in between the oxidation ponds and are lined with aggregate base and/or dirt, except for the asphalt paved roadway connecting the chemical processing area to Highway 116 along the southeastern side of Ponds No. 2, 3, 6, 7, and 10. Based on the current (undated) elevation topographic map of the Sites provided to us by Dudek (the lead designer for this project), the elevations at the top of the levee embankment for the WEPS facility and the chemical processing area range from about 14 to 15 feet and 12 to 13 feet, respectively.

According to historic aerial photographs and historic topographic maps, the Site area was originally a marsh land/floodplain associated with the Petaluma River until about 1947. By 1955, the area was used for agriculture until about 1975. According to the geotechnical report by Fugro dated April 2005 (see detailed reference in the "Previous Investigations" section below), the oxidation ponds were constructed in 1972 by a combination of excavating and placing fill over the native alluvial and marsh deposits.

1.2 Project Description

The City of Petaluma (City) intends to upgrade the existing chemical system that was constructed in the 1970s with some upgrades in the 1990s to comply with current regulations and safety standards while also improving efficiency. The existing chemical system was part of the previous wastewater treatment system prior to ECWRF operations. The main chemical processing area is in the southern corner of the

ECWRF and consists of sodium hypochlorite (hypochlorite) tanks, sodium bisulfite (bisulfite) tanks, chemical pumps, and the Motor Control Center (MCC) to run and monitor the equipment. The hypochlorite travels through 2,400 feet of parallel pipes to the Wetlands Effluent Pump Station (WEPS). These pipelines are occasionally affected by gas bubbles that disrupt the treatment process, and the long length of the lines makes them difficult to repair and replace. Additionally, one of the pipes has failed, leaving the hypochlorite system with no redundancy.

Due to the age of the infrastructure and to prioritize regaining redundancy, the project has been divided into two phases. The **first phase** will relocate and rebuild a portion of the hypochlorite dosing system from its current location to the WEPS location. The **second phase** of the project will upgrade structural, mechanical, and electrical deficiencies at the chemical processing area, and will include other demolition and reconstruction activities.

Phase one will replace and relocate components of the hypochlorite system to the WEPS location. The new location will include three 6,500-gallon hypochlorite tanks with secondary containment, two chemical pumps, all associated electrical and SCADA monitoring equipment, a potable water tank and associated pressure system, and an emergency shower/eyewash station. The tanks and pumps will be located outside under a shade cover (shelter structure) and the electrical and SCADA equipment will likely be installed in the existing WEPS motor control center. The shelter structure for the new tanks and associated equipment will be approximately 60 feet in length and 30 feet in width (see Figure 3). Depending on the settlement constraints, the shelter structure will likely be supported on a mat foundation, cast-in-drilled-hole (CIDH) piers, a combination of both, or a combination of a mat foundation and ground improvement. To provide a level area for the three new tanks, either a retaining wall will need to be constructed or a portion of the levee embankment housing the WEPS will need to be widened. This would require placing up to about 3½ feet of new fill adjacent to the northwest side of the existing levee embankment. The widened portion of the levee embankment would need to be steepened from its current slope gradient of about a 3H:1V to a steeper slope gradient of about 2H:1V. Additionally, road improvements consisting of asphalt concrete and Portland Cement Concrete pavement will be constructed to provide all-weather access for chemical trucks (representative of HS-20 live load) from Highway 116 to the new sodium hypochlorite storage tanks and water to wash down equipment. The planned improvements at the WEPS facilities are depicted in the draft drawings presented in Appendix B of this report.

Phase two will consist of evaluating whether to upgrade and retrofit the existing facilities at the chemical processing area or construct new facilities (see Figure 4). The evaluation will include as-needed seismic and electrical improvements, mitigating risks from rising sea levels, and standby power. The chemical processing area consists of a hypochlorite tank building, a bisulfite tank building, an office building (Ponds Office), an MCC for the chemical dosing system, and a chemical pump room. Additionally, phase two will include improvements to the Chlorine Contact Basin as well as demolition of the existing Main Pump Station and Control Building. We understand that consideration is being given to demolishing the hypochlorite tank building, bisulfite tank building, office building, MCC for the chemical dosing system, and the chemical pump room. If this is the case, the new building(s) would be constructed with the existing footprint of the existing buildings. Depending on the settlement constraints, the new structures will likely be supported on continuous and isolated spread footings, mat foundations, cast-in-drilled-hole (CIDH) piers, or a combination of shallow foundations and ground improvement.

Similar to the separate Oxidation Pond Storage Expansion project, the planned improvements for the CSU may also include raising the levee embankments for the WEPS and chemical processing area about 2 to 3 feet in vertical height in order to address rising sea levels.

If the actual project description differs significantly from that anticipated above, we should be notified so that we may review our proposed scope of services presented herein for applicability.

1.3 Purpose and Scope of Services

The purpose of this investigation was to explore and evaluate the subsurface conditions at the Sites for the CSU project to provide geotechnical input for the design and construction of the planned improvements. The scope of services, as outlined in our January 19, 2023 proposal (Proposal No. G00000357), consisted of administration of BSK's services, additional subsurface investigation and laboratory testing beyond what was originally planned for the Oxidation Pond Transfer Structure Rehabilitation and Oxidation Pond Storage Expansion project, engineering analysis, and preparation of this report.

Our investigation specifically excludes the assessment of site environmental characteristics, particularly those involving hazardous substances. Our scope of services did not include the evaluation of contaminants in the soil, water, or air.

1.4 Previous Investigations

Previous investigations were performed within the levees for the oxidation ponds at the ECWRF by other subconsultants. These investigations were presented in the following documents:

1. Fugro West, Inc. (Fugro, 2005), Integrated Geotechnical Study, Lakeville Highway WRF – Parcel A, Petaluma, California, dated April 29, 2005 (Fugro West Project No. 3045.022). **This report included numerous previous exploration points performed by Harza in 2001 (Harza was acquired by Fugro in the early 2000's) as well as tabulated logs for borings performed in 1971 and 1995 by Moore and Taber and Harding Lawson**, and
2. RGH Consultants (RGH, 2012), Limited Geotechnical Study, Ellis Creek Oxidation Ponds 7 and 10, Sheet Pile Levee Project, Petaluma, California, dated December 4, 2012 (RGH Consultants Project No. 2553.08.04.1).

Pertinent information from these previous reports was considered in the preparation of this report. Available boring logs and lab data from these previous investigations that are proximate to the Site are included in Appendix A. The approximate locations of the previous exploration points are shown on Figure 2 and Figure 4.

2. SITE INVESTIGATION

2.1 Field Investigation

Please refer to the "Field Investigation" section of BSK's June 26, 2023 report in Appendix A for discussion on the field investigation performed for this project.

2.2 Laboratory Testing

Please refer to the "Laboratory Testing" section of BSK's June 26, 2023 report in Appendix A for discussion on the laboratory testing program performed for this project.

3. SITE GEOLOGY AND SEISMICITY

Please refer to the “Site Geology and Seismicity” section of BSK’s June 26, 2023 report in Appendix A for discussion on the geology and seismicity for the site area for this project.

4 SUBSURFACE CONDITIONS

4.1 Current Subsurface Data

Below is a general description of the soil conditions encountered at the Sites for the CSU project. For a more detailed description of the soils encountered, refer to the current boring logs, current CPT logs, and previous subsurface data presented in Appendix A. It should be noted that subsurface conditions can deviate from those conditions encountered in the current and previous investigations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments, as necessary.

According to our current borings and CPTs, the Sites are underlain by levee fill and native soils. The fill is present in the upper approximately 8 to 15 feet below the existing ground surface (BGS)[1] and generally consists of firm to hard lean and fat clays. A layer of very loose clayey sand was encountered at a depth of approximately 5 feet BGS in boring B-3.

Immediately beneath the fill, approximately 6 to 10 feet of soft to firm Bay Mud consisting primarily of lean and fat clay was encountered in borings B-3 and B-5. Bay Mud is highly compressible and susceptible to high long-term consolidation settlement upon loading. Based on our pocket penetrometer and TXUU test results, the upper portion of the Bay Mud layer has a higher shear strength than the lower portion of the Bay Mud Layer. This is attributed to desiccation of the Bay Mud due to repeated cycles of rising and falling groundwater in marsh lands and exposure to sunshine and wind. As a result, the upper portion of the Bay Mud layer is commonly referred to as “Bay Mud Crust”, which typically has significantly higher strength than regular Bay Mud and is less susceptible to high consolidation. According to NAVFAC 7.01[2], soils having an organic content by weight of less than 5 percent are slightly organic, while soils having an organic content between 5 and 30 percent are considered to be organic soils. Soils having an organic content of over 30 percent are considered as highly organic and classified as peat. A peat layer within the Bay Mud layer was encountered in boring B-5 with an organic content of approximately 37 percent from a depth of about 14½ to 16½ feet BGS. Bay Mud was also encountered in CPT-2 from about 10 to 19 feet BGS and CPT-5 from about 12 to 21 feet BGS.

Below the Bay Mud layer in borings B-3 and B-5, our borings generally encountered firm to hard lean and fat clays to the maximum depth of our borings (approximately 31½ BGS). A layer of loose clayey sand was encountered at a depth of approximately 29 feet BGS in boring B-3. Below the levee fill and the Bay Mud layer, our CPTs generally encountered firm to hard clayey soils to the maximum depth of our CPTs (approximately 50 feet BGS).

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[1] Any reference made to “below the existing ground surface (BGS)” throughout this report refers to the ground surface at the crest of the existing levees for the oxidation ponds.

[2] Naval Facilities Engineering Command (NAVFAC), Design Manual 7.01, Revalidated by Change 1 September 1986.

4.2 Previous Subsurface Data

As shown on Figure 2, various previous exploration points consisting of borings and CPTs were performed by other consultants along the ponds. Within the Sites for the CSU project, boring HB-8 and CPT-2 were performed by Harding Lawson in 1995 and Harza in 2001, respectively. Boring HB-8 encountered primarily medium stiff (soft) to very stiff (firm to hard) clayey soils interbedded with loose to dense layers of clayey sand to the maximum depth explored (approximately 50 feet BGS). Based on our interpretation, boring HB-8 encountered approximately 9½ feet of levee fill underlain by approximately 9 feet of Bay Mud. CPT-2 encountered primarily firm to hard layers clayey soils to the maximum depth explored (approximately 80 feet BGS).

4.3 Groundwater

Free groundwater was observed at depths of approximately 10 and 25 feet BGS in borings B-3 and B-5, respectively. Based on pore pressure dissipation tests, groundwater was encountered at approximately 7 feet BGS in CPT-3. According to piezometer monitoring conducted by Fugro/Harza from 2001 to 2004 (refer to Plate 8 in Appendix A), the groundwater level at the Sites for the CSU project was about 8 feet BGS.

It should be noted that groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties. Also, free groundwater levels shown in the current and previous exploration points may not be representative of stabilized groundwater levels.

4.4 Additional Discussion

The above is a general description of soil and groundwater conditions encountered at the Sites for the CSU project. For a more detailed description of the soils encountered, refer to the boring and CPT logs in Appendix A. It should be noted that subsurface conditions can deviate from those conditions encountered at the boring and CPT locations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments as necessary.

5. DISCUSSIONS AND CONCLUSIONS

Based on the results of our investigation, it is our opinion that the proposed improvements are feasible geotechnically. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated into the design and construction of this project. The primary geotechnical concerns for the Site are:

1. The potential for strong ground shaking to affect the Site during a future significant seismic event (typical of the entire San Francisco Bay Area). Ground shaking can be addressed by incorporating the seismic design parameters presented herein and other seismically related aspects of the 2022 California Building Code (CBC) into the design of the project.
2. The presence of shallow groundwater and associated hydrostatic and buoyancy pressures.
3. The presence of highly compressible Bay Mud and high organic content soils containing peat underneath the Site and associated potential for significant long-term settlement.
4. The presence of highly expansive surficial soils, which can be addressed by providing deeper embedment depth of shallow foundations, a continuous perimeter shallow foundation around structures, use of "non-expansive" fill underneath slabs-on-grade, and proper moisture conditioning of subgrade soils.

Additional discussions of the conclusions drawn from our investigation, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in the "Recommendations" section of this report.

5.1 Shallow Groundwater

As discussed in the "Subsurface Conditions" of this report, free groundwater was observed at depths ranging from about 10 to 25 feet BGS within the current exploration points performed at the Sites for the CSU project. However, the actual depth at which groundwater may be encountered in trenches and excavations may vary. Therefore, excavations deeper than about 5 feet BGS will likely require dewatering during construction. In addition, the design of new below-grade improvements will need to consider buoyancy forces. We recommend using a design groundwater depth of 5 feet BGS for the design of buoyancy forces. **As previously discussed, any reference made to "below the existing ground surface (BGS)" throughout this report refers to the ground surface at the crest of the existing levees for the oxidation ponds.**

We assume pertinent oxidation ponds will be drained where the planned improvements extend down into the ponds. Due to the predominantly clayey nature of the levee fill and the underlying Bay Mud layer, we expect that dewatering can be conducted primarily via use of sumps and open pumping. However, the contractor should be responsible for the means and methods for dewatering the Sites for the CSU project provided the design/construction management team is afforded an opportunity to review and comment on the proposed dewatering method(s) to be used.

5.2 Existing Levee Fill

Based on the findings from the current and previous investigations, the existing levee fill appears to consist of properly engineered fill. The fill generally has consistent and adequate strength based on laboratory strength testing and pocket penetrometer readings, the fill has consistent and adequate dry density and moisture content based on test results, and the fill has consistent and high blow counts. The existing fill also appears to be free of debris, deleterious matter, and organics based on the current and previous borings. Therefore, other than having to scarify the crest of the levees during placement of fill as discussed later in this report, there is no need to overexcavate and replace or recompact the existing levee fill.

In December of 2021, a sinkhole was discovered on the levee roadway between the flow transfer structure on the aerated lagoon and Pond No. 4. The sinkhole appears to have been the result of corrosion of a corrugated metal pipe. We understand that another sinkhole has been identified more recently, which is located on the inboard side of the levee for Pond No. 9 just south of the WEPS facility. Consideration should be given to performing a video survey of existing underground utilities throughout the Sites for the CSU project to check the integrity of existing pipelines. Consideration should also be given to performing a geophysical survey of the levees to check for potential voids withing the levees that could lead to future sinkholes.

5.3 Impact of Bay Mud on the Site's Development

Based on our interpretation of the current and previous subsurface data presented in Appendix A, the Sites for the CSU project are underlain by Bay Mud as shown on Figure 2. As previously mentioned, Bay Mud is susceptible to high long-term consolidation settlement upon loading. The Bay Mud thickness ranges from about 6 to 10 feet within the vicinity of Ponds No. 9 and 10. Based on our findings, the upper half of the Bay Mud layer consists of a higher strength "crust" that is less susceptible to higher consolidation settlement than the lower half.

5.3.1 Long-Term Consolidation Settlement

Once new fill is placed to raise the pond levees, the levee for the WEPS facility, and the levee for the chemical processing area, it will trigger long-term consolidation settlement of the underlying Bay Mud layer. To help us evaluate potential consolidation settlement if 2 to 3 feet of fill is placed over the existing oxidation pond levees, we ran consolidation testing on a sample collected at a depth of approximately 16 feet BGS at boring B-3. We also ran consolidation settlement analyses using the program Settle3D (Version 2.016) for:

1. A generic levee cross section,
2. The chemical processing area,
3. The proposed fill underneath the planned shelter structure at the WEPs facility,
4. The proposed mat foundation for the planned shelter structure at the WEPs facility, and

5. Various shallow foundation configurations at the chemical processing area.

The results of our analyses as well as the assumed geometry and geotechnical parameters used in the analyses are provided in the sections below.

5.3.1.1 Generic Levee Cross Section

We evaluated the long-term consolidation settlement for a generic levee cross section using the following parameters/assumptions in our analysis:

- 2 to 3 feet of new fill will be added to the crest of the levee.
- The levee is 20 feet wide at the crest.
- The levee has side slopes with gradients of 3H:1V.
- The existing levee fill is 10 feet thick.
- The existing levee fill has been in place for 50 years (i.e., since circa 1972).
- The Bay Mud layer underlying the underlying the existing levee fill is 10 feet thick.

Based on our analyses, and the stated assumptions, we estimate total long-term consolidation settlement of **about 3 to 6 inches.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill. This settlement will occur areawide and should have higher magnitude where the Bay Mud layer is thicker and lower magnitude where the Bay Mud layer is thinner.

5.3.1.2 Chemical Processing Area

We evaluated the long-term consolidation settlement for the chemical processing area (CPA) using the following parameters/assumptions in our analysis:

- 2 to 3 feet of new fill will be added to the crest of the CPA levee.
- The CPA levee is 190 feet long by 140 feet wide at the crest.
- The CPA levee has side slopes with gradients of 5H:1V.
- The CPA existing levee fill is 7 feet thick.
- The CPA existing levee fill has been in place for at least 30 years.
- The Bay Mud layer underlying the underlying the CPA existing levee fill is 10 feet thick.

Based on our analyses, and the stated assumptions, we estimate total long-term consolidation settlement of **about 3 to 6 inches.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill.

5.3.1.3 Proposed WEPS Shelter Structure

We evaluated the long-term consolidation settlement of the proposed WEPS shelter structure using the following parameters/assumptions in our analysis:

- 3.5 feet of new fill will be placed to widen the levee embankment.
- The WEPS levee has side slopes with gradients of 5H:1V.
- The WEPS existing levee fill is 5 feet thick.
- The WEPS existing levee fill has been in place for at least 50 years.
- The Bay Mud layer underlying the underlying the WEPS existing levee fill is 10 feet thick.
- The proposed mat foundation will be 62 feet long by 28 feet wide and will be located as approximately shown on Figure 3 and in Appendix B.
- We assume allowable bearing pressures of 500 and 1,000 psf for the mat foundation.

Based on our analyses, and the stated assumptions, we estimate total long-term consolidation settlement ranging from **about 5 to 9 inches** where the mat foundation will lie on the existing levee fill and from about **2 to 2½ feet** where the mat foundation will lie directly on the new fill to be placed to widen the levee embankment. **This could result in a differential settlement of about 2 feet across the span of the mat foundation.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill and constructing the shelter structure.

5.3.1.4 Proposed Shallow Foundations for Chemical Processing Area

Except for the long-term settlement discussed above for new fill placed over the chemical processing area, we anticipate little to no long-term settlement if new shallow foundations are constructed within the limits of existing shallow foundations that are demolished/removed provided similar loading is applied to the new foundations. However, where this is not the case, we have analyzed the long-term settlement of adding new mat foundations and continuous and isolated spread footings over the chemical processing area using the same parameters/assumptions listed for the "Chemical Processing Area" above and the allowable bearing pressure discussed below. The results of our analyses are presented below.

5.3.1.4.1 Mat Foundations

Based on our analyses, allowable bearing pressures of 500 and 1,000 psf, and mat foundations 10 to 20 feet long by 5 to 10 feet wide, we estimate total long-term consolidation settlement of **about 2 to 8 inches, with differential settlement equal to half of this amount distributed across the span of the mat foundations.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill.

5.3.1.4.2 Isolated Spread Footings

Based on our analyses, allowable bearing pressures of 1,000 to 2,500 psf, and 4-foot square footings embedded 2 feet BGS, we estimate total long-term consolidation settlement of **about 2 to 5 inches, with differential settlement equal to half of this amount between adjacent foundation support or across a horizontal distance of approximately 30 feet, whichever is less.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill.

5.3.1.4.3 Continuous Spread Footings

Based on our analyses, allowable bearing pressures of 1,000 to 2,500 psf, and continuous footings about 1- to 1.5-foot wide and embedded 2 feet BGS, we estimate total long-term consolidation settlement of **about 2 to 6 inches, with differential settlement equal to half of this amount between adjacent foundation support or across a horizontal distance of approximately 30 feet, whichever is less.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill.

5.3.1.5 Additional Differential Settlement

In addition to the differential settlement discussed above for new foundations, existing linear improvements, such as pavements, concrete flatwork, and underground utilities should not be subject to abrupt differential settlement as a result of placement of the new fill because the settlement should occur uniformly areawide. However, upwards of 3 inches of abrupt differential settlement could occur where these linear improvements are located adjacent to or are connected to existing structures that are supported on deep foundations that extend below the Bay Mud layer. Therefore, site grades may need to be re-adjusted near such structures in the future to eliminate trip hazards that develop as a result of this differential settlement. Also, underground and above ground utilities may eventually be damaged where they connect to such structures. This could be mitigated by installing flexible joints at these connections or by repairing the damage after it occurs.

5.3.2 Mitigation of Long-Term Settlements

Depending on the settlement tolerance of the planned improvements, the long-term settlements discussed above could be mitigated via a combination of measures, including:

1. Placing new structures over a similar footprint and under similar loading conditions as previous structures that are demolished.
2. Waiting up to 2 years after new fill is placed atop the existing levees and to widen the levee embankment for the proposed shelter structure at the WEPS facility. This would allow for a majority of the long-term settlement associated with the new fill to take place before the new improvements are constructed.
3. Using light weight fill, such as geofoam (which can have unit weights as low as approximately 1 pound per cubic foot, pcf) or cellular foam concrete (which can have unit weights as low as 20 pcf)

to offset increased loading. An in-situ soil unit weight of 110 pcf may be assumed to establish the amount of excavation of the existing levee fill and replacement with light weight fill. However, if this measure is used, the geofoam should not extend below the recommended design groundwater depth of 5 feet BGS below the crest of the levees. A petroleum-resistant geomembrane would need to be installed above the geofoam to protect it from future hydrocarbon spills at the project Sites.

4. Founding new structures on a deep foundation system consisting of CIDH piers that extend well below the Bay Mud layer and are designed to carry drag loading associated with consolidation of the Bay Mud layer if fill is used to raise the pond levees.
5. Installing a grid of ground improvement columns underneath shallow foundations.

5.3.3 Construction Considerations

The contractor should exercise extreme care during construction to not disturb the Bay Mud Crust layer underlying the Sites for the CSU project to avoid the potential for causing a bearing capacity failure of the Bay Mud Crust. Otherwise, this could lead to a phenomenon typically referred to as a Bay Mud "wave", where adjacent sections of the Bay Mud layer are pushed up and down, severely impacting existing improvements situated atop the Bay Mud layer. Therefore, **earthwork equipment, soil stockpiles, or construction supplies should not be placed directly over the surface of the Bay Mud Crust layer** either within the oxidation ponds or in sections of the levee that are excavated during construction. **Excavators with long reach arms should be used during excavation, removal of existing piping, placement of new piping, and backfill operations. Such excavators should work from the top of the existing levees only**. If this is not possible, BSK should be consulted to provide additional input/recommendations prior to placing additional loading over the Bay Mud Crust layer.

Any Bay Mud excavated as part of the planned improvements should not be re-used as engineered fill or backfill at the Sites.

5.4 New Foundations

New structures for this project can be supported spread footings, mat foundations, CIDH piers, or a combination of shallow foundations and ground improvement columns depending on their settlement tolerance.

5.5 Anticipated Settlements

The subsections below present our estimated elastic, consolidation, liquefaction-induced, and dynamic compaction/seismic settlements for the planned improvements for this project. For design purposes, these settlements should be assumed to be cumulative.

5.5.1 Elastic Settlement

Total and differential elastic settlements for shallow foundations (i.e., spread footings and mats) are estimated to be less than ½-inch and ¼-inch. Differential settlement is defined herein as the vertical difference in settlement between adjacent fountain supports or across a horizontal distance of approximately 30 feet, whichever is less. Most of the elastic settlement is expected to occur during construction as the loads are applied. These estimates assume the recommendations presented in this report are properly implemented.

5.5.2 Consolidation Settlement

The consolidation settlement for this project is discussed in the preceding "Long-Term Consolidation Settlement" section of this report.

5.5.3 Liquefaction-Induced Settlement

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits and some low plasticity clays. If liquefaction occurs, foundations resting above or within the liquefiable layer may undergo settlements and/or a loss of bearing capacity.

We ran liquefaction analysis for our current CPTs (CPT-3 and CPT-5) using the methods by Boulanger and Idriss (2014)[3] using the program software Cliq. For our analyses, we assumed a design groundwater depth of 5 feet BGS and a peak ground acceleration of 0.68g and earthquake magnitude of M7.22 per the site-specific ground motion hazard analysis presented in Appendix A of this report. The results of our liquefaction hazard analysis are presented in Appendix A and are summarized in the table below. Based on these results, we conclude that the potential for liquefaction analysis to occur at the Sites for the CSU project to be low.

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[3] Boulanger, R. W., and Idriss, I. M. (2014), CPT and SPT Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, California Report No. UCD/CGM-14/01, April 2014.

SUMMARY OF LIQUEFACTION-INDUCED SETTLEMENTS		
CPT	**Estimated Total Liquefaction-Induced Settlement (inches)**	**Estimated Differential[1] Liquefaction-Induced Settlement (inches)**
CPT-3	Less than ¼	Less than ¼
CPT-5	0	0

Note:

1. Differential settlement is defined herein as the vertical difference in settlement between adjacent foundation supports or across a horizontal distance of approximately 30 feet, whichever is less.

Based on Youd and Garris (1995)[4] and the depth and thickness of the potentially liquefiable layers shown in Appendix A, we consider the overall potential for ground surface disruption (such as sand boils, ground fissures, etc.) to occur at the Sites for the CSU project to be low due to relative thickness of the non-liquefiable layers overlying the liquefiable layers.

5.5.4 Dynamic Compaction/Seismic Settlement

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Due to the composition, consistency, and apparent relative density of the soils above the design groundwater level within the current and previous exploration points, we conclude that the potential for dynamic compaction/seismic settlement to affect the Sites for the CSU project during a seismic event is low.

5.6 Geologic and Seismic Hazards

5.6.1 Faulting and Seismic Shaking

The Sites for the CSU project are not located within an Alquist-Priolo Earthquake Fault Zone and no mapped active fault traces are known to transverse the project Sites. Therefore, we conclude that the potential for surface fault rupture to occur across the project Sites is low. Nevertheless, the project Sites are in a seismically active area of California. We expect the project Sites to be subjected to moderate to intense ground shaking due to a significant seismic event on the nearby active faults in the Bay Area and surrounding regions during the design life of the project. The nearby active faults include the Rogers Creek, approximately 3 miles northeast, the West Napa, approximately 13½ miles northeast, and the San Andreas, approximately 17 miles southwest of the project Sites.

[4] Youd, T. L. and Garris, C. T. (1995), Liquefaction-Induced Ground-Surface Disruption, Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 11, November, pp. 805-809.

In 2015, scientists and engineers released a new earthquake forecast for the State of California[5]. It updates the earthquake forecast made for the greater San Francisco Bay Area by the 2007 Working Group for California Earthquake Probabilities. According to this recent study, there is a 72 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area between 2014 to 2044.

As has been demonstrated recently by the 1989 (M6.9) Loma Prieta, the 1994 (M6.7) Northridge, and the 2014 (M6.0) Napa County earthquakes, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban environments. Therefore, the design of new structures should incorporate the seismic design parameters presented in the "Seismic Design Criteria" section of this report.

5.6.2 Slope Stability

Based on our limited slope stability analysis (refer to the "Limited Slope Stability Analysis" section in Appendix A), we expect the existing levees to be globally stable under static and seismic conditions if 2 to 3 feet of additional fill is placed over the levees to increase overall storage capacity for the oxidation ponds. However, it is still possible that some sections of the levees could fail globally during a future significant seismic event at locations where higher concentrations of peat are present or where the Bay Mud Crust layer is thinner (or nonexistent) than assumed in our analysis. Rather than spending significant sums to try and mitigate this potential (which may or may not happen during the design life of the facility), we believe that a more feasible approach would be to repair sections of the levees that fail globally during a significant seismic event.

The above conclusions assume that existing levee slope gradients will be maintained when raising the levees. If steepening of the levee slope gradients is desired, BSK should be consulted to evaluate the potential impact on the global stability of the levees. **For this project, BSK takes no exception to steepening the portion of the levee fill embankment where the proposed shelter structure for the new sodium hypochlorite storage tanks will be located to a 2H:1V slope due to the limited amount of fill height involved (about 3½ feet).**

5.6.3 Expansive Soils

According to the current and previous Atterberg limits testing, the surficial soils at the Sites for the CSU project have a high shrink and swell potential (i.e., high expansive potential) when exposed to moisture fluctuation. Mitigation of expansive soil behavior is recommended by deepening shallow foundations, using continuous perimeter footings, and moisture conditioning the subgrade soils as discussed in the "Spread Footings and Mat Foundations" and "Earthwork" sections, respectively, of this report.

[5] Field, E.H., and 2014 Working Group on California Earthquake Probabilities (2015), UCERF3: A new earthquake forecast for California's complex fault system: U.S. Geological Survey 2015–3009, 6 p., https://dx.doi.org/10.3133/fs20153009.

5.6.4 Liquefaction Potential

The project Sites' liquefaction potential is discussed in the preceding "Liquefaction-Induced Settlement" section of this report.

5.6.5 Dynamic Compaction/Seismic Settlement Potential

The project Sites' dynamic compaction settlement is discussed in the preceding "Dynamic Compaction/ Seismic Settlement" section of this report.

5.6.6 Lateral Spread Potential

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to temporary lateral migration of subsurface liquefied soils during a design seismic event. These phenomena typically occur adjacent to free faces such as slopes and creek channels. Based on our liquefaction analysis results for the current CPTs and the subsurface conditions encountered in the current and previous borings, we conclude that the potential for lateral spreading to occur at the project Sites is low.

5.6.7 Flood Hazard

According to the 2015 Federal Emergency Management Agency (FEMA) flood insurance rate maps[6], the project Sites are located in within Zone AE – Special Flood Hazard Area with a Base Flood Elevation (BFE) determined. The BFE for the area is 10 feet (see Exhibit 1 below). According to the current elevation topographic map of the project Sites, the elevation at the top of the levees for the WEPs facility and the chemical processing area range from about 13 to 15 feet.

[6] Federal Emergency Management Agency (FEMA 2015), FEMA Flood Insurance Rate Map, Sonoma County, California and Incorporated Areas, Map Number 06097C1002G, October 2, 2015.

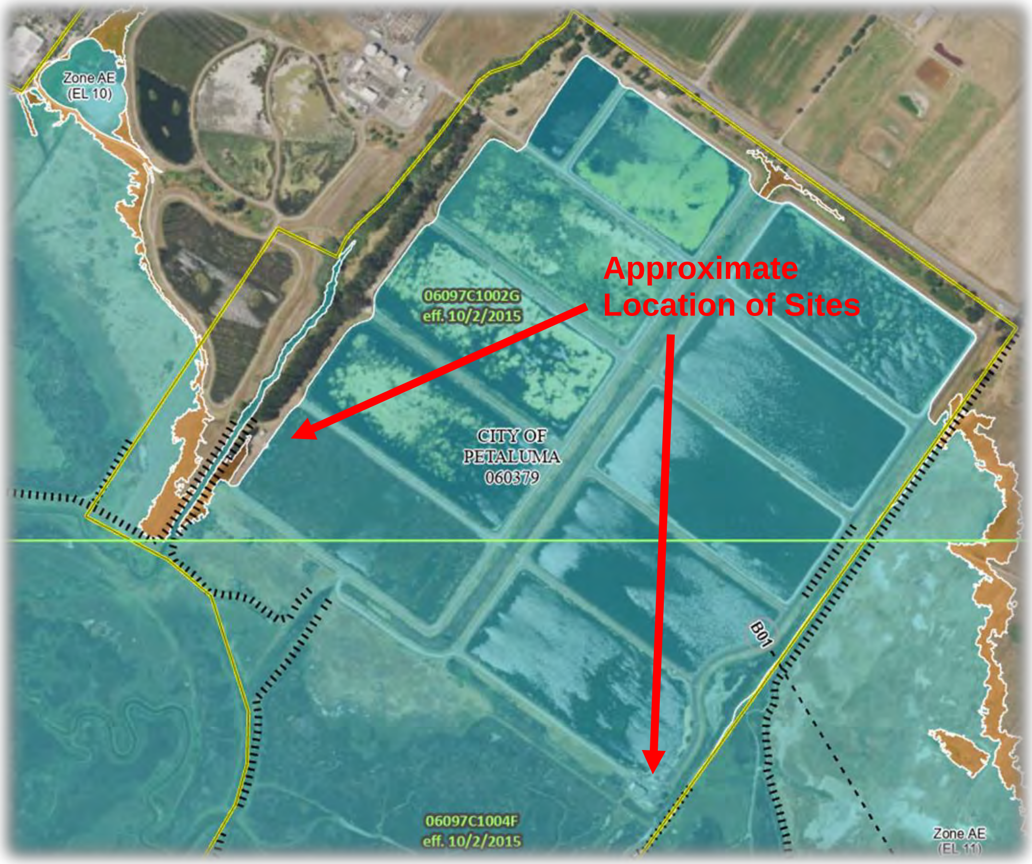

Exhibit 1 – FEMA Flood Map

5.6.8 Tsunami Hazard

According to the CGS (2022[7]) Tsunami hazard area map, the project Sites are just outside the tsunami hazard area (see Exhibit 2 below).

[7] Patton, J.R. and Wilson, R.I. (2022), Tsunami Hazard Area Map, Sonoma County; produced by the California Geological Survey and the California Governor's Office of Emergency Services, dated 2022, displayed at multiple scales.

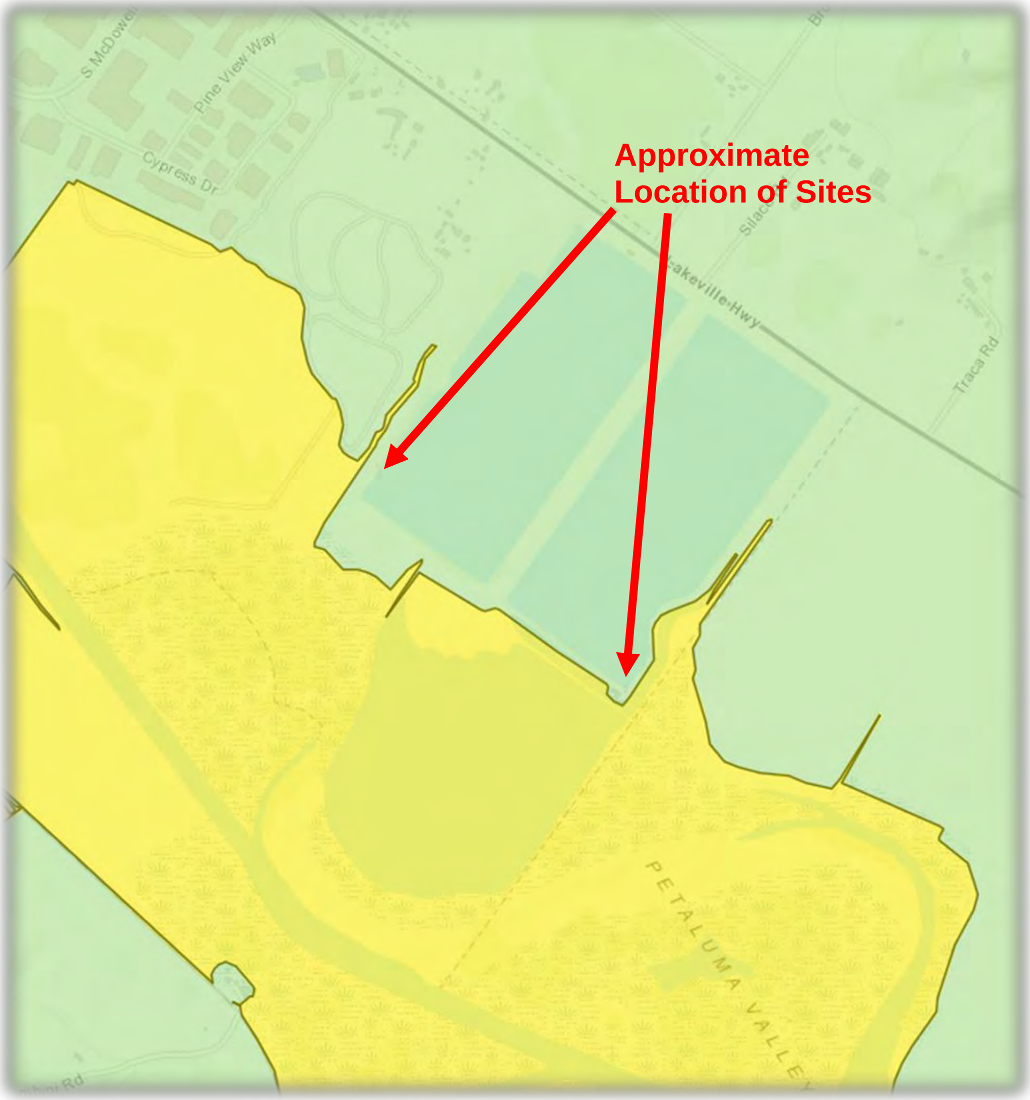

Exhibit 2 – Tsunami inundation map (yellow = tsunami hazard area)

6. RECOMMENDATIONS

Presented below are recommendations for foundations, ground improvement, uplift loading due to buoyancy, retaining walls, seismic considerations, vertical loads on pipes, earthwork, slabs-on-grade, pavements, site drainage, and construction considerations for this project.

6.1 Foundation Recommendations

6.1.1 Spread Footings and Mat Foundations

We recommend the criteria presented in the tables below be incorporated into the design of new structures for this project. The low allowable bearing capacities provided below take into account the presence of Bay Mud underlying the project Sites. Due to the high expansion potential of the surficial soils, **a continuous perimeter footing should be constructed for the new buildings** (unless they are supported on mat foundations) to reduce the potential for moisture fluctuation underneath these structures, which could lead to vertical movement associated with shrinkage/swell cycles.

SPREAD FOOTING DESIGN CRITERIA	
Static Allowable Bearing Capacity[1]	1,000 psf (4,000 psf)
Seismic/Wind Allowable Bearing Capacity[1]	1,500 psf (6,000 psf)
Passive Resistance (Equivalent Fluid Pressure)[2,3]	300 pcf
Allowable Lateral Sliding Resistance Adhesion[3]	600 psf
Minimum Embedment Depth[4]	24 inches
Minimum Width	12 inches (continuous) 18 inches (isolated)

Notes:

1. Includes a factor of safety of at least 3 for static loading and at least 2 for transient loading (i.e., seismic or wind conditions). **Values shown in parenthesis may only be used for footings that are supported on ground improvement columns.**
2. Neglect upper 1 foot if surface is not confined by concrete slab or pavement. **For foundations located on or proximate to sloping ground, such as a levee, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 7 feet between the slope face and the nearest edge of the foundation.**
3. The sliding resistance and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading. Values include a factor of safety of at least 1½. The sliding resistance adhesion should be multiplied by the foundation area to obtain horizontal sliding resistance.
4. Below lowest adjacent grade.

MAT SLAB FOUNDATION CRITERIA[1]	
Static Allowable Bearing Capacity[2]	500 psf (2,500 psf)
Seismic/Wind Allowable Bearing Capacity[2]	750 psf (3,750 psf)
Passive Resistance (Equivalent Fluid Pressure)[3, 4]	300 pcf
Allowable Friction Coefficient[4]	0.30
Modulus of Vertical Subgrade Reaction[5]	30 psi/in
Minimum Slab Thickness[6] at the Edges	12 inches

Notes:

1. Mat slab foundations for below-grade structures should be supported on a minimum of 3 inches of CLSM (refer to the "Site Preparation and Grading" section of this report for CLSM recommendations).

   Mat slab foundations for at-grade structures should be supported on a minimum of 12 inches of compacted Caltrans Class 2 aggregate base to provide enhanced slab support. If moisture vapor through the slab is objectionable (i.e., moisture sensitive flooring or objects will be placed over slabs), a vapor barrier at least 15 mils thick (meeting the requirements of the "Floor Slab Moisture" section of this report) and capillary moisture break consisting of a minimum 6-inch-thick layer of crushed drain rock should be installed underneath mat foundations. If used, the crushed drain rock layer may substitute an equivalent amount of the recommended aggregate base layer. The crushed rock layer should be ¾-inch maximum size with no more than 10 percent by weight passing the No. 4 sieve.

2. Includes a factor of safety of at least 3 for static loading and at least 2 for transient loading (i.e., seismic or wind conditions). **Values shown in parenthesis may only be used for mat slab foundations that are supported on ground improvement columns.**
3. Neglect upper 1 foot if surface is not confined by concrete slab or pavement. **For foundations located on or proximate to sloping ground, such as a levee, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 7 feet between the slope face and the nearest edge of the foundation.**
4. The friction coefficient and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading. Values include a factor of safety of at least 1½. The friction coefficient should be multiplied by the normal force to obtain horizontal sliding resistance.
5. Based on a one square foot bearing plate. This unadjusted value needs to be adjusted for the actual size of the mat as follows:
   a. Multiply by [(m+0.5)/(1.5 x m)] where m is the ratio of the mat length divided by its width (unitless).
   b. If a computer program is used to design the mat for this project and it requires the input of a modulus of subgrade reaction for the Site, the designer should check whether the program requires input of the unadjusted or adjusted modulus of vertical subgrade reaction.
6. Below lowest adjacent finished grade. The thickened edge should be a minimum of 12 inches wide. The slab designer should determine the slab concrete thickness and reinforcing.

6.1.2 Additional Considerations for Shallow Foundations

Where foundations are located adjacent to below-grade structures (including existing footings) or near major underground utilities, the foundation should extend below a 1H:1V (horizontal to vertical) plane projected upward from the structure foundation or bottom of the underground utility to avoid surcharging the below grade structure and underground utility with foundation loads. Where this is not possible or feasible, we recommend that CLSM be used to backfill the portion of the utility trench that extends below the 1H:1V projection. Also, if a utility crosses perpendicular to a footing, if it is located within 2 x W of the bottom of the footing, where W = width of footing, the utility should be encased in CLSM or lean concrete. If a perpendicular utility is located below a depth of 2 x W below the footing, the utility does not need to be encased in CLSM or lean concrete, but the trench should be backfilled with impervious material a distance of 2 feet laterally on each side of the perimeter footing centerline as recommended in the “Excavation and Backfill” section of this report.

Concrete for foundations should be placed neat against firm existing levee fill or engineered fill. **It is critical that foundation excavations not be allowed to dry before placing concrete.** If shrinkage cracks appear in the foundation excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The foundation excavations should be monitored by a representative of BSK for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials.

Where utilities cross under the perimeter footings line and enter “interior” space, the trench backfill should consist of a vertical barrier of impervious type material as explained in the “Excavation and Backfill” section of this report. In addition, where utilities cross through footings, flexible waterproof caulking should be provided between the sleeve and the pipe. Utility plans should be reviewed by BSK prior to trenching for conformance to these requirements.

6.1.3 Short Drilled Piers

We recommend the criteria presented in the table below be incorporated into the design of short drilled pier foundations for non-critical improvements, such as light poles, railings, and fencing for this project. **The criteria presented in the table below should not be used to design the CIDH piers for new structures, particularly for new buildings or the WEPS shelter structure.**

SHORT DRILLED PIER FOUNDATION CRITERIA	
Allowable Downward Skin Friction[1,5]	300 psf
Allowable Passive Resistance (Equivalent Fluid Pressure)[2,5]	300 pcf
Minimum Pier Diameter	12 inches
Minimum Pier Depth Below Ground Surface	3 feet
Maximum Pier Depth Below Ground surface	5 feet
Minimum Pier Center to Center Spacing	3D[3] (axial loading) 6D[3,4] (lateral loading)

Notes:

1. Includes a factor of safety of at least 2. Values may be increased by 1/3 for seismic or wind loads. Uplift resistance may be taken as 2/3 of downward capacity. Weight of piers may be used to resist upward loading.
2. Neglect upper 1 foot if surface is not confined by concrete slab or pavement. **For piers located on or proximate to sloping ground, such as a levee, the passive resistance should be neglected in the upper portion of the piers until there is a horizontal distance of at least 7 feet between the slope face and the nearest edge of the piers.** Passive resistance should be limited to 1,500 psf and may be applied to twice the diameter of the piers. Passive resistance may be increased by 1/3 for seismic or wind loads. Value includes a factor of safety of at least 1½.
3. D = pier diameter. Minimum spacing for lateral loading only applies to piers aligned in the direction of loading (i.e., one or more piers shadow another pier).
4. For piers spaced less than 6D apart and where the loading direction is such that there is one or more trailing pier(s) shadowing the leading pier, reductions to lateral capacity of the trailing pier(s) should be applied as follows:
    a. For trailing[8] piers spaced 3D (D = pier diameter) apart, reduce trailing pier capacity by 50 percent (multiply contribution of trailing piers to group capacity by 0.5),
    b. For trailing piers spaced between 4D and 5D apart, reduce trailing pier capacity by 40 percent (multiply contribution of trailing piers to group capacity by 0.6),
    c. For trailing piers spaced 6D or greater apart, no reduction is needed, and
    d. For trailing piers spaced between 3D and 4D apart and 5D and 6D apart, interpolate the reduction factors provided above.
5. Factor of safety may be used to convert from allowable to ultimate capacity.

6.1.4 Axial Capacity of CIDH Piers

Plots illustrating the ultimate downward (compressive) axial capacity of a unit (1-foot) diameter, straight-sided, cast-in-drilled-hole (CIDH) pier foundation installed from the existing ground surface are shown on Figures 5 and 6. The first plot (Figure 5) is for piers installed within the existing levees without raising the levees. The second plot (Figure 6) is for piers installed after the levees are raised up to 3 feet. Figure 6 should also be used to design the piers for the WEPS storage shelter. The axial capacity for piers with diameters larger than 1-foot may be obtained by multiplying the capacity for the 1-foot diameter pier presented on Figures 5 and 6 by the desired pier diameter (in units of feet). The plots are applicable for piers of up to 36 inches in diameter.

[8] The leading pier is defined as the pier that has no pier in front of it in the direction of lateral loading, while the trailing pier is defined as the pier that is behind (i.e., shadows) the leading pier in the direction of lateral loading.

Due to the presence of Bay Mud underneath the project Sites, the axial capacity plot shown on Figure 6 includes a downdrag zone (associated with consolidation of the Bay Mud layer due to the addition of fill to raise the levees) where axial capacity of CIDH piers should be neglected. It also provides a recommended drag load that should be included in the design of the CIDH piers unfactored. Figure 6 should be used if the CIDH piers are installed less than 2 years after fill is placed to raise the levees. However, **if the CIDH piers are installed 2+ years after the levees are raised, then Figure 5 may be used to design the piers (i.e., no downdrag zone or drag load needs to be applied) because by then we estimate that 90 to 95% of the consolidation settlement will already have occurred**.

The axial capacity was computed based on the Federal Highway Administration (FHWA) procedures for design of drilled piers (Brown et al., 2010)[9] using the computer program SHAFT (version 2012). The ultimate uplift capacity may be obtained by multiplying the ultimate frictional compressive capacity by 2/3 and by adding the weight of the pier foundation. The weight of the foundation is not included in the allowable resistance shown on Figures 5 and 6. Piers should be at least 18 inches in diameter and have a minimum spacing (center to center) of three pier diameters or the vertical capacity provided should be reduced.

The ultimate downward axial capacity does not include end-bearing due to strain incompatibility issues associated with the installation of CIDH piers (i.e., the piers need to settle a certain amount, typically 5 percent of the pier diameter, upon loading before the end bearing capacity can be mobilized).

6.1.5 Lateral Capacity of CIDH Piers

We estimated the displacement, shear, and bending moment for 24-inch diameter CIDH piers under lateral loads using the linear-elastic model in the computer program Lpile (version 2018.10.07). Both fixed-head (no pile top rotation allowed) and free-head (pile top rotation allowed) conditions were analyzed. If a different diameter CIDH pier is proposed, BSK should be consulted to provide an updated Lpile analysis.

Figures 7 through 9 present the Lpile deflection, shear force, and bending moment versus pier length for each pier size analyzed for ¼-, ½-, ¾-, and 1-inch lateral displacements at the pier top. As noted in these figures, the plots are based on unfactored values and the designer should consider applying a factor of safety to the results. **The plots shown on Figures 7 through 9 apply only to piers spaced at least 6 diameters apart (center to center) or where the loading direction is such that there is no trailing pier shadowing the leading pier.** For piers spaced less than 6 diameters apart and where the loading direction is such that there is one or more trailing pier(s) shadowing the leading pier, reductions to lateral capacity of the trailing pier(s) should be applied as follows:

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9 Brown, D.A., Turner, J.P., and Castelli, R.J. (2010), Drilled Shafts: Construction Procedures and LRFD Design Methods, prepared for U.S. Department of Transportation, Federal Highway Administration, Publication No. FHWA-NHI-10-016, 2010.

- For trailing piers spaced 3D (D = pier diameter) apart, reduce trailing pier capacity by 50 percent (multiply contribution of trailing piers to group capacity by 0.5),
- For trailing piers spaced between 4D and 5D apart, reduce trailing pier capacity by 40 percent (multiply contribution of trailing piers to group capacity by 0.6),
- For trailing piers spaced 6D or greater apart, no reduction is needed, and
- For trailing piers spaced between 3D and 4D apart and 5D and 6D apart, interpolate the reduction factors provided above.

6.1.6 Lateral Capacity of Pier Caps

The same lateral capacity parameters recommended in the "Spread Footing and Mat Foundations" section of this report may be used for the design of pier caps. Passive resistance may be used for both static and seismic conditions. Mobilization of passive resistance will require lateral movement of up to 0.004H to 0.04H, where H is the height of the pier cap embedded in the soil. In addition, a side friction based on an allowable friction coefficient of 0.25 and an equivalent fluid pressure of 60 pcf (to be used as the normal force in conjunction with the friction coefficient) may be used for the pier caps. This side friction should be neglected in the upper 1 foot if the ground surface is not confined by a concrete slab or pavement. Also, the side friction should be neglected in the upper portion of the pier caps until there is a horizontal distance of at least 7 feet between the slope face and the nearest edge of the foundation. The applicable pier cap capacities may be use concurrently with the CIDH pier capacities.

6.1.7 Construction Considerations for CIDH Piers and Short Drilled Piers

Due to the presence of the Bay Mud layer, **we recommend that the pier holes deeper than about 7 feet BGS either be temporarily cased during installation or be drilled using the slurry displacement method to reduce the potential for the Bay Mud to cave or squeeze into the pier hole**. We recommend that drilled pier steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each drilled hole. As a minimum, the holes should be poured the same day they are drilled. If the holes cannot be backfilled the same day they are drilled, the holes need to be checked for caving, sloughing or squeezing prior to setting the rebar cage and checked again before pouring concrete. The steel reinforcement should be centered in the drilled hole. Concrete used for pier construction should be discharged vertically into the holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction.

Based on the discussion presented in the "Groundwater" section of this report, groundwater should be anticipated below a depth of 5 feet BGS. However, the actual depth at which groundwater may be encountered in excavations may vary. If water more than 6 inches deep is present during concrete placement, either the water needs to be pumped out or the concrete needs to be placed into the hole using tremie methods. If tremie methods are used, the end of the tremie pipe must remain below the surface of the in-place concrete at all times. Unit prices for dewatering and/or tremie placement methods should be obtained during the bidding process.

Concrete for drilled piers should be designed and placed in general conformance with the recommendations provided in ACI 336.3R-14, Design and Construction of Drilled Piers[10]. The recommendations provided within ACI 336.3R-14 should be followed, in particular when concrete placement is necessary below groundwater level, in caving or sloughing soils, or in sand, which may necessitate casing or the slurry displacement method for concrete placement. These methods require concrete placement at higher slumps than "dry" conditions and concrete mix specifications, including the addition of concrete admixtures and consideration of consolidation methods, should be provided by the design team. If temporary casing is used, it should consist of smooth walled steel. **Corrugated metal pipe (CMP) should <u>not</u> be used as temporary casing because it has a tendency to create voids or disturbed zones during removal and temporary smooth-walled casing should not be left in the hole.**

A BSK representative should be present on a full-time basis during installation of the piers to confirm that subsurface conditions are similar to those encountered in our borings and to check if the contractor is properly casing or using slurry to drill the pier holes that extend deeper than 7 feet BGS.

6.2 Ground Improvement

Based on the subsurface conditions encountered at the project Sites, we believe that drill displacement columns, DDC (or similar methods, such as Geopier® concrete elements) could be used successfully underneath the planned structures for this project to mitigate long-term settlement associated with the presence of Bay Mud. This method also can provide greater bearing capacity for shallow foundations constructed above the ground improvement columns. We do not believe stone columns (or similar methods) would be a feasible ground improvement alternative for this project due to the reduced lateral confinement provided by Bay Mud for such methods.

At this time, we anticipate that the zone requiring ground improvement would need to extend to a depth of about 30 to 40 feet BGS. On a preliminary basis, we anticipate spacings of 8 to 10 feet on centers between ground improvement columns and column diameters of 18 to 24 inches. **The ground improvement columns need to be installed below the footprint of foundations and interior slabs. If ground improvement columns are not installed underneath interior slabs, then such slabs should be designed as structural slabs that span unsupported between adjacent foundations.** The final spacing, diameter, and depth of the ground improvement columns should be designed by a qualified and experienced ground improvement contractor based on magnitude and distribution of the structural loads.

Ground improvement contractors bidding on this project individually or as subcontractors to the general contractors should demonstrate a minimum of 5 years of continuous experience designing and installing ground improvement columns in similar subsurface conditions as that found at the project Sites. They should also provide examples of instances when things went wrong during particular projects and how they were remediated during construction. BSK should be provided an opportunity to review the ground

[10] ACI Committee 336, 2014

improvement plans before they are finalized to confirm they satisfactorily address BSK's findings and recommendations.

During construction, a BSK representative should observe the installation of the ground improvement columns to check that they consistent with the ground improvement plans.

6.3 Uplift Loading Due to Buoyancy

Below-grade structures and new piping for this project should be designed to resist a buoyancy force based on a recommended design groundwater depth of 5 feet BGS (referenced to the crest of the existing oxidation pond levees). The weight of the below-grade structures and piping (assume empty case) may be used to resist this uplift pressure as well as friction between the below-grade structure walls and the surrounding backfill and the backfill above the piping. An allowable friction coefficient of 0.25 between the walls and surrounding backfill may be used. This value includes a factor of safety of about 1½. Normal pressures of 60D psf and 30D psf above and below the design groundwater depth, respectively, where D is the depth in feet of the below-grade structure below the ground surface, may be used to compute the normal force to be used with the allowable friction coefficient.

If the mat foundation for below-grade structures extends beyond the outer reinforced concrete basin wall limits to form a "lip", the weight of the backfill above the lip plus a soil wedge extending upward at a 65-degree angle from the horizontal from the edge of the lip may be used to resist uplift pressure in lieu of the wall friction discussed in the paragraph above. Effective soil unit weights of 120 and 58 pcf may be used above and below the design groundwater depth, respectively.

If additional resistance to buoyancy is required, this could be provided via use of thicker walls and a greater weight for the below-grade structures, deadman anchors, or placing CLSM/lean concrete backfill above the lip of the mat foundation extending beyond the walls. Deadman anchors for new piping could consist of concrete slabs or ballast strapped to the piping.

6.4 Retaining Walls

Above- and below-grade retaining walls up to 10 feet high are anticipated for this project. These walls may be supported on spread footings or mat foundations per the recommendations presented in the "Spread Footings and Mat Foundations" section above. An active earth pressure should be used where the walls are allowed to deflect and an at-rest pressure should be used for restrained walls. The active earth pressure condition will develop only when the wall is allowed to yield sufficiently. The amount of outward displacement at the top of the wall designed for active earth pressures may be up to 0.004H to 0.04H, where H is the height of the wall. Retaining walls may be designed using the lateral earth pressures provided in the table below, which are expressed as equivalent fluid pressures (unit weights) in units of pounds per cubic foot (pcf). If the walls do not include a drainage system, then hydrostatic pressures should be included in the design of the walls regardless of if they are located entirely above the recommended design groundwater depth or not.

LATERAL EARTH PRESSURES FOR WALLS UP TO 10 FEET IN HEIGHT		
Earth Pressures	Equivalent Fluid Pressures (pcf)[A]	
	Above Water[B]	Below Water[B]
Active (Flexible walls)	45	85[C]
At-Rest (Rigid walls)	60	90[C]
Seismic (Flexible walls)	27[D,E]	13[D,E]
Seismic (Rigid walls)	47[D,E]	23[D,E]

Notes:

A. The lateral earth pressures presented herein are applicable for level backfill up to 6H:1V.
B. Design groundwater is at a depth of 5 feet BGS (referenced to the crest of the existing oxidation pond levees).
C. Includes hydrostatic pressure.
D. Only applicable for walls retaining more than 6 feet of soil/backfill.
E. Section 1803.5.12 of the 2022 CBC requires that the design for foundation walls include seismic earth pressures and retaining walls supporting backfill heights greater than 6 feet include seismic earth pressures. These pressures are expressed as equivalent fluid pressures and should be added to the wall design in addition to the static active or at-rest pressures. The seismic earth pressure should be applied as a triangular distribution with the resultant force acting 1/3 times the wall height above the base of the wall. The seismic earth pressures presented herein are based on Agusti and Sitar (2013)[11] and the PGA value of 0.68g per Appendix A of this report.

6.4.1 Wall Drainage

Retaining walls higher than 2 feet should be either designed to resist hydrostatic pressures or be well-drained to reduce the potential for hydrostatic pressures to develop behind the walls. A typical drainage system for a cantilevered wall may consist of a 1- to 2-foot-wide zone of Caltrans Class 2 Permeable material immediately behind the wall with a perforated pipe at the base of the wall discharging to a storm drain or other appropriate discharge facility via gravity flow. As an alternative, a prefabricated drainage board may be used in lieu of the Class 2 Permeable material. Where conditions allow for the use of weep holes, they may be used in lieu of the perforated pipe. The holes should be a minimum of 2 inches in diameter and spaced at 4 feet or less on-center. Filter fabric or wire mesh should be placed over the holes at the backside of the wall to inhibit the permeable material, if used in lieu of a drainage board, from washing through the holes. **Unless the drainage zone behind retaining walls is protected by concrete flatwork or pavement, it should be capped with a minimum 12-inch-thick layer of properly compacted on-site clayey soil to reduce the risk of surface runoff discharging into the wall drain.**

6.4.2 Surcharge Loads

Surcharge loads caused by vehicular and/or construction traffic adjacent to the walls, such as HS-20 live load, may be assumed to consist of a rectangularly distributed uniform pressure of 100 psf acting over a depth of 10 feet below the ground surface of the retained soil. For other surcharge loads, a rectangular

[11] Agusti, G.C. and Sitar, N. (2013), Seismic Earth Pressures on Retaining Structures in Cohesive Soils, report submitted to the California Department of Transportation (Caltrans) under Contract No. 65A0367 and NSF-NEES-CR Grant No. CMI-0936376: Seismic Earth Pressures on Retaining Structures, Report No. UCB GT 13-02, August 2013.

distribution with a uniform pressure equal to one-third and one-half of the surcharge pressure should be used for an unrestrained wall (active earth pressure condition) and for a restrained wall (at-rest earth pressure conditions), respectively. The additional surcharge pressure should be applied over the entire height of the wall. Additional analyses during design may be needed to evaluate the effects of non-uniform surcharge loads such as point loads, line loads, or other such presently undefined surcharge loads. In that case, we should be consulted for supplemental geotechnical recommendations.

6.5 Seismic Design Criteria

The project Sites are in located in a region of high seismic activity and will likely be subjected to moderate to intense ground shaking during the life of the project. As a result, structures to be constructed for the project should be designed in accordance with applicable seismic provisions of the 2022 California Building Code (CBC).

6.5.1 Mapped 2022 CBC Seismic Design Parameters

Based on Section 1613.2.2 of the 2022 CBC, the project Sites shall be classified as Site Class A, B, C, D, E or F based on the Sites' soil properties and in accordance with Chapter 20 of ASCE 7-16. Based on the current and previous subsurface data for the project Sites, we recommend the Sites be classified as a Site Class D. **A site-specific ground motion hazards analysis for this project is presented in Appendix A of this report and is discussed in the next section of this report.** However, as an option (if desired by the structural engineer), we have provided mapped 2022 CBC seismic design parameters in the table below, including increased values for S_{M1} and S_{D1} per the exception for Site Class D sites provided in ASCE 7-16, Supplement 3, Section 11.4.8, Item 1.

2022 CBC SEISMIC DESIGN PARAMETERS (Lat: 38.222148, Lon: -121.568094)			
Seismic Design Parameter	**Value**		**Reference**
Site Class	D		ASCE 7-16, Table 20.3-1
MCE_R Mapped Spectral Acceleration (g)	S_S = 1.847	S_1 = 0.704	USGS Mapped Values
Site Coefficients (Site Class D)	F_a = 1.0	$F_v = 1.7^A$	ASCE 7-16, Table 11.4-1 & -2 **(Supplement 3)**
MCE_R Mapped Spectral Acceleration Adjusted for Site Class Effects (g)	S_{MS} = 1.847	S_{M1} = 1.795 **(See Note B below)**	ASCE 7-16, Eq. 11.4-1 & -2 **(Supplement 3)**
Design Spectral Acceleration (g)	S_{DS} = 1.231	S_{D1} = 1.197 **(See Note B below)**	ASCE 7-16, Eq. 11.4-3 & -4 **(Supplement 3)**
Site Short Period – T_S (Seconds)	T_S = 0.972		$T_S = S_{D1}/S_{DS}$
Site Long Period T_L (Seconds)	8		USGS Mapped Value
Seismic Design Category (SDC)	D		ASCE 7-16, Section 11.6
MCE_G peak ground acceleration adjusted for Site Class effects (g)	PGA_M = 0.854		ASCE 7-16, Section 11.8.3

Definitions:
MCE_R = Risk-Targeted Maximum Considered Earthquake
MCE_G = Maximum Considered Earthquake Geometric Mean
Notes:
A. See requirements for site-specific ground motions in ASCE 7-16, Section 11.4.8. This value of F_v shall be used only for calculation of T_S, determination of Seismic Design Category, linear interpolation for intermediate values of S_1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of S_{D1}.
B. **S_{M1} and S_{D1} values with a 50% increase assuming the exception for Site Class D described in ASCE 7-16 Supplement 3, Section 11.4.8, Item 1 is taken. Otherwise, a site-specific ground motion analysis per ASCE 7-16 Section 21.2 is required.**

6.5.2 Site-Specific Ground Hazard Analysis and 2022 CBC Seismic Design Parameters

A site-specific ground motion hazard analysis based on Section 21.2 of ASCE 7-16 for the project area is presented in Appendix A of this report. 2022 CBC seismic design parameters based on the site-specific ground motion hazard analysis are also presented in Appendix A.

6.6 Vertical Loads on Pipe

The piping for the project should be capable of supporting vertical loads due to the soil overburden (trench backfill) and surcharge, including traffic loads. An in-place density of 130 pounds per cubic foot may be assumed for the trench backfill, and Marston's Formula[12] may be used. The table below presents the

[12] Marston, A, and Anderson, A.P., "The Theory of Loads on Pipes in Ditches and Tests of Cement and Clay Drain Tile and Sewer Pipe." Iowa Eng. Sta., Bull. No. 31 (1913).

vertical pressure on the pipe due to an HS-20 live load as defined in the "American Iron and Steel Institute, Handbook of Steel Drainage and Highway Construction Products".

VERTICAL LOADS ON PIPE	
Height of Cover Over Pipe (Feet)	**Vertical Pressure on Pipe (psf)**
1	1,800
2	800
4	400
6	200
8	100
>8	Neglect live load

Additional surcharge loads on the pipe should be considered in the design if the loads are located above the pipe or within a 1H:1V plane projected upwards from the spring line of the pipe.

6.7 Foundation Support and Backfill for Below-Grade Structures

Removal of existing pipes, installation of new pipes, and removal of existing and construction of new below-grade structures (if applicable) for the project will occur within existing levees. Therefore, **typical pipe bedding and shading material consisting of granular soils should <u>not</u> be used for new pipes or below-grade structures that protrude through the levee embankments**. Otherwise, adverse seepage conditions could lead to failure of the levees via internal erosion of the levee embankments, which is commonly referred to as "piping”[13]. Concrete ballast a minimum of 6 inches thick should be installed immediately below the new pipes that protrude through the levee embankments. The purpose of the ballast is to provide pipe support and a gap below the new pipes to allow proper backfill under the new pipes. Backfill under and around the new pipes and extending at least 6 inches above the crown of the new pipes should consist of CLSM. The ballast should be installed in a manner that allows the CLSM to flow freely to fill all voids under and around the new pipes. The new pipes should be secured to the ballast using straps or other means to avoid having the pipes float when they are being backfilled with CLSM.

Once the CLSM has sufficiently cured to allow soil backfill to be placed above it and mechanically compacted, the soil excavated from the levee fill may be used to backfill the remainder of the pipe excavation provided it is free of deleterious matter, organics, and Bay Mud. If imported fill is needed to backfill the zone above the pipe, it should meet the levee fill criteria provided in the “Site Preparation and Grading” section of this report.

Also, **the excavation bottom for new below-grade structures (if applicable) should <u>not</u> be covered by crushed drain rock or similar material to create a stable base on which to construction the new foundation for such structures**. If the exposed surface at the bottom of the excavation is unstable, a layer

[13] A condition where flowing water transports soil particles out of the inner core of an earthen dam/levee creating a hole within the dam/levee embankment.

of CLSM a minimum of 6 inches thick should be placed over the bottom of the excavation. Backfill around new below-grade structures should consist of the soil excavated from the levee fill provided it is free of deleterious matter, organics, and Bay Mud. If imported fill is needed for backfill, it should meet the levee fill criteria provided in the "Site Preparation and Grading" section of this report.

6.8 Demolition

6.8.1 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines that are 2 inches in diameter or less may be left in place beneath the planned structures provided they are cut off and capped at the structure perimeters. Pipelines larger than 2 inches in diameter within the planned structure footprint should be removed or filled with CLSM meeting the project specifications. Active utilities to be reused should be carefully located and protected during demolition and during construction.

6.8.2 Excavation and Backfill of Existing Foundations and Below-Grade Structures

If applicable, all existing foundations and below-grade structures to be abandoned should be demolished and removed. The resulting excavations should then be properly backfilled with compacted engineered fill per the requirements of the "Earthwork" section of this report. A BSK representative should observe and test the compaction for earthwork activities during construction.

6.8.3 Reuse of On-site Concrete, Asphalt Concrete, and Aggregate Base

Where applicable, existing asphalt concrete (AC) may be pulverized and mixed with the underlying gravel layer (i.e., aggregate base) for re-use in the lower 6 inches of the aggregate base layer for new gravel roadways and paved areas after the levees are raised 2 to 3 feet. The processing should be performed in such a manner that the pulverized AC meets the gradation, R-Value, durability index, and sand equivalent requirements of Section 26 of the 2018 Caltrans Standard Specifications, unless otherwise indicated by BSK during construction. Also, **the contractor should exercise extreme care not to contaminate the pulverized AC and existing AB with the underlying clayey subgrade soils during removal or this could result in rejection of a portion or all the removed materials for use as aggregate base for new gravel roadways and paved areas.**

6.9 Earthwork

6.9.1 Site Preparation and Grading

Our general site preparation and grading recommendations are as follows:

1. The areas to be graded should be cleared of debris, significant surface vegetation and obstructions including abandoned underground pipes, foundations, and concrete slabs. Stripped surface

organics should be disposed off-site.

2. **From a geotechnical standpoint only, the levee fill is generally suitable for re-use as general engineered fill[14] provided it is free of deleterious matter, organics, and Bay Mud and properly processed so that particle sizes are not greater than 3 inches in largest dimension.** At least 90 percent by weight of the general engineered fill/backfill materials should be passing the 1-inch sieve. Nesting (i.e., concentration) of larger particles should be avoided to reduce the potential that this could create voids and allow future settlement in the overlying fill/backfill or "piping" failure of the levee. All fill materials should be subject to evaluation and approval by a BSK representative prior to their use.

   If zones of loose/soft or saturated soils, including in existing fill areas, are encountered during excavation and compaction, deeper excavations may be required to expose firm soils. This should be evaluated in the field by a BSK representative. Where deleterious matter is encountered in excavations, this material should be overexcavated and disposed off-site.

3. Controlled Low Strength Material (CLSM) typically consists of a mixture of cement, fly ash, coarse and fine aggregate, an air entrainment admixture, and water. Where foundations will bear on CLSM, the CLSM should have a 28-day compressive strength of at least 50 pounds per square inch (psi) tested in conformance with ASTM D4832 and sampled in accordance with ASTM D5971. For future excavatability of the CLSM, its 28-day compressive strength should not exceed 1,000 psi. A minimum of one set of cylinders should be cast each day CLSM is placed. One flowability test should be conducted per ASTM D6103 each day CLSM is placed and should be at least 8 inches diameter prior to placement.

   The CLSM mix design should be reviewed by the design team and BSK for approval at least 10 business days prior to its use. CLSM placement should be observed and tested by a qualified representative of BSK.

4. **Imported levee fill** material should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." The levee fill should meet the criteria presented in the California Code of Regulation, Title 23, Section 120, which is summarized in the table below (unless otherwise permitted by BSK). **Highly pervious materials such as pea gravel or clean sands should not be used.**

IMPORT LEVEE FILL CRITERIA	
Plasticity Index	8 or greater
Liquid Limit	Less than 50%
% Passing the 3-inch Sieve	100%
% Passing No. 200 Sieve	20% or greater

[14] "General engineered fill" is defined in this report as suitable **on-site soil** that is used to backfill excavations or raise site grade and is properly moisture conditioned and compacted per the requirements of this report. The requirements for the suitability of on-site soils are provided in the "Site Preparation and Grading" section of this report.

5. Following stripping and removal of deleterious materials in areas of the project Sites to receive fill, the Site should be scarified to a minimum depth of 12 inches, moisture conditioned to at least 2 percent above optimum moisture content, and re-compacted to a minimum of 90 percent relative compaction. **It is important to meet this minimum moisture conditioning due to the expansion potential of the near-surface soils.** Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density determined by ASTM D1557 compaction test procedures. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. Scarification and recompaction should extend laterally a minimum of 5 feet beyond the limits of the planned improvements, where achievable. **Per the "Grading of Levee Slopes" recommendations below, the bottom of keyways for levee slopes should not be scarified.**

6. We expect new fill to settle an amount equivalent to 1 percent of the fill thickness even if it is compacted to a minimum of 90 percent compaction. For instance, if the fill thickness is 8 feet, that would be equivalent to about 1 inch of settlement. Although most of this settlement is expected to occur during construction, a portion of this settlement could occur several months to 1+ year after grading for the project is completed. To address this potential settlement, the required compaction for deeper fills should be increased. Therefore, **where fills/backfills are greater than 7 feet in depth below finish grade, the zone below a depth of 7 feet should be compacted to a minimum of 95 percent relative compaction**. Note that increasing the compaction effort should reduce the amount of fill settlement, but it will not eliminate it.

7. **In areas to be exposed to vehicular traffic**, the upper 12 inches of the soil subgrade immediately below the aggregate base layer should be compacted to a minimum of 92 percent relative compaction at least 2 percent above optimum moisture content. Subgrade preparation should extend a minimum of 5 feet laterally beyond the edge of flatwork, pavers, and pavements, where achievable. The aggregate base layer underneath such flatwork, pavers, and pavement should be compacted to a minimum of 95 percent relative compaction at near optimum moisture content. In addition to these compaction requirements, areas to be exposed to vehicular traffic should be firm and stable and should be proof rolled with a heavy piece of construction equipment, such as a loaded dump truck or water truck, to check for signs of subgrade instability.

8. Unless otherwise indicated above, all fill and backfill should be placed in thin lifts up to 8-inch maximum uncompacted thickness, properly moisture conditioned to at least 2 percent above optimum moisture content for clayey soils and to near optimum moisture content for granular soils, and compacted to at least 90 percent compaction per ASTM D1557. Aggregate base should be moisture conditioned to near-optimum moisture content.

9. **Grading of Levee Slopes**: Current levee slope gradients should be maintained when raising the levees unless BSK is consulted to evaluate the feasibility of steepening slope gradients. **As noted in the "Slope Stability" section of this report, BSK takes no exception to steeping the portion of the levee fill embankment, where the proposed shelter structure for the new sodium hypochlorite storage tanks will be located, to a 2H:1V slope due to the limited amount of fill height involved (about 3½ feet)**. As previously discussed, existing levee slopes have gradients of

about 3H:1V or flatter. If existing levee slope gradients are maintained while raising the levees by 2 to 3 feet vertically, this would require widening one or both sides of the levees a total width of at least 12 to 18 feet at the base depending on the thickness of new fill placed and the existing slope gradient. Our recommendations for widening the levees are discussed below.

a. During widening of the levees, the new levee fill should be overbuilt a minimum of 2 feet laterally and then cut back to finished grade to allow proper compaction of the finished slope face. The widened portion of the levees should be supported on 18-inch-deep keyways that are a minimum of 3 to 5 feet wide or as indicated by a BSK representative during construction. A layer of Mirafi RS280i geotextile fabric or equivalent should be placed over the bottom of the keyways unless indicated otherwise by BSK during construction. The geotextile fabric should be overlapped a minimum of 2 feet at the seams. The contractor should exercise extreme care not to excavate the keyways any deeper than recommended herein. Otherwise, the integrity of the Bay Mud Crust layer could be compromised. For this reason, the bottom of the keyways should <u>not</u> be scarified. The backside (back cut) of the new levee fill should be benched into the existing levee fill at regular vertical intervals of about 2 to 3 feet as the new levee fill placement proceeds upslope of the keyway base. The bench width should be a minimum of 2 feet wide.

b. Consideration should be given to installing rock slope protection (RSP) as part of the outer surface of the new levee fill slope to provide long-term protection against future surface erosion. The RSP layer should be a minimum of 1-foot thick and should consist of Class II rock gradation per Section 72-2.02B of the 2018 Caltrans Standard Specifications. The RSP layer should be underlain by Class 10 RSP fabric meeting the requirements of Section 96-1.02I of the 2018 Caltrans Standard Specifications, such as Mirafi® 1100NC or equivalent overlapped at minimum of 1 foot at the seams and fixed to the surface of the slope using staples per the manufacturer's requirements.

c. At the conclusion of construction operations, portions of the levee slopes that are not protected by RSP should be hydroseeded to help encourage growth of vegetation on the surface to serve as an additional long-term erosion control measure. Consideration should be given to covering these areas with a biodegradable woven coir erosion control blanket to help provide temporary erosion protection until vegetation is re-established over the area. If used, the woven coir erosion control blanket should meet the requirements of Section 21-2.02O(4), Type B of the 2018 Caltrans Standard specifications, such as North American Green BioNet® 125 (C125BN) or equivalent. The woven coir erosion control blanket should be overlapped a minimum of 1 foot at the seams and fixed to the surface of the slope using wooden stakes or staples per the manufacturer's requirements.

10. Observations and compaction testing should be carried out by a BSK representative during grading and backfill operations, especially during widening of the levees, to assist the contractor in obtaining the required degree of compaction and proper moisture content. Where the moisture

content or compaction is outside the range required, additional compactive effort and adjustment of moisture content should be made until the specified compaction and moisture conditioning is achieved.

11. BSK should be notified at least 48 hours prior to any grading and backfill operations. The procedure and methods of grading may then be discussed between the contractor and BSK.

6.9.2 Excavation and Backfill

All excavations should conform to current OSHA requirements for work safety. Where trenches or other excavations extend deeper than 5 feet, the excavations may become unstable and should be evaluated by the contractor to monitor stability prior to personnel entering the trenches. Shoring or sloping of any trench wall may be necessary to protect personnel and to provide stability. It is the contractor's responsibility to follow OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be adjusted as necessary in the field to suit the actual conditions encountered, in order to protect personnel and equipment within excavations. Based on the subsurface conditions encountered in the current and previous exploration points, we expect the sidewalls of trenches that extend to depths of up to about 5 feet to remain relatively vertical for a period of several days. Nevertheless, the longer the trenches remain open the higher the potential for the sidewalls to start to slough off or cave.

As discussed in the "Subsurface Conditions" of this report, free groundwater was observed at depths ranging from about 10 to 25 feet BGS within the current and previous exploration points performed at the project Sites. However, the actual depth at which groundwater may be encountered in trenches and excavations may vary. We assume pertinent oxidation ponds will be drained where the planned improvements extend down into the ponds. Due to the predominantly clayey nature of the levee fill and the underlying Bay Mud layer, we expect that dewatering can be conducted primarily via use of sumps and open pumping. However, the contractor should be responsible for the means and methods for dewatering the project Sites provided the design/construction management team is afforded an opportunity to review and comment on the proposed dewatering method(s) to be used. **Groundwater should be lowered and maintained at least 2 feet below the bottom of the planned excavations in order to maintain the undisturbed state of the supporting soils and to allow proper compaction of backfill after below-grade structures and utility lines are installed.**

Where new utility trenches extend from the exterior into the interior limits of a building or pavement, CSLM or lean concrete should be used as backfill material for a distance of 2 feet laterally on each side of the perimeter footing centerline or the pavement edge to reduce the potential for the trench to act as a conduit for exterior surface water. Utility trenches located in landscaped or unimproved areas of the project Sites should also be capped with a minimum of 12 inches of compacted on-site clayey soils.

6.9.3 Weather/Moisture Considerations

If earthwork operations and construction for this project are scheduled to be performed during the rainy season (usually November to May) or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures, such as quicklime treatment, to the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations.

6.10 Slabs-on-Grade

Slabs-on-grade for this project may consist of concrete floor slabs and exterior flatwork/pavers. The near-surface soils have a high expansion potential and will be subject to shrink/swell cycles with fluctuations in moisture content. To reduce these potentially adverse effects, we recommend that interior concrete slabs and exterior flatwork/pavers be underlain by 24 inches and 12 inches of "non-expansive" engineered fill, respectively, placed on subgrade prepared as described in the "Site preparation and Grading" section of this report. The properties of this "non-expansive" fill should meet the criteria presented in the table below. **As discussed in the "Interior Floor Slabs" section below, the upper 6 inches of the 24-inch "non-expansive" fill below interior slabs should consist of crushed drain rock.**

"NON-EXPANSIVE" FILL CRITERIA	
Plasticity Index	12 or less
Liquid Limit	Less than 30%
% Passing the 3-inch Sieve	100%
% Passing the 1-inch Sieve	90%
% Passing #200 Sieve	8% – 40%
Corrosivity	Not be more corrosive than the on-site soils and not be classified as being more corrosive then "moderately corrosive".

The "non-expansive" fill should extend laterally a minimum horizontal distance of 5 feet beyond the limits of structures (defined as the outside perimeter of building walls or foundation outer limits, whichever results in the greatest building envelope) and 3 feet beyond the edge of flatwork/pavers where achievable. Where "non-expansive" fill is used, it is important that placement of this material be done as soon as possible after compaction of the subgrade to prevent drying of the native subgrade soils and that slabs be constructed as soon as possible after "non-expansive" material is placed, as subgrades will dry out even through "non-expansive" fills. A representative of BSK should be present to observe the condition of the subgrade, and observe and test the installation of the "non-expansive" engineered fill prior to slab construction.

Where "non-expansive" fill is removed to install utilities within the limits of buildings, exterior flatwork, and pavers, this layer should be backfilled with new imported "non-expansive" fill and not the "non-

expansive" fill that was removed from the trench. This is because it is difficult to keep "non-expansive" fill separated from other soil excavated from the trench.

6.10.1 Interior Floor Slabs

Concrete floor slabs should be supported on at least 6 inches of crushed drain rock to enhance subgrade support for the slab and provide a capillary moisture break. This material may be considered part of the required minimum of 24 inches of "non-expansive" fill. If moisture vapor through interior slabs is objectionable (i.e., moisture sensitive flooring or objects will be placed over slabs), a vapor barrier at least 15 mils thick (meeting the requirements of the "Floor Slab Moisture" section of this report) should be placed above the crushed drain rock layer and the crushed drain rock material should be ¾ inch maximum size with no more than 10 percent by weight passing the No. 4 sieve. It is important that placement of this material and concrete be done as soon as possible after compaction of the "non-expansive" fill to reduce drying of the subgrade below.

A Structural Engineer should design reinforcing and slab thickness. The floor slab should be separated from footings, structural walls, and utilities and provisions made to allow for settlement or swelling movements at these interfaces. If this is not possible from a structural or architectural design standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

6.10.2 Floor Slab Moisture

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation), **a vapor barrier should be incorporated into the floor slab design in all areas where moisture sensitive floor coverings, coating, underlayments, adhesives, moisture sensitive goods, humidity-controlled environments, or climate-cooled environments are anticipated initially or in the future**. The vapor barrier should consist of a minimum 15 mil extruded polyolefin plastic, such as 15 mil Stego® Wrap vapor barrier or equivalent. The vapor barrier material should not include any recycled or woven materials and should have a permeance (as tested before and after mandatory conditioning per ASTM E1745 Section 7.1, latest edition) of less than 0.01 perms and should comply with ASTM E1745 Class A requirements. The vapor barrier should also meet Sections 8.1 and 9.3 of ASTM E1745 and subsequent documentation should be provided by the vapor barrier manufacturer. The vapor barrier should be installed in accordance with ASTM E1643, latest edition, including proper perimeter seal, such as Stego® Crete Claw® tape.

The vapor barrier should be placed directly over the crushed rock layer recommended in the "Interior Floor Slabs" section of this report. **A sand layer should not be placed between the vapor barrier and the concrete slab or it could serve as a reservoir for trapped moisture that could lead to long-term vapor transmission through the slab.**

It should be noted that although vapor barrier systems are currently the industry standard, these systems may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the interior slab-on-grade floor design. Building design and construction have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture and can control future performance. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual.

It is emphasized that we are not floor moisture vapor proofing experts. We make no guarantee nor provide any assurance that use of capillary break/vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Exterior grading will have an impact on potential moisture beneath the floor slab. Recommendations for exterior drainage are provided in the “Site Drainage” section of this report.

It should be noted that the purpose of vapor barrier systems is to mitigate floor moisture vapor. These systems should <u>not</u> be used for waterproofing against shallow groundwater or surface water.

6.10.3 Exterior Concrete Flatwork and Pavers

New exterior concrete flatwork and pavers will be constructed on soils subject to swell/shrink cycles. Some of the adverse effects of swelling and shrinking can be reduced with proper moisture treatment. The intent is to reduce the fluctuations in moisture content by moisture conditioning the soils, sealing the moisture in, and controlling it. Near-surface soils to receive exterior concrete flatwork and pavers should be moisture conditioned according to the recommendations in the “Site Preparation and Grading” section of this report.

In addition, all exterior flatwork and pavers should be supported on a minimum of 12 inches of "non-expansive" fill. Where concrete flatwork and pavers are to be exposed to vehicle traffic, the upper 6 inches of the "non-expansive" fill should consist of Caltrans Class 2 aggregate base.

Practices recommended by the Portland Cement Association (PCA) and the American Concrete Institute (ACI) for proper placement and curing of concrete, as well as for joint spacing and construction, should be followed during exterior concrete flatwork slab construction. Due to the presence of highly expansive soils near the site surface, flatwork should have control joints (i.e., weakened plane joints) spaced no more than 8 feet on centers. New pedestrian concrete flatwork should have a minimum thickness of 4 inches and minimum reinforcing of #4 bars at 18 inches on center (both ways). The rebar should be discontinued at expansion joints. Slip Dowels should be used at expansion joints. Vehicular concrete should be designed as discussed in the "Portland Cement Concrete Pavements" section of this report. Final design of exterior concrete flatwork is the responsibility of the civil or structural engineer for the project.

Exterior flatwork and pavers will be subjected to edge effects due to the drying out of subgrade soils. To protect against edge effects adjacent to unprotected areas, such as vacant or landscaped areas, lateral cutoffs, such as inverted curbs (i.e., turndown edges) that extend at least 2 inches below the aggregate base or "non-expansive" fill layer into the subgrade soils, are recommended. Alternatively, a moisture barrier at least 80 mils thick extending at least 6 inches below the aggregate base or "non-expansive" fill layer into the subgrade soils could be installed at the edge of the flatwork and pavers. If quicklime treatment is used in lieu of "non-expansive" fill, the cutoff can be eliminated where no aggregate base is used.

Prior to construction of the flatwork and pavers, the aggregate base should be moisture conditioned to near optimum moisture content. If the aggregate base is not covered within about 30 days after placement, the soils below this material will need to be checked to confirm that their moisture content is at least 2 percent over optimum. If the moisture is found to be below this level, the aggregate base layer over flatwork and paver areas will need to be soaked until the proper moisture content is reached. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork/pavers and buildings, including concrete driveways.

6.11 Pavements

6.11.1 Asphalt Concrete Pavements

The near surface soils at the project Sites have a high expansion potential and are therefore expected to have a low Resistance Value (R-Value). Based on our experience, we used an R-Value of 5 to develop the asphalt pavement sections provided in the table below, which may be used at this project. Based on the anticipated HS-20 live load and assuming one traffic delivery a week, we believe a Traffic Index of 6.0 would be suitable for the design of the asphalt concrete pavement section for this project. Using the Caltrans Flexible Pavement design method, a Traffic Index of 6.0, and an R-Value of 5, **we recommend that the pavement section consist of a minimum of 3.0 inches of asphalt concrete over a minimum of 13.0 inches of Caltrans Class 2 aggregate base**.

6.11.2 Portland Cement Concrete Pavements

Portland Cement Concrete (PCC) pavement should have a minimum thickness of 6 inches supported over 6 inches of Caltrans Class 2 aggregate base. This section is equivalent to a Traffic Index of at least 6.0 based on our experience and is expected to support an HS-20 live load. The aggregate base and subgrade for PCC pavements should be properly moisture conditioned and compacted. Construction joints should be located no more than 12 feet apart in both directions. Concrete compressive strength should be tested in lieu of third point loading for rupture strength. A minimum 28-day compressive strength of 3,000 pounds per cubic foot (psi) should be specified for the concrete mix design. The PCC pavement should be continuously reinforced using No. 4 bars (or larger) spaced no more than 18 inches on center in both directions. Final design of the PCC pavement is the responsibility of the civil or structural engineer for the project.

6.11.3 Gravel Roadways

We recommend that a minimum of 12 inches of Caltrans Class 2 aggregate base be used for new gravel-covered roadways. For enhanced performance, considerations should be given to underlaying the aggregate base section with Mirafi® RS280i geotextile fabric or equivalent. The subgrade and aggregate base layer should be compacted per the requirements of areas to be exposed to vehicular traffic as discussed in the “Site Preparation and Grading” section of this report.

6.11.4 Additional Pavement Recommendations

Paved areas should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water ponding should not be allowed anywhere on the project Sites during or after construction. We recommend that the pavement section be isolated from non-developed areas and areas of intrusion of irrigation water from landscaped areas. **Concrete curbs should extend a minimum of 2 inches below the aggregate base and into the subgrade to provide a barrier against drying of the subgrade soils or reduction of migration of landscape water into the pavement section.** Weep holes spaced at 4 feet on centers should also be provided. In lieu of the weep holes, a more effective system is to install a subdrain behind the curbs.

6.12 Site Drainage

Proper site drainage is important for the long-term performance of the planned improvements. The Sites for the CSU project should be graded to provide positive drainage towards ditches, drain inlets, catch basins, bioretention areas, and similar drainage collection facilities, and away from levee slope faces where possible. The Sites should be graded so as to carry surface water away from the buildings and other structures at a minimum of 2 percent in flatwork areas and 5 percent in landscaped areas to a minimum of 10 feet laterally from a structure’s perimeter foundations as required by the 2022 CBC. If used, roof gutters should be connected directly into the storm drainage system or drain onto impervious surfaces provided that a safety hazard is not created. Water should not be allowed to pond anywhere on-site.

6.13 Corrosion Potential

Soil samples were collected during our current subsurface investigation from boring B-3 from depths of about 0 to 5 and 15½ feet BGS and from boring B-5 from depths of about 0 to 5 feet BGS. These samples were submitted for corrosion testing. The samples were tested by CERCO Analytical, a State-certified laboratory in Concord, California, for redox potential, pH, resistivity, chloride content, and sulfate content in accordance with ASTM test methods. The test results are presented in Appendix A. Also included is the evaluation by CERCO Analytical of the corrosion test results.

Based upon the resistivity measurements, the samples tested were classified as "corrosive” to “severely corrosive” by CERCO Analytical. The sulfate ion concentrations ranged from 27 to 390 mg/kg (ppm). These results are indicative of an exposure category S1 per Table 19.3.1.1 of ACI 318-19. For an S1 exposure class, Table 19.3.2.1 indicates that the minimum f’c of the concrete is 4,000 psi. CERCO Analytical concludes that the sulfate ion concentrations are sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel. They recommend that concrete that comes into contact with the soil should use sulfate resistant cement such as Type II with a maximum water-to-cement ratio of 0.55. CERCO Analytical also recommends that all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron be properly protected against corrosion depending upon the critical nature of the structure. They also recommend that all buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details.

The above are general discussions. A more detailed investigation may include more or fewer concerns and should be directed by a corrosion expert. BSK does not practice corrosion engineering. Consideration should also be given to soils in contact with concrete that will be imported to the project Sites during construction, such as topsoil and landscaping materials, which typically contain fertilizers and other chemicals that can be highly corrosive to metals and concrete. Any imported soil or landscaping materials should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

6.14 Plan Review and Construction Observation

We recommend that BSK will be retained by the Client to review the geotechnical aspects of the plans and specifications before they go out to bid. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings of our recommendations prior to the start of construction.

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained and/or reviewed during this investigation and the actual soil conditions encountered during construction, we recommend that BSK be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in the current and previous exploration points performed at the Sites for the CSU project and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by BSK during construction. BSK should be notified at least two weeks prior to the start of construction and prior to when observation and testing services are needed.

7. ADDITIONAL SERVICES AND LIMITATIONS

7.1 Additional Services

The review of plans and specifications, and field observation and testing during construction by BSK are an integral part of the conclusions and recommendations made in this report. If BSK is not retained for these services, the client will be assuming BSK's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein. The recommended tests, observations, and consultation by BSK during construction include, but are not limited to:

- review of plans and specifications;
- observations of site grading, including stripping and engineered fill construction;
- observation of foundation and below-grade wall excavations;
- observation of ground improvement operations (if applicable);
- observation of levee widening operations, including keyway excavations and levee fill placement; and
- in-place density testing of fills, backfills, and finished subgrades.

7.2 Limitations

The recommendations contained in this report are based on our field observations and current and previous subsurface exploration, limited laboratory tests, review of available geologic maps and publications, and our present knowledge of the proposed construction. It is possible that soil conditions could vary between or beyond the points explored. If soil conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the Site area at the time of our study. No warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by BSK during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference", as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the project Sites have changed. If this report is used beyond this period, BSK should be contacted to evaluate whether site conditions have changed since the report was issued.

Also, land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, BSK may recommend that additional work be performed and that an updated report be issued.

The scope of services for this subsurface investigation and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the air, soil, surface water, or groundwater at the project Sites.

BSK conducted subsurface exploration and provided recommendations for this project. We understand that BSK will be given the opportunity to perform a formal geotechnical review of the final project plans and specifications. In the event BSK is not retained to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted, we will assume no responsibility for misinterpretation of our recommendations.

We recommend that all earthwork during construction be monitored by a representative from BSK, including site preparation, foundation excavation, ground improvement (if applicable), placement of engineered fill, levee fill widening operations, and trench/wall backfill. The purpose of these services would be to provide BSK the opportunity to observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

FIGURES

References: 1. https://www.arcgis.com/apps/mapviewer/index.html, 2023

Note: Location is approximate

BSK ASSOCIATES

PROJECT NO.	**G00000357**
DRAWN:	06/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

VICINITY MAP

Chemical System Upgrade
Ellis Creek Water Recycling Facility (WRF)
Petaluma, California

FIGURE 1

References: 1. http://earth.google.com, 2023

Legend

B-1 Approximate Boring Location (BSK, 2023)

CPT-1 Approximate Cone Penetration Test Locations (BSK, 2023)

B-1 Approximate Boring Location (RGH Consultants, 2012)

EB-3 Approximate Boring Location (Harza, 2001)

EB-1 Approximate Bloring Location (Harza, 2001)
Converted into Stand Pipe Piezometers

HB-1 Approximate Boring Location (Harding Lawson, 1995)

EB-24 Approximate Boring Location (Fugro West, 2002)

CPT-1 Approximate CPT Location (Harza, 2001)

MT71-10 Approximate Boring Locations (Moore and Taber, 1971)

Approximate Extent of Bay Mud
(inferred from exploration points)

BSK ASSOCIATES

PROJECT NO.	G00000357
DRAWN:	06/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	SitePlan.indd

SITE PLAN

Chemical System Upgrade
Ellis Creek Water Recycling Facility (WRF)
Petaluma, California

FIGURE

2

References: 1. http://earth.google.com, 2023

Note:1. All locations are approximate

Legend

B-1 Approximate Boring Location (BSK, 2023)

CPT-1 Approximate Cone Penetration Test Locations (BSK, 2023)

BSK ASSOCIATES

PROJECT NO.	**G00000357**
DRAWN:	06/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

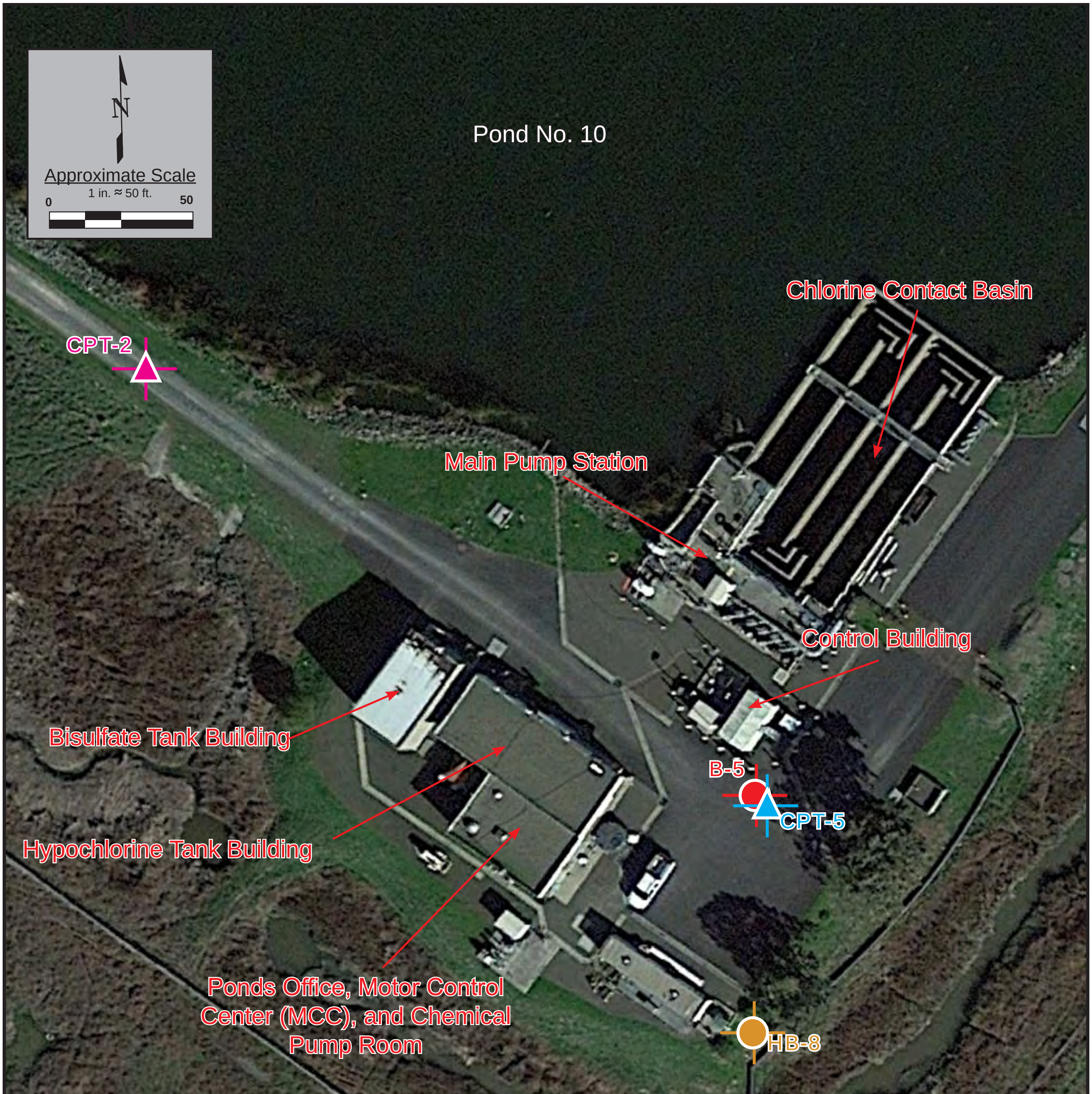
SITE PLAN

Wetlands Effluent Pump Station (WEPS)

Chemical System Upgrade
Ellis Creek Water Recycling Facility (WRF)
Petaluma, California

FIGURE

3



References: 1. http://earth.google.com, 2023

Note:1. All locations are approximate

Legend

B-1 Approximate Boring Location (BSK, 2023)

CPT-1 Approximate Cone Penetration Test Locations (BSK, 2023)

HB-8 Approximate Boring Location (Harding Lawson, 1995)

CPT-2 Approximate CPT Location (Harza, 2001)

BSK ASSOCIATES

PROJECT NO.	G00000357
DRAWN:	06/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

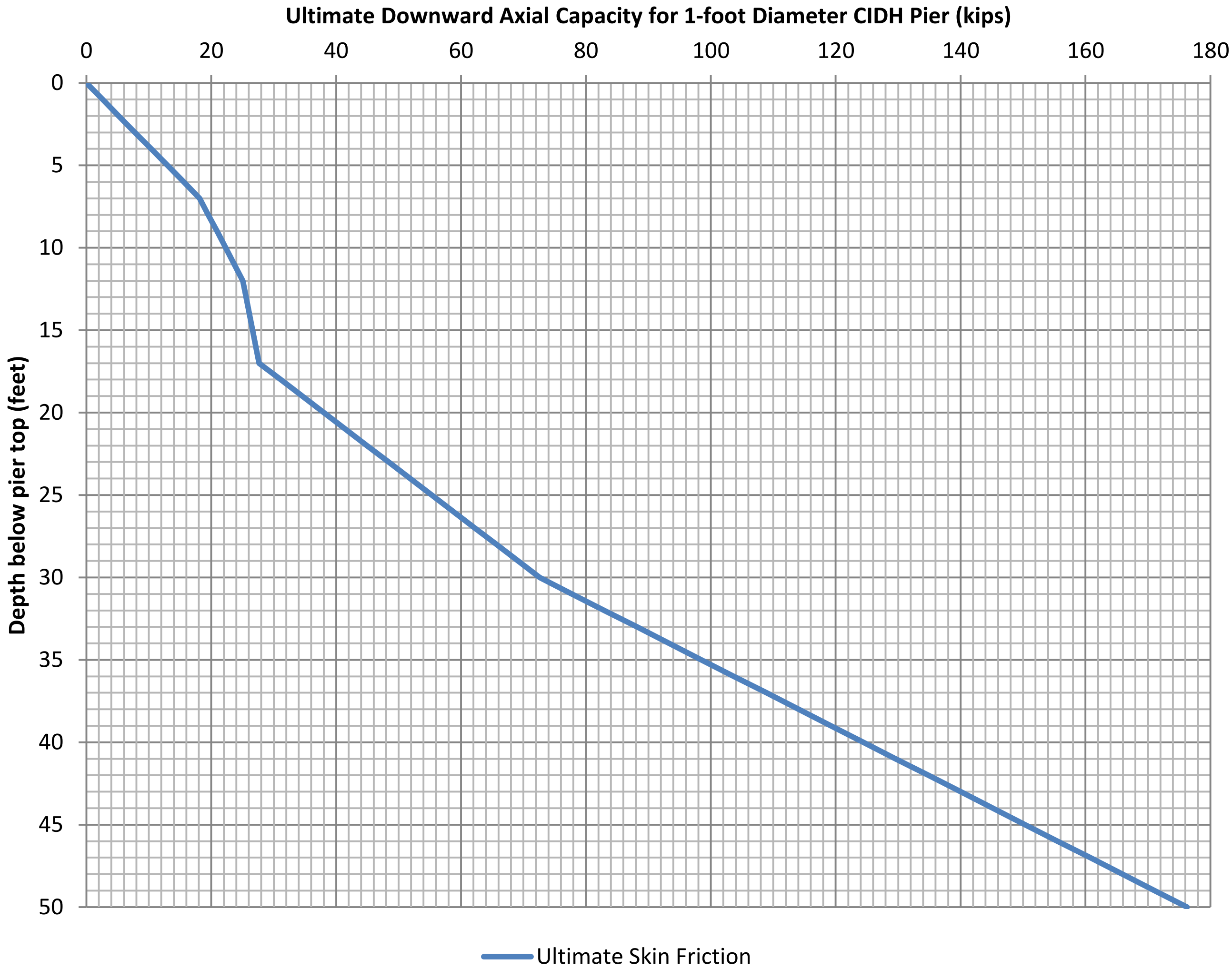
SITE PLAN

Chemical Processing Area

Chemical System Upgrade
Ellis Creek Water Recycling Facility (WRF)
Petaluma, California

FIGURE

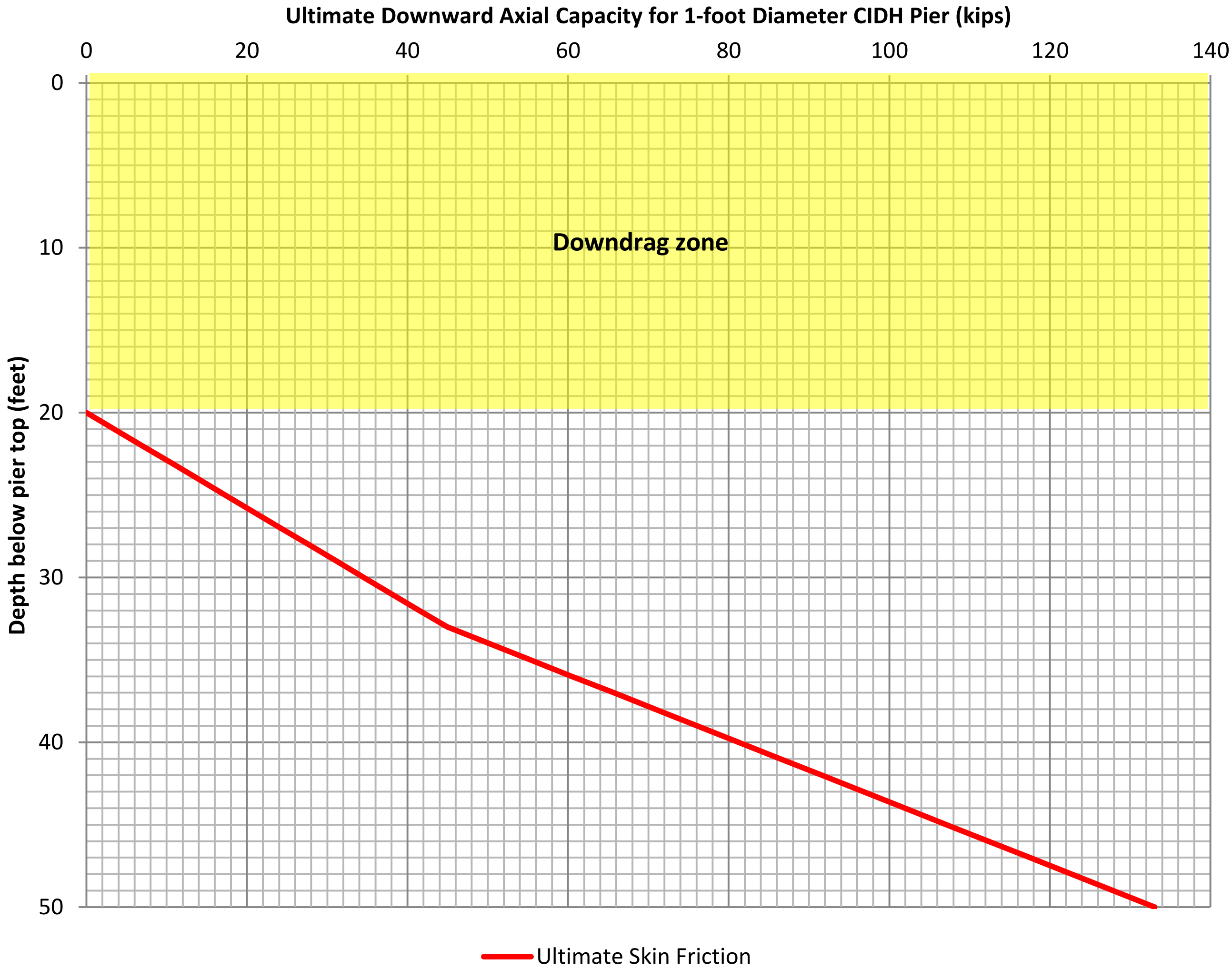
4



Notes:

1. We recommend applying factors of safety of 2 and 1.5 to the ultimate values provided in the plot above for static and transitent loading (wind/seismic), respectively.
2. The downard axial capacity of a CIDH pier having a diameter larger than 1-foot may be obtianed by multiplying the capacity for the 1-foot diameter pier presented in the plot above by the desired pier diameter in units of feet. As an example, for an 18-inch CIDH pier, multiply the above values by 1.5.
3. Piers must be spaced at least 3 diameters apart (center to center) for axial capacities presented here to be valid.
4. The uplift axial capacity may be obtained by multiplying the downward skin friction values presented in the plot above by 2/3. The pier weight can also be added to the uplift capacity.
5. **This plot should <u>only</u> be used if the levees are not raised (i.e., no new fill is placed over the crest of the levees) or if the CIDH piers are installed 2+ years after fill is placed to raise the levees.**

BSK ASSOCIATES				
	PROJECT NO.	G00000357	**Ultimate Axial Capacity**	FIGURE
	DRAWN:	6/27/2023		
	DRAWN BY:	C. Melo	**24-inch CIDH Piers (No Down Drag)**	**5**
	CHECKED BY:	C. Foulk		
	FILE NAME:	FIGURE 5-6	**Chemical System Upgrade Ellis Creek Recycling Facility (WRF) Petaluma, California**	



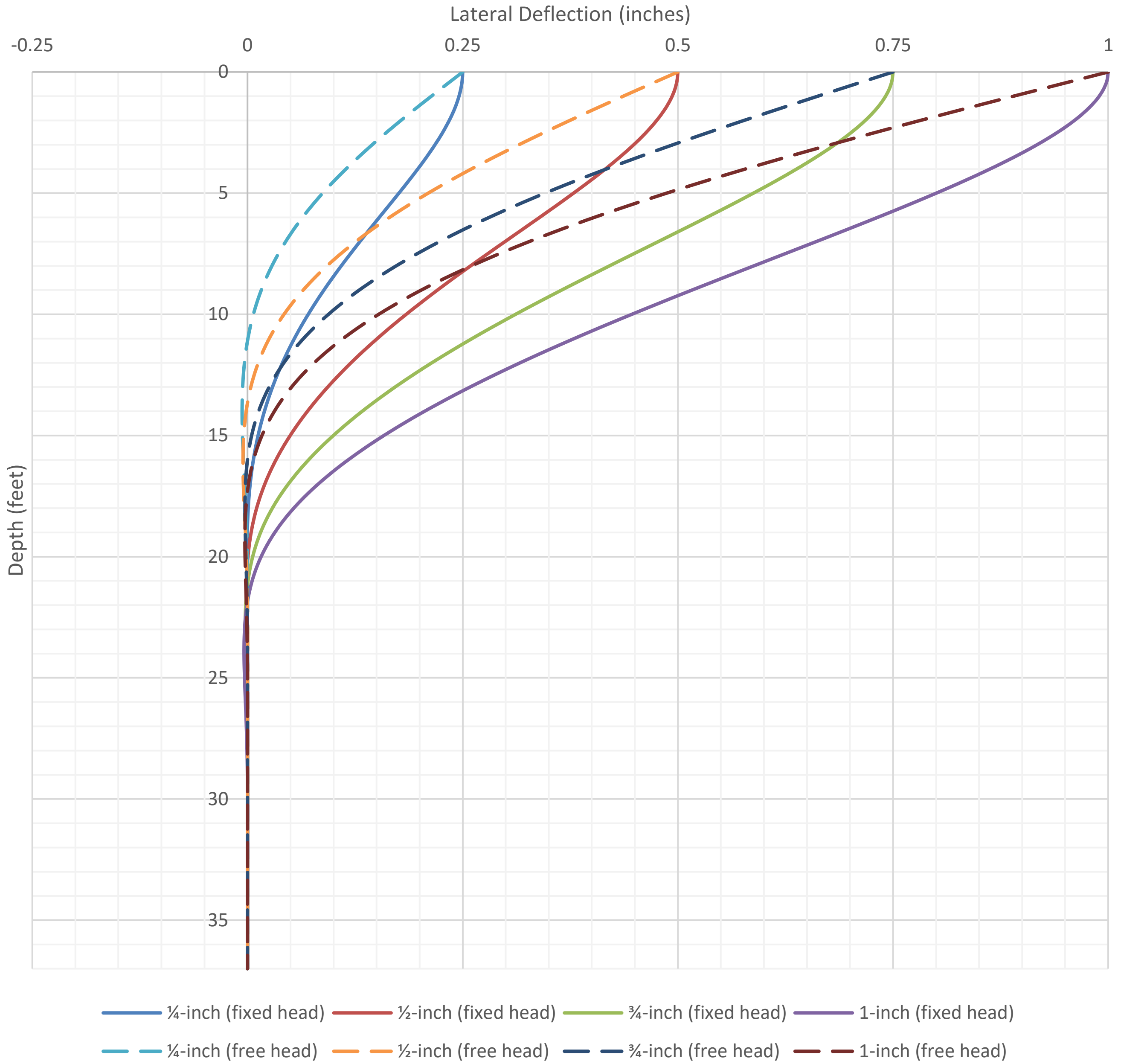
Notes:

1. We recommend applying factors of safety of 2 and 1.5 to the ultimate values provided in the plot above for static and transitent loading (wind/seismic), respectively.
2. The downard axial capacity of a CIDH pier having a diameter larger than 1-foot may be obtianed by multiplying the capacity for the 1-foot diameter pier presented in the plot above by the desired pier diameter in units of feet. As an example, for an 18-inch CIDH pier, multiply the above values by 1.5.
3. Piers must be spaced at least 3 diameters apart (center to center) for axial capacities presented here to be valid.
4. The uplift axial capacity may be obtained by multiplying the downward skin friction values presented in the plot above by 2/3. The pier weight can also be added to the uplift capacity.
5. An unfactored (i.e., ultimate) drag load of 35D kips (where D is the pier diameter in feet) should be added to the design loads. **This load should be applied <u>unfactored</u> for the design of the piers.**

6. This plot should be used if the CIDH piers are installed less than 2 years after fill is placed to raise the levees.

BSK ASSOCIATES

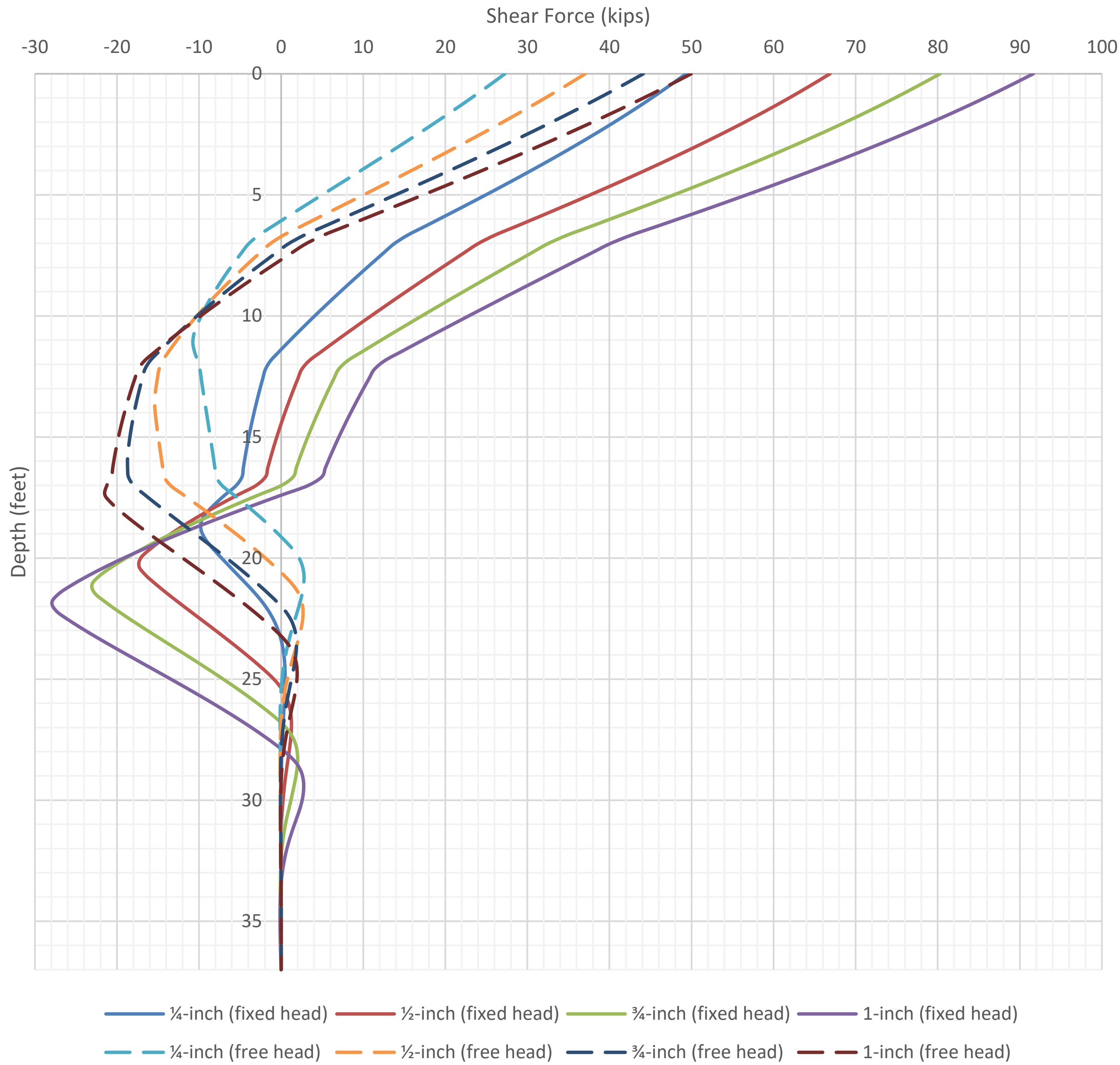
PROJECT NO. G00000357	**Ultimate Axial Capacity**	**FIGURE**
DRAWN: 6/27/2023		
DRAWN BY: C. Melo	**24-inch CIDH Piers (With Down Drag)**	**6**
CHECKED BY: C. Foulk		
FILE NAME: FIGURE 5-6	**Chemical System Upgrade Ellis Creek Recycling Facility (WRF) Petaluma, California**	



Notes:

1. Plots are based on unfactored values. Designer should consider applying resistance factors per AASHTO to the results.
2. Minimum pier depth to achieve fixity is 37 feet.

BSK ASSOCIATES				
PROJECT NO.	G00000357	Lpile Deflection Curves 24-inch CIDH Piers	FIGURE
DRAWN:	06/27/23		7
DRAWN BY:	C. Melo		
CHECKED BY:	C. Foulk	Chemical System Upgrade Ellis Creek Recycling Facility (WRF) Petaluma, California	
FILE NAME:	Figures 7-9		

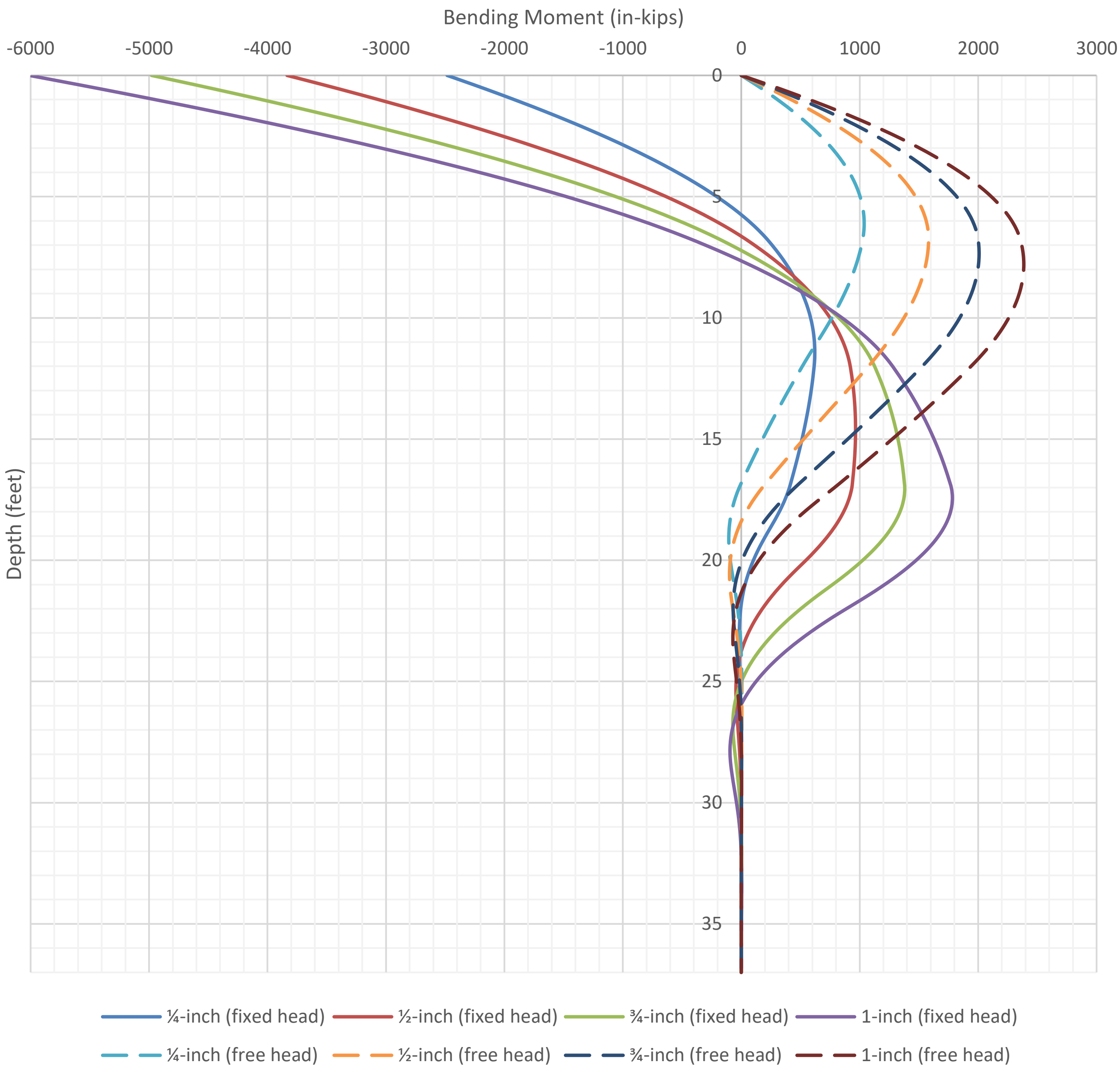


Notes:

1. Plots are based on unfactored values. Designer should consider applying resistance factors per AASHTO to the results.
2. Minimum pier depth to achieve fixity is 37 feet.

BSK ASSOCIATES

PROJECT NO. G00000357	**Lpile Shear Force Curves** **24-inch CIDH Piers**	**FIGURE**
DRAWN: 06/27/23		**8**
DRAWN BY: C. Melo		
CHECKED BY: C. Foulk	**Chemical System Upgrade** **Ellis Creek Recycling Facility (WRF)** **Petaluma, California**	
FILE NAME: Figures 7-9		



Notes:

1. Plots are based on unfactored values. Designer should consider applying resistance factors per AASHTO to the results.
2. Minimum pier depth to achieve fixity is 37 feet.

BSK ASSOCIATES				
PROJECT NO.	G00000357	Lpile Bending Moment Curves	PLATE
DRAWN:	06/27/23	24-inch CIDH Piers	
DRAWN BY:	C. Melo		9
CHECKED BY:	C. Foulk	Chemical System Upgrade	
FILE NAME:	Figures 7-9	Ellis Creek Recycling Facility (WRF) Petaluma, California	

APPENDIX A

June 26, 2023 Geotechnical Report by BSK Associates

ASSOCIATES

REVISED GEOTECHNICAL INVESTIGATION REPORT
OXIDATION POND TRANSFER STRUCTURE REHABILITATION AND OXIDATION POND STORAGE EXPANSION
ELLIS CREEK WATER RECYCLING FACILITY
PETALUMA, CALIFORNIA

BSK PROJECT NO.: G00000075

PREPARED FOR:

DUDEK
1630 SAN PABLO AVENUE, SUITE 300
OAKLAND, CALIFORNIA 94612

June 26, 2023

DUDEK PETALUMA SONOMA COUNTY CALIFORNIA-USA

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Livermore CA 94551
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www.bskassociates.com

Sent via email: pgiori@dudek.com

June 26, 2023 BSK Proposal No. G00000075

Mr. Phillip Giori, PE
Dudek
1630 San Pablo Avenue, Suite 300
Oakland, California 94612

SUBJECT: **Revised Geotechnical Investigation Report**
Oxidation Pond Transfer Structure Rehabilitation and Oxidation Pond Storage Expansion
Ellis Creek Water Recycling Facility
Petaluma, California

Dear Mr. Giori:

BSK Associates (BSK) is pleased to submit our geotechnical investigation report for the above-referenced project at the City of Petaluma (City) Ellis Creek Water Recycling Facility located at 3890 Cypress Drive in Petaluma, California. The enclosed report describes our geotechnical investigation performed along the levees for the oxidation ponds, and our conclusions and geotechnical design recommendations for the project. This revised report supersedes our original report for this project dated June 14, 2023.

In summary, it is our opinion that the project site (Site) is suitable for the proposed improvements provided the recommendations presented in this report are incorporated in the design and construction of the project. The primary geotechnical concerns at the Site are the potential for strong ground shaking to affect the Site during a future significant seismic event (typical of California), the presence of shallow groundwater and associated hydrostatic and buoyancy pressures, the presence of highly compressible Bay Mud and high organic content soils containing peat, and the presence of highly expansive surficial soils. The impact of these concerns on the project and ways to design/mitigate for them are discussed in the report.

The conclusions and recommendations presented in the enclosed report are based on limited subsurface investigation and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil conditions may be found in localized areas during construction. If significant variation in the subsurface conditions is encountered during construction, BSK should review the recommendations presented herein and provide supplemental recommendations if necessary.

Additionally, design plans should be reviewed by our office prior to their issuance for conformance with the general intent of our recommendations presented in the enclosed report.

We appreciate the opportunity of providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please contact us at (925) 315-3151.

Sincerely,
BSK Associates, Inc.

Omar K. Khan, GIT
Project Geologist

Cristiano Melo, PE, GE #2756
Livermore Branch Manager

REGISTERED PROFESSIONAL ENGINEER
CRISTIANO B.C. DEMELO
No. 2756
Exp. 09/30/24
GEOTECHNICAL
STATE OF CALIFORNIA

Carrie L. Foulk, PE, GE #3016
Geotechnical Group Manager

Table of Contents

Figures and Appendices

1. INTRODUCTION

This report presents the results of our geotechnical investigation for the planned oxidation pond transfer structure rehabilitation and oxidation pond storage expansion at the Ellis Creek Water Recycling Facility (ECWRF) located at 3890 Cypress Drive in Petaluma, California. A Vicinity Map showing the location of the project site (Site) is presented on Figure 1. This report contains a description of our site investigation methods and findings along the levees for the oxidation ponds, including field and laboratory data. Based on these findings, this report presents conclusions regarding the geotechnical concerns for the planned improvements. This revised report supersedes our original report for this project dated June 14, 2023.

1.1 Site and Project Description

The Ellis Creek Water Recycling Facility is located at the southern end of Petaluma along the southwest side of Lakeville Highway (Highway 116) and Browns Lane within the floodplain of the Petaluma River. As shown on the Site Plan, Figure 2, the oxidation ponds are situated at the southeastern area of the ECWRF. The Ellis Creek separates the oxidation ponds from the main facilities of the ECWRF. The Site consists of 11 ponds – eight (8) designated as oxidation ponds (labeled Ponds No. 1 through 8), two (2) designated as wetland ponds (labeled Ponds No. 9 and 10), and one designated as an aerated lagoon adjacent to Pond No. 1. For simplicity, we refer to these 11 ponds as the “oxidation ponds” throughout this report. The wetlands effluent pump station (WEPS) is located on the northwest side of Pond No. 9, while the chlorination building and other improvements associated with the chemical processing area are located at the southern corner of Pond No. 10. The oxidation ponds are separated by a drainage canal that drains from the northeast near State Route 116 to the southwest into Petaluma River. On either side of the drainage canal are five oxidation ponds. The oxidation ponds are surrounded by unlined earthen dams (i.e., levees) that range from about 7 to 10 feet in height and have side slopes typically ranging from approximately 3H:1V (horizontal to vertical) to 5H:1V. The width at the top of the levees ranges from about 15 to 20 feet. The top of the levees act as vehicular pathways in between the oxidation ponds. Most of the slopes are lined with rock slope protection (RSP) on the pond side (inboard side) of the levees and the vehicular pathways are lined with aggregate base and/or dirt, except for the asphalt paved roadway connecting the chemical processing area to Highway 116 along the southeastern side of Ponds No. 2, 3, 6, 7, and 10. Based on the current (undated) elevation topographic map of the Site provided to us by Dudek (the lead designer for this project), the elevation at the top of the oxidation pond levees ranges from about 13 to 23 feet.

According to historic aerial photographs and historic topographic maps, the Site area was originally a marsh land/floodplain associated with the Petaluma River until about 1947. By 1955, the area was used for agriculture until about 1975. According to the geotechnical report by Fugro dated April 2005 (see detailed reference in the “Previous Investigations” section below), the oxidation ponds were constructed in 1972 by a combination of excavating and placing fill over the native alluvial and marsh deposits.

The project consists of rehabilitation of the concrete flow transfer structures which allow treated/untreated wastewater to flow in between the oxidation ponds. The flow transfer structures are comprised of a reinforced concrete basin structure with a 48-inch cast iron gate valve to control flow, 24-inch corrugated metal pipe (CMP) inflow piping, and 48-inch CMP outflow piping. The flow structures are nearing the end of their useful life and need rehabilitation. In December of 2021, a sinkhole was discovered on the levee roadway between the flow transfer structure on the aerated lagoon and Pond No. 4. The 48-inch CMP discharge piping of the flow transfer structure had experienced catastrophic failure as the entire top half of the pipe had corroded, causing undermining of the vehicular pathway over the levee. **Based on our recent communication with Dudek, the rehabilitation of the transfer structures will consist of lining the interior of portions of the existing 48-inch CMPs with carbon-fiber-reinforced-polymer, demolishing portions of the existing 24- to 48-inch CMPs, and replacing them with new 42- to 48-inch HDPE pipes as depicted in the draft repair drawings presented in Appendix E. Replacement of the pipes will require excavation and backfill within the levee cross section. None of the reinforced concrete basin structures are expected to be replaced.**

The project also includes raising the levees about 2 to 3 feet in vertical height in order to increase the storage capacity of the existing oxidation ponds and address rising sea levels.

If the actual site and project descriptions differ significantly from that anticipated above, we should be notified so that we may review our proposed scope of services presented herein for applicability.

1.2 Purpose and Scope of Services

The purpose of this investigation was to explore and evaluate the subsurface conditions at the Site to provide geotechnical input for the design and construction of the planned improvements. The scope of services, as outlined in our July 29, 2022 proposal (Proposal No. G00000075), consisted of pre-field activities, field investigation, laboratory testing, engineering analysis, and preparation of this report.

Our investigation specifically excludes the assessment of site environmental characteristics, particularly those involving hazardous substances. Our scope of services did not include the evaluation of contaminants in the soil, water, or air.

1.3 Previous Investigations

Previous investigations were performed within the levees for the oxidation ponds at the ECWRF by other subconsultants. These investigations were presented in the following documents:

1. Fugro West, Inc. (Fugro, 2005), Integrated Geotechnical Study, Lakeville Highway WRF – Parcel A, Petaluma, California, dated April 29, 2005 (Fugro West Project No. 3045.022). **This report included numerous previous exploration points performed by Harza in 2001 (Harza was acquired by Fugro in the early 2000's) as well as tabulated logs for borings performed in 1971 and 1995 by Moore and Taber and Harding Lawson**, and

2. RGH Consultants (RGH, 2012), Limited Geotechnical Study, Ellis Creek Oxidation Ponds 7 and 10, Sheet Pile Levee Project, Petaluma, California, dated December 4, 2012 (RGH Consultants Project No. 2553.08.04.1).

Pertinent information from these previous reports was considered in the preparation of this report. Available boring logs and lab data from these previous investigations that are proximate to the Site are included in Appendix D. The approximate locations of the previous exploration points are shown on Figure 2.

2. SITE INVESTIGATION

2.1 Field Investigation

Our field investigation was performed on February 16th, April 13th, and April 14th, 2023 to evaluate the subsurface conditions along the oxidation pond levees. The field investigation consisted of advancing five (5) Cone Penetration Tests (CPTs), labeled CPT-1 through CPT-5, to a depth of approximately 50 to 95 feet below the existing ground surface (BGS)[1] each and drilling five (5) borings, labeled B-1 through B-5 to depths of approximately 16½ to 31½ feet BGS. The approximate locations of these exploration points are shown on Figure 2.

Prior to the subsurface exploration, Underground Service Alert (USA North 811) was notified to provide utility clearance, each exploration location was cleared for detectable underground utilities by a private utility locator, and assistance from the ECWRF and City of Petaluma (City) personnel via existing as-built plans. Nevertheless, while drilling boring B-3 on February 16, 2023, we damaged an existing underground utility line (which was subsequently repaired by the City) at a depth of about 7 to 8 feet BGS, which resulted in postponement of further investigation of the Site until April 13th. A drilling permit was obtained from the Sonoma County Department of Environmental Health Services (County). Upon completion of the field investigation, the borings and CPTs were backfilled with cement grout and capped with excess soil except for boring B-5, which was capped with rapid set concrete. Excess soil cuttings generated by the borings during drilling operation were spread out over unimproved areas of the Site.

The locations of our exploration points were estimated by our field representative based on rough measurements from existing features at the Site. The elevations shown on the boring logs were estimated using the elevation contours contained in the current (undated) elevation topographic map of the Site provided to us by Dudek. As such the elevations and locations of the exploration points should be considered approximate to the degree implied by the methods used.

2.1.1 Auger Borings

The borings were drilled using a truck-mounted drill rig equipped with hollow stem and solid stem augers to depths of about 16½ to 31½ feet BGS by Taber Drilling of West Sacramento, California. The borings were logged by a BSK field geologist. Relatively undisturbed samples of the subsurface materials were obtained using a split spoon sampler with a 2.5-inch inside diameter (I.D.) and a 3-inch outside diameter (O.D.) fitted with stainless steel liners and a 3-inch I.D. Shelby Tube. Disturbed samples were obtained from the auger cuttings and from a Standard Penetration Test (SPT) split spoon sampler with a 1.4-inch I.D. without stainless steel liners. Except for the Shelby Tube, the samplers were driven 18 inches using a 140-pound, automatic hammer falling 30 inches, and blow counts for successive 6-inch penetration intervals were recorded. The blow counts were reported on the final boring logs. The Shelby Tube was

[1] Any reference made to "below the existing ground surface (BGS)" throughout this report refers to the ground surface at the crest of the existing levees for the oxidation ponds.

pushed into the ground using the hydraulic pressure exerted by the drilling equipment. After the sampler was withdrawn from the boreholes, the samples were removed, sealed to reduce moisture loss, labeled, and returned to our laboratory. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs.

Soil classifications made in the field, based on visual/manual assessment of the auger cuttings and samples, were re-evaluated in the laboratory after further examination and testing. Where laboratory tests were performed, most of the test results appear in the final boring logs (refer to the "Laboratory Testing" section below for further details). Final soil classification was assessed through the judgement of a responsible Geotechnical Engineer supplemented with the laboratory testing at various intervals in general accordance with the ASTM *Standard Practice for Classification of Soils for Engineering Purposes* (D2487). A summary of the Unified Soil Classification System (USCS), adapted by ASTM D2487 and D2488 is presented in Appendix A, Figure A-1. The Soil Description Key and Log Key are presented on Figure A-2 and A-3. Sample classifications, blow counts and hydraulic pressure recorded during sampling, and other related information are presented on the soil boring logs within Appendix A. Discussion of the subsurface conditions encountered at the Site is presented in the "Subsurface Conditions" section of this report.

2.1.2 Cone Penetration Tests

Cone penetration test probes were advanced to depths of approximately 50 to 95 feet BGS. Taber Drilling of West Sacramento, California was subcontracted to provide CPT services. The CPTs were performed using an integrated electronic cone system in accordance with ASTM D3441. The cone has a tip area of 10 square centimeters, a friction sleeve area of 150 square centimeters, and a ratio of end area friction sleeve to tip end area equal to 0.80. The cone resistance and sleeve friction were measured and recorded during the tests at approximately 5-centimeter (about 2 inches) depth intervals. In addition, shear wave velocity measurements were taken every 5 feet at CPT-1 and CPT-5. Pore pressure dissipation tests were also performed for CPT-1 and CPT-3.

The cone system was pushed using a 40,000-pound, all-wheel drive, CPT rig, having a down pressure capacity of approximately 20 tons. The information gathered from the CPTs was used for identifying potentially liquefiable and soft soils and for foundation design. The correlated CPT data collected from our CPT (cone resistance, friction ratio, pore pressure, and soil behavior type) versus penetration depth below the existing ground surface, generated using the CPT liquefaction assessment computer software CLiq[2], is presented in Appendix B along with shear wave velocity measurements and pore pressure dissipation test results.

The stratigraphic interpretation of the CPT data was performed based on relationships between cone resistance (also known as tip resistance) and sleeve friction versus penetration depth. The friction ratio,

[2] CLiq v2.0 by Geologismiki

which is sleeve friction divided by cone resistance, is a calculated parameter which is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone resistance and generate small excess pore water pressures. The interpretation of soil properties from the cone data has been carried out using correlations developed by Robertson et al, 1990[3], and Lunne, Robertson & Powell, 1997[4]. It should be noted that it is not always possible to clearly identify a soil type based on cone resistance and sleeve friction. In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included dry unit weight and moisture content, Atterberg limits, unconsolidated-undrained triaxial compression (TXUU), organic content, consolidation, and moisture density relationship (i.e., compaction curve) testing. Most of the laboratory test results are presented on the boring logs. The results of the Atterberg limits, TXUU, consolidation, and moisture density relationship tests are presented graphically in Appendix C.

Analytical testing was performed on samples obtained from depths of about 0 to 15½ feet BGS in borings B-3 and B-5 to assist in evaluating the corrosion potential of the near-surface soils at the Site. The corrosion results are presented at the end of Appendix C and were performed by CERCO Analytical of Concord, California using ASTM methods.

[3] Robertson P.K., 1990. Soil classification using the cone penetration test. Canadian Geotechnical Journal, 27(1): 151-158

[4] Lunne, T., Robertson, P.K., and Powell, J.J.M 1997. Cone penetration testing in geotechnical practice, E & FN Spon Routledge, 352 p, ISBN 0-7514-0393-8.

3. SITE GEOLOGY AND SEISMICITY

The City of Petaluma is located within the Petaluma River Valley immediately north of San Pablo Bay. The valley is located within a structural depression that trends northwest and is part of the Coast Ranges geomorphic province. The valley extends from San Pablo Bay northward to a series of low hills near the town of Penngrove. The valley is bounded on the west by the Mendocino Range and on the east by the Sonoma Mountains. A few northwest trending folds and a faults are the most important geologic structures of the Petaluma River Valley. The Petaluma River is the principal stream draining the Petaluma River Valley and is tidally influenced from its mouth at San Pablo Bay upstream to the town of Petaluma. The valley is comprised of late Tertiary to Quaternary age sedimentary deposits of marine and continental origin and volcanic rocks. According to the California Geological Survey (CGS, 2002[5]) and as shown in Exhibit 1 below, the Site is underlain by Holocene alluvial fan deposits (map symbol Qhf) and Holocene Bay Mud (map symbol Qhbm). These Holocene deposits are described by the CGS as follows:

- Holocene Bay Mud – silt, clay, peat, and fine sand deposited at or near sea level in San Pablo Bay. This soil deposit is highly compressible and susceptible to high long-term consolidation settlement upon loading.
- Holocene alluvial fan deposits – sand, gravel, silt, and clay deposited by streams emanating from canyons onto alluvial valley floors. Sediment is poorly to moderately sorted and bedded.

According to Figure 9-4 of the Sonoma County Multijurisdictional Hazard Mitigation Plan Update 2021[6], both of these geologic units have been assigned a moderate liquefaction susceptibility.

The City and the Site are located within a highly seismic area of the greater San Francisco Bay Area. The seismic activity within the Bay Area is associated with the San Andreas Fault System which constitutes one of Earth's major tectonic plate boundaries, separating the North American and Pacific tectonic plates. The two plates are moving past each other in a right lateral sense. Stresses built up by plate motion are periodically released predominately by strike slip movement along the San Andreas Fault System, which in the Bay Area includes the San Andreas Fault, Hayward Fault, Calaveras Fault, and other associated active faults. The nearest of these active faults to the Site are the Rodgers Creek and San Andreas located approximately 3 miles to the northeast and 17 miles to the southwest, respectively. These faults are zoned and considered active by the CGS. Approximately ½-mile northeast and one mile south of the Site are the Tolay and Lakeview Faults, respectively, which are not zoned or considered active by the CGS. According to the CGS, the Site is not within a state designated Alquist-Priolo Earthquake Fault Zone and no mapped faults are known to traverse the Site. However, due to proximity to active faults in the region, the Site will

[5] California Geological Survey Staff (2002), Geologic Map of the Petaluma River 7.5' Quadrangle, Marin and Sonoma Counties, California: A Digital Database.
[6] Sonoma County (2021), Sonoma County Multijurisdictional Hazard Mitigation Plan Update 2021, Volume 1, October 2021.

likely be subjected to moderate to intense ground shaking from a future significant earthquake on the aforementioned faults or other active faults in the Bay Area.

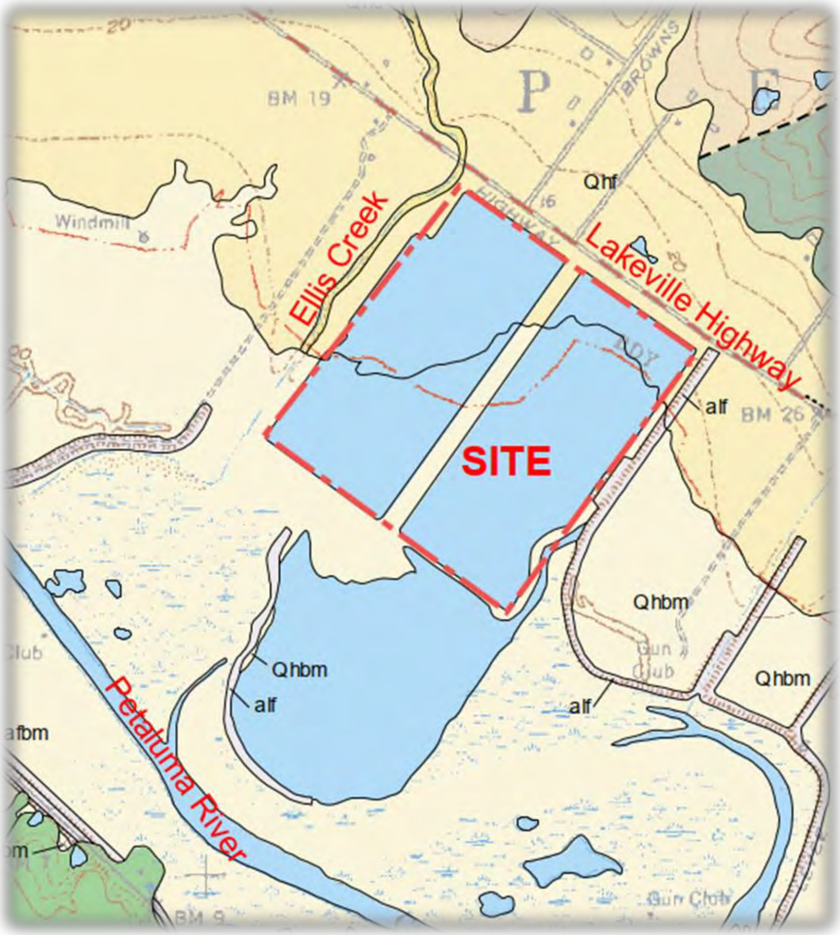

Exhibit 1 – Site Geology Map (CGS, 2002)

4 SUBSURFACE CONDITIONS

4.1 Current Subsurface Data

Below is a general description of the soil conditions encountered at the Site. For a more detailed description of the soils encountered, refer to the current boring logs in Appendix A, current CPT logs in Appendix B, and previous subsurface data in Appendix D. It should be noted that subsurface conditions can deviate from those conditions encountered in the current and previous investigations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments, as necessary.

According to our current borings and CPTs, the Site is underlain by levee fill and native soils. The fill is present in the upper approximately 6 to 15 feet BGS and generally consists of firm to hard lean and fat clays interbedded with very loose to medium dense clayey sand.

Immediately beneath the fill, approximately 6 to 10 feet of soft to firm Bay Mud consisting primarily of lean and fat clay was encountered in borings B-3, B-4, and B-5. Based on our pocket penetrometer and TXUU test results, the upper portion of the Bay Mud layer has a higher shear strength than the lower portion of the Bay Mud Layer. This is attributed to desiccation of the Bay Mud due to repeated cycles of rising and falling groundwater in marsh lands and exposure to sunshine and wind. As a result, the upper portion of the Bay Mud layer is commonly referred to as “Bay Mud Crust”, which typically has significantly higher strength than regular Bay Mud and is less susceptible to high consolidation. According to NAVFAC 7.01[7], soils having an organic content by weight of less than 5 percent are slightly organic, while soils having an organic content between 5 and 30 percent are considered to be organic soils. Soils having an organic content of over 30 percent are considered as highly organic and classified as peat. Organic or peat layers (within the Bay Mud layer that underlies the southwest portion of the Site) were encountered in boring B-4 with an organic content of approximately 13 percent from a depth of about 14 to 20 feet BGS and in boring B-5 with an organic content of approximately 37 percent from a depth of about 14½ to 16½ feet BGS. Bay Mud was also encountered in CPT-2 from about 10 to 19 feet BGS, CPT-3 from about 12 to 16 feet BGS, and CPT-5 from about 12 to 21 feet BGS.

Below the levee fill in borings B-1 and B-2 and the Bay Mud layer in borings B-3, B-4, and B-5, our borings generally encountered firm to hard lean and fat clays with some interbedded layers of loose to medium dense clayey to poorly graded sand to the maximum depth of our borings (approximately 31½ BGS). Below the levee fill and the Bay Mud layer, our CPTs generally encountered firm to hard clayey soils with some interbedded layers of medium dense to very dense sandy layers to the maximum depth of our CPTs (approximately 95 feet BGS).

[7] Naval Facilities Engineering Command (NAVFAC), Design Manual 7.01, Revalidated by Change 1 September 1986.

4.2 Previous Subsurface Data

As shown on Figure 2, various previous exploration points consisting of borings and CPTs were performed by other consultants along the ponds. A summary of their investigations and findings is provided below.

- RGH (2012)
    - Drilled five (5) borings (B-1 through B-5) along the levee between Ponds No. 7 and 10 to depths ranging from about 20 to 25 feet BGS. The borings were drilled with a truck-mounted drill rig equipped with 6-inch diameter solid augers.
    - The borings encountered stiff (firm) levee fill soils that extended to depths of about 13 to 14 feet BGS. Below the levee fill, the borings encountered medium stiff (soft) Bay Mud, which contained intermittent thin layers of peat and fine sand. Free groundwater was observed at depths of approximately 13½ and 15 feet BGS in borings B-1 and B-5, respective.
- Fugro (2005) – includes exploration points by Harza (2001), Harding Lawson (1995), and Moore and Taber (1971)
    - In 2002, numerous soils borings and CPTs were performed along the levees for the oxidation ponds. In the area of Ponds No. 9 and 10, previous explorations encountered 9 to 13 feet of fill consisting of stiff to very stiff (firm to hard) clayey soils underlain by approximately 7- to 12-foot-thick layer of soft to firm clay (Bay Mud). The thickness of the Bay Mud increased from the western end of Pond 9 to the eastern end of Pond 10. Below the Bay Mud, stiff to very stiff (firm to hard) clayey soils interbedded with medium dense to dense sand was encountered at various depths to the maximum depth explored (about 80 feet BGS).
    - In the area of Pond No. 1/Aerated Lagoon, previous borings encountered approximately 14 feet of fill consisting of stiff (firm) clayey soil underlain by stiff (firm) to hard clayey soil interbedded with medium dense to very dense clayey sand and gravel to the maximum depth explored (about 40 feet BGS).
    - The above subsurface conditions are in general agreement with the subsurface conditions encountered in the borings performed in 1971 and 1995 by Moore and Taber and Harding Lawson.
    - Free groundwater was observed at depths of 7½ to 24½ feet BGS in the previous borings by Fugro and Harza. However, free groundwater was observed at a depth of 3 feet BGS at boring EB-2. According to piezometer monitoring conducted by Fugro/Harza from 2001 to 2004, groundwater levels ranged from about 9 to 13 feet BGS within the oxidation ponds. Groundwater information was not provided in the Fugro (2005) report for the borings performed in 1971 and 1995 by Moore and Taber and Harding Lawson.

4.3 Groundwater

Free groundwater was observed at depths of approximately 10, 23, and 25 feet BGS in borings B-3, B-4 and B-5, respectively. Based on pore pressure dissipation tests, groundwater was encountered at approximately 13 and 7 feet BGS in CPT-1 and CPT-3, respectively. Free groundwater was observed in the borings performed by Harza and Fugro in in the early 2000's at depths of about 7½ to 24½ feet BGS (except at boring EB-2 as previously discussed). According to piezometer monitoring conducted by Fugro/Harza from 2001 to 2004 (refer to Plate 8 shown in Appendix D), groundwater levels ranged from about 9 to 13 feet BGS within the oxidation ponds. Groundwater information was not provided in the Fugro (2005) report for the borings performed in 1971 and 1995 by Moore and Taber and Harding Lawson. Free groundwater was observed in the RGH (2012) borings at depths of about 13½ to 15 feet BGS.

It should be noted that groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties. Also, free groundwater levels shown in the current and previous exploration points may not be representative of stabilized groundwater levels.

4.4 Additional Discussion

The above is a general description of soil and groundwater conditions encountered at the Site. For a more detailed description of the soils encountered, refer to the boring and CPT logs in Appendices A, B, and D. It should be noted that subsurface conditions can deviate from those conditions encountered at the boring and CPT locations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments as necessary.

5. LIMITED SLOPE STABILITY ANALYSIS

We performed limited slope stability analysis for a generic levee cross section to evaluate existing conditions as well as the impact of adding 2 to 3 feet of new fill to raise the levees. The generic levee cross section was based on our review of the current elevation topographic map of the Site and the subsurface conditions encountered in the current and previous exploration points performed at the Site. The generic levee cross section assumed the following:

- The levee is 20 feet wide at the crest.
- The levee has side slopes with gradients of 3H:1V.
- The existing levee fill is 10 feet thick.
- The existing levee fill is underlain by 5 feet of Bay Mud Crust which is underlain by 5 feet of Bay Mud which is in turn underlain by firm/hard clay.

Our analysis consisted of evaluating the static and seismic slope stability for circular failure surfaces for this generic levee cross section. The soil shear strength parameters used in our analysis was based on the results of the TXUU testing performed on samples obtained from our borings and our past experience with Bay Mud. The table below summarizes the moist unit weights and shear strength parameters used in our slope stability analysis. The unit weights are based on the laboratory test results and our experience.

SUMMARY OF SOIL SHEAR STRENGTH PARAMETERS USED

Layer Description (depth[1], feet)	Moist Unit Weight (pcf)	Friction Angle (degrees)		Cohesion (psf)	
		Effective[2]	Total[3]	Effective[2]	Total[3]
Existing Levee Fill (0 to 10 feet)	100	20	15	300	400
Bay Mud Crust (10 to 15 feet)	85	0	0	800	800
Bay Mud (15 to 20 feet)	85	0	0	300	240[4]
Firm/Hard Clay (greater than 20 feet)	130	20	15	500	600

Notes:
1. Depth below the top of the levee.
2. The effective strength parameters (i.e., drained conditions) were used for static analysis, which represents long-term conditions.
3. The total strength parameters (i.e., undrained conditions) were used for seismic analysis, which represent short-term conditions.
4. Total shear strength of the Bay Mud layer was reduced by 20 percent to account for the potential of cyclic softening of this layer during a significant seismic event.

We assumed groundwater to lie 5 feet below the top of the levee. The limit-equilibrium Bishop simplified, Spencer, and Morgenstern and Price's methods and the slope stability program Slide (Version 7) were used in our analyses. Based on the methodology provided by Special Publication 117A[8], we used a

[8] California Geological Survey (2008), Guidelines for Evaluating and Mitigating Seismic Hazards in California: Special Publication 117A.

horizontal seismic coefficient of 0.20 in our analyses for pseudo-static (seismic) conditions, which was developed using the following parameters:

- Adjusted PGA_M of 0.569g (i.e., PGA_M of 0.68g divided by 1.5) as permitted by checklist No. 25 in CGS Note 48[9]. The PGA_M was obtained from the site-specific ground motion hazard analysis presented in Appendix F of this report.
- Earthquake moment magnitude of M7.22 per Appendix F.
- Fault distance of less than 10 km (about 6 miles).
- Displacement threshold of 15 centimeters (about 6 inches).

According to Special Publication 117A, a slope is considered stable when its factor of safety (FOS) is greater than or equal to 1.5 and 1.0 under static and seismic conditions, respectively.

The results of our limited slope stability analysis are summarized in the table below. Based on these results, the generic levee cross section for existing conditions and adding 2 to 3 feet of fill to raise the levees have Factors of Safety (FOS) greater than 1.5 under static conditions and 1.0 under seismic conditions and are considered globally stable. Note that the model used in our limited slope analysis is based on our interpretation of the current topographic map, field observations, current and previous subsurface exploration, and our past experience with similar subsurface conditions.

SLOPE STABILITY RESULTS FOR GENERIC LEVEE CROSS SECTION						
Slope Configuration	Stability Condition	Failure Surface Search	Estimated FOS[1]			Required FOS
			Bishop Simplified	Spencer	Morgenstern Price	
Existing Condition	Static (long-term)	Circular	3.01	3.00	3.00	1.5
	Seismic (short-term)		1.45	1.45	1.45	1
2 feet of fill added to the levee	Static (long-term)	Circular	2.34	2.33	2.33	1.5
	Seismic (short-term)		1.27	1.26	1.26	1
3 feet of fill added to the levee	Static (long-term)	Circular	2.11	2.10	2.10	1.5
	Seismic (short-term)		1.19	1.20	1.19	1
Note: 1. FOS = Factor of safety						

[9] California Geological Survey (2013), Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings, October 2013.

6. DISCUSSIONS AND CONCLUSIONS

Based on the results of our investigation, it is our opinion that the proposed improvements are feasible geotechnically. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated into the design and construction of this project. The primary geotechnical concerns for the Site are:

1. The potential for strong ground shaking to affect the Site during a future significant seismic event (typical of the entire San Francisco Bay Area). Ground shaking can be addressed by incorporating the seismic design parameters presented herein and other seismically related aspects of the 2022 California Building Code (CBC) into the design of the project.
2. The presence of shallow groundwater and associated hydrostatic and buoyancy pressures.
3. The presence of highly compressible Bay Mud and high organic content soils containing peat underneath the Site and associated potential for significant long-term settlement.
4. The presence of highly expansive surficial soils, which can be addressed by providing deeper embedment depth of shallow foundations and proper moisture conditioning of subgrade soils.

Additional discussions of the conclusions drawn from our investigation, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in the “Recommendations” section of this report.

6.1 Shallow Groundwater

As discussed in the “Subsurface Conditions” of this report, free groundwater was observed at depths ranging from about 7 to 25 feet BGS within the current and previous exploration points performed at the Site. However, the actual depth at which groundwater may be encountered in trenches and excavations may vary. Therefore, excavations deeper than about 5 feet BGS will likely require dewatering during construction. In addition, the design of the proposed below-grade improvements, such as new piping for the oxidation pond transfer structures, will need to consider buoyancy forces. We recommend using a design groundwater depth of 5 feet BGS for the design of buoyancy forces. **As previously discussed, any reference made to “below the existing ground surface (BGS)” throughout this report refers to the ground surface at the crest of the existing levees for the oxidation ponds.**

We assume pertinent oxidation ponds will be drained during repair operations for the transfer structures. Due to the predominantly clayey nature of the levee fill and the underlying Bay Mud layer, we expect that dewatering can be conducted primarily via use of sumps and open pumping. However, the contractor should be responsible for the means and methods for dewatering the Site provided the design/construction management team is afforded an opportunity to review and comment on the proposed dewatering method(s) to be used.

6.2 Existing Levee Fill

Based on the findings from the current and previous investigations, the existing levee fill appears to consist of properly engineered fill. The fill generally has consistent and adequate strength based on laboratory strength testing and pocket penetrometer readings, the fill has consistent and adequate dry density and moisture content based on test results, and the fill has consistent and high blow counts. The existing fill also appears to be free of debris, deleterious matter, and organics based on the current and previous borings. Therefore, other than having to scarify the crest of the levees during placement of fill as discussed later in this report, there is no need to overexcavate and replace or recompact the existing levee fill.

As noted in the “Site and Project Description” section of this report, in December of 2021, a sinkhole was discovered on the levee roadway between the flow transfer structure on the aerated lagoon and Pond No. 4. The sinkhole appears to have been the result of corrosion of a corrugated metal pipe. We understand that another sinkhole has been identified more recently, which is located on the inboard side of the levee for Pond No. 9 just south of the WEPS facility. Consideration should be given to performing a video survey of existing underground utilities throughout the Site to check the integrity of existing pipelines. Consideration should also be given to performing a geophysical survey of the levees to check for potential voids withing the levees that could lead to future sinkholes.

6.3 Impact of Bay Mud on the Site’s Development

Based on our interpretation of the current and previous subsurface data presented in Appendices A, B, and D, most of the Site is underlain by Bay Mud as shown on Figure 2. As previously mentioned, Bay Mud is susceptible to high long-term consolidation settlement upon loading. The Bay Mud thickness ranges from about 6 to 10 feet within the vicinity of Ponds No. 9 and 10, to about 4 to 7 feet within the vicinity of Ponds 5 and 6, and to about 1 to 4 feet within the vicinity of Ponds No. 2, 3, and 4. Based on our findings, the upper half of the Bay Mud layer consists of a higher strength “crust” that is less susceptible to higher consolidation settlement than the lower half.

6.3.1 Long-Term Consolidation Settlement

Once new fill is placed to raise the levees and the chemical processing area, it will trigger long-term consolidation settlement of the Bay Mud layer underlying the Site. To help us evaluate potential consolidation settlement if 2 to 3 feet of fill is placed over the existing oxidation pond levees, we ran consolidation testing on a sample collected at a depth of approximately 16 feet BGS at boring B-3. We also ran consolidation settlement analyses using the program Settle3D (Version 2.016) for a generic levee cross section and for the chemical processing area, which is significantly wider than the rest of the Site levees. The results of our analyses as well as the assumed geometry and geotechnical parameters used in the analyses are provided in the sections below.

6.3.1.1 Generic Levee Cross Section

We evaluated the long-term consolidation settlement for a generic levee cross section using the following parameters/assumptions in our analysis:

- 2 to 3 feet of new fill will be added to the crest of the levee.
- The levee is 20 feet wide at the crest.
- The levee has side slopes with gradients of 3H:1V.
- The existing levee fill is 10 feet thick.
- The existing levee fill has been in place for 50 years (i.e., since circa 1972).
- The Bay Mud layer underlying the underlying the existing levee fill is 10 feet thick.

Based on our analyses, and the stated assumptions, we estimate total long-term consolidation settlement of **about 3 to 6 inches.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill. This settlement will occur areawide and should have higher magnitude where the Bay Mud layer is thicker and lower magnitude where the Bay Mud layer is thinner.

6.3.1.2 Chemical Processing Area

We evaluated the long-term consolidation settlement for the chemical processing area (CPA) using the following parameters/assumptions in our analysis:

- 2 to 3 feet of new fill will be added to the crest of the CPA levee.
- The CPA levee is 190 feet long by 140 feet wide at the crest.
- The CPA levee has side slopes with gradients of 5H:1V.
- The CPA existing levee fill is 7 feet thick.
- The CPA existing levee fill has been in place for at least 30 years.
- The Bay Mud layer underlying the underlying the CPA existing levee fill is 10 feet thick.

Based on our analyses, and the stated assumptions, we estimate total long-term consolidation settlement of **about 3 to 6 inches.** We estimate that 90 to 95% of the settlement will occur within the first 1 to 2 years after placing the new fill.

6.3.1.3 Differential Settlement

Existing linear improvements, such as pavements, concrete flatwork, and underground utilities should not be subject to abrupt differential settlement as a result of placement of the new fill because the settlement should occur uniformly areawide. However, upwards of 3 inches of abrupt differential settlement could occur where these improvements are located adjacent to or are connected to existing structures that are supported on deep foundations that extend below the Bay Mud layer. Therefore, site grades may need to be re-adjusted near such structures in the future to eliminate trip hazards that develop as a result of this differential settlement. Also, underground and above ground utilities may eventually be damaged where they connect to such structures. This could be mitigated by installing flexible joints at these connections or by repairing the damage after it occurs.

6.3.2 Construction Considerations

The contractor should exercise extreme care during construction to not disturb the Bay Mud Crust layer present immediately below the existing levee fill during repair operations for the oxidation pond transfer structures to avoid the potential for causing a bearing capacity failure of the Bay Mud Crust. Otherwise, this could lead to a phenomenon typically referred to as a Bay Mud "wave", where adjacent sections of the Bay Mud layer are pushed up and down, severely impacting existing improvements situated atop the Bay Mud layer. Therefore, **earthwork equipment, soil stockpiles, or construction supplies should not be placed directly over the surface of the Bay Mud Crust layer** either within the oxidation ponds or in sections of the levee that are excavated during construction to repair the transfer structures. **Excavators with long reach arms should be used during excavation, removal of existing piping, placement of new piping, and backfill operations. Such excavators should work from the top of the existing levees only**. If this is not possible, BSK should be consulted to provide additional input/recommendations prior to placing additional loading over the Bay Mud Crust layer.

Any Bay Mud excavated as part of the planned rehabilitation of the existing transfer structures or from the oxidation ponds in the future should not be re-used as engineered fill or backfill at the Site.

6.4 Foundations and Lateral Earth Pressures

Based on our recent communication with Dudek, none of the reinforced concrete basin structures for the existing transfer structures are expected to be replaced. Nevertheless, we have included recommendations for spread footings, mat foundations, and below-grade walls in this report in case one or more of the reinforced concrete basins for the existing transfer structures end up having to be replaced.

6.5 Anticipated Settlements

The subsections below present our estimated elastic, consolidation, liquefaction-induced, and dynamic compaction/seismic settlements for the planned improvements for this project. These estimated settlements should be re-evaluated on a project-specific basis for future projects at the Site. For design purposes, these settlements should be assumed to be cumulative.

6.5.1 Elastic Settlement

Total and differential elastic settlements for shallow foundations (i.e., spread footings and mats) are estimated to be less than ½-inch and ¼-inch. Differential settlement is defined herein as the vertical difference in settlement between adjacent fountain supports or across a horizontal distance of approximately 30 feet, whichever is less. Most of the elastic settlement is expected to occur during construction as the loads are applied. These estimates assume the recommendations presented in this report are properly implemented.

6.5.2 Consolidation Settlement

The Site's consolidation settlement is discussed in the preceding "Long-Term Consolidation Settlement" section of this report.

6.5.3 Liquefaction-Induced Settlement

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits and some low plasticity clays. If liquefaction occurs, foundations resting above or within the liquefiable layer may undergo settlements and/or a loss of bearing capacity.

We ran liquefaction analysis for our current CPTs (CPT-1 through CPT-5) using the methods by Boulanger and Idriss (2014)[10] using the program software Cliq. For our analyses, we assumed a design groundwater depth of 5 feet BGS and a peak ground acceleration of 0.68g and earthquake magnitude of M7.22 per the site-specific ground motion hazard analysis presented in Appendix F of this report. The results of our liquefaction hazard analysis are presented in Appendix B and are summarized in the table below. Based on these results, we conclude that the potential for liquefaction analysis to occur at the Site to be low to moderate.

SUMMARY OF LIQUEFACTION-INDUCED SETTLEMENTS

CPT	Estimated Total Liquefaction-Induced Settlement (inches)	Estimated Differential[1] Liquefaction-Induced Settlement (inches)
CPT-1	1	½
CPT-2	¼	Less than ¼
CPT-3	Less than ¼	Less than ¼
CPT-4	½	Less than ¼
CPT-5	0	0

Note:
1. Differential settlement is defined herein as the vertical difference in settlement between adjacent foundation supports or across a horizontal distance of approximately 30 feet, whichever is less.

[10] Boulanger, R. W., and Idriss, I. M. (2014), CPT and SPT Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, California Report No. UCD/CGM-14/01, April 2014.

Based on Youd and Garris (1995)[11] and the depth and thickness of the potentially liquefiable layers shown in Appendix B, we consider the overall potential for ground surface disruption (such as sand boils, ground fissures, etc.) to occur at the Site to be low due to relative thickness of the non-liquefiable layers overlying the liquefiable layers.

6.5.4 Dynamic Compaction/Seismic Settlement

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Due to the composition, consistency, and apparent relative density of the soils above the design groundwater level within the current and previous exploration points, we conclude that the potential for dynamic compaction/seismic settlement to affect the Site during a seismic event is low.

6.6 Geologic and Seismic Hazards

6.6.1 Faulting and Seismic Shaking

The Site is not located within an Alquist-Priolo Earthquake Fault Zone and no mapped active fault traces are known to transverse the Site. Therefore, we conclude that the potential for surface fault rupture to occur across the Site is low. Nevertheless, the Site is in a seismically active area of California. We expect the Site to be subjected to moderate to intense ground shaking due to a significant seismic event on the nearby active faults in the Bay Area and surrounding regions during the design life of the project. The nearby active faults include the Rogers Creek, approximately 3 miles northeast, the West Napa, approximately 13½ miles northeast, and the San Andreas, approximately 17 miles southwest of the Site.

In 2015, scientists and engineers released a new earthquake forecast for the State of California[12]. It updates the earthquake forecast made for the greater San Francisco Bay Area by the 2007 Working Group for California Earthquake Probabilities. According to this recent study, there is a 72 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area in the next 30 years (2014 to 2044).

As has been demonstrated recently by the 1989 (M6.9) Loma Prieta, the 1994 (M6.7) Northridge, and the 2014 (M6.0) Napa County earthquakes, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban environments. Therefore, the design of new structures

[11] Youd, T. L. and Garris, C. T. (1995), Liquefaction-Induced Ground-Surface Disruption, Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 11, November, pp. 805-809.

[12] Field, E.H., and 2014 Working Group on California Earthquake Probabilities (2015), UCERF3: A new earthquake forecast for California's complex fault system: U.S. Geological Survey 2015–3009, 6 p., https://dx.doi.org/10.3133/fs20153009.

should incorporate the seismic design parameters presented in the "Seismic Design Criteria" section of this report.

6.6.2 Slope Stability

Based on our limited slope stability analysis (refer to the "Limited Slope Stability Analysis" section of this report), we expect the existing levees to be globally stable under static and seismic conditions if 2 to 3 feet of additional fill is placed over the levees to increase overall storage capacity for the oxidation ponds. However, it is still possible that some sections of the levees could fail globally during a future significant seismic event at locations where higher concentrations of peat are present or where the Bay Mud Crust layer is thinner (or nonexistent) than assumed in our analysis. Rather than spending significant sums to try and mitigate this potential (which may or may not happen during the design life of the facility), we believe that a more feasible approach would be to repair sections of the levees that fail globally during a significant seismic event.

The above conclusions assume that existing levee slope gradients will be maintained when raising the levees. If steepening of the levee slope gradients is desired, BSK should be consulted to evaluate the potential impact on the global stability of the levees.

6.6.3 Expansive Soils

According to the current and previous Atterberg limits testing, the surficial soils at the Site have a high shrink and swell potential (i.e., high expansive potential) when exposed to moisture fluctuation. Mitigation of expansive soil behavior is recommended by deepening shallow foundations and moisture conditioning of the subgrade soils as discussed in the "Spread Footings and Mat Foundations" and "Earthwork" sections, respectively, of this report.

6.6.4 Liquefaction Potential

The Site's liquefaction potential is discussed in the preceding "Liquefaction-Induced Settlement" section of this report.

6.6.5 Dynamic Compaction/Seismic Settlement Potential

The Site's dynamic compaction settlement is discussed in the preceding "Dynamic Compaction/ Seismic Settlement" section of this report.

6.6.6 Lateral Spread Potential

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to temporary lateral migration of subsurface liquefied soils during a design seismic event. These phenomena typically occur adjacent to free faces such as slopes and creek channels. Based on our liquefaction analysis results for the current CPTs and the subsurface conditions encountered in the current and previous borings, we conclude that the potential for lateral spreading to occur at the Site is low.

Although our liquefaction analyses estimated up to about 1-inch of liquefaction-induced settlement at CPT-1 (2023), more than 80 percent of the settlement is estimated to occur at or below an elevation of 4 feet (approximately 18 feet BGS), which appears to be well below the bottom of the nearby Pond No. 4 and Aerated Lagoon (which has elevations ranging from 8 to 9 feet according to the current elevation topographic map) and the Ellis Creek channel (which has an elevation of about 13 feet according to Google Earth Pro). Also, borings B-1, EB-24, and HB-2 (which were advanced proximate to CPT-1) encountered clayey soil layers extending to elevations of about 6 (the maximum depth of boring B-1), -8, and -17 feet, respectively. These elevations and conditions are consistent with our conclusion that the potential for lateral spreading to occur at the Site is low.

6.6.7 Flood Hazard

According to the 2015 Federal Emergency Management Agency (FEMA) flood insurance rate maps[13], the Site is located in within Zone AE – Special Flood Hazard Area with a Base Flood Elevation (BFE) determined. The BFE for the area is 10 feet (see Exhibit 2 below). According to the current elevation topographic map of the Site, the elevation at the top of the oxidation pond levees ranges from about 13 to 22 feet.

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[13] Federal Emergency Management Agency (FEMA 2015), FEMA Flood Insurance Rate Map, Sonoma County, California and Incorporated Areas, Map Number 06097C1002G, October 2, 2015.

Exhibit 2 – FEMA Flood Map

6.6.8 Tsunami Hazard

According to the CGS (2022[14]) Tsunami hazard area map, the Site is just outside the tsunami hazard area (see Exhibit 3 below).

[14] Patton, J.R. and Wilson, R.I. (2022), Tsunami Hazard Area Map, Sonoma County; produced by the California Geological Survey and the California Governor's Office of Emergency Services, dated 2022, displayed at multiple scales.

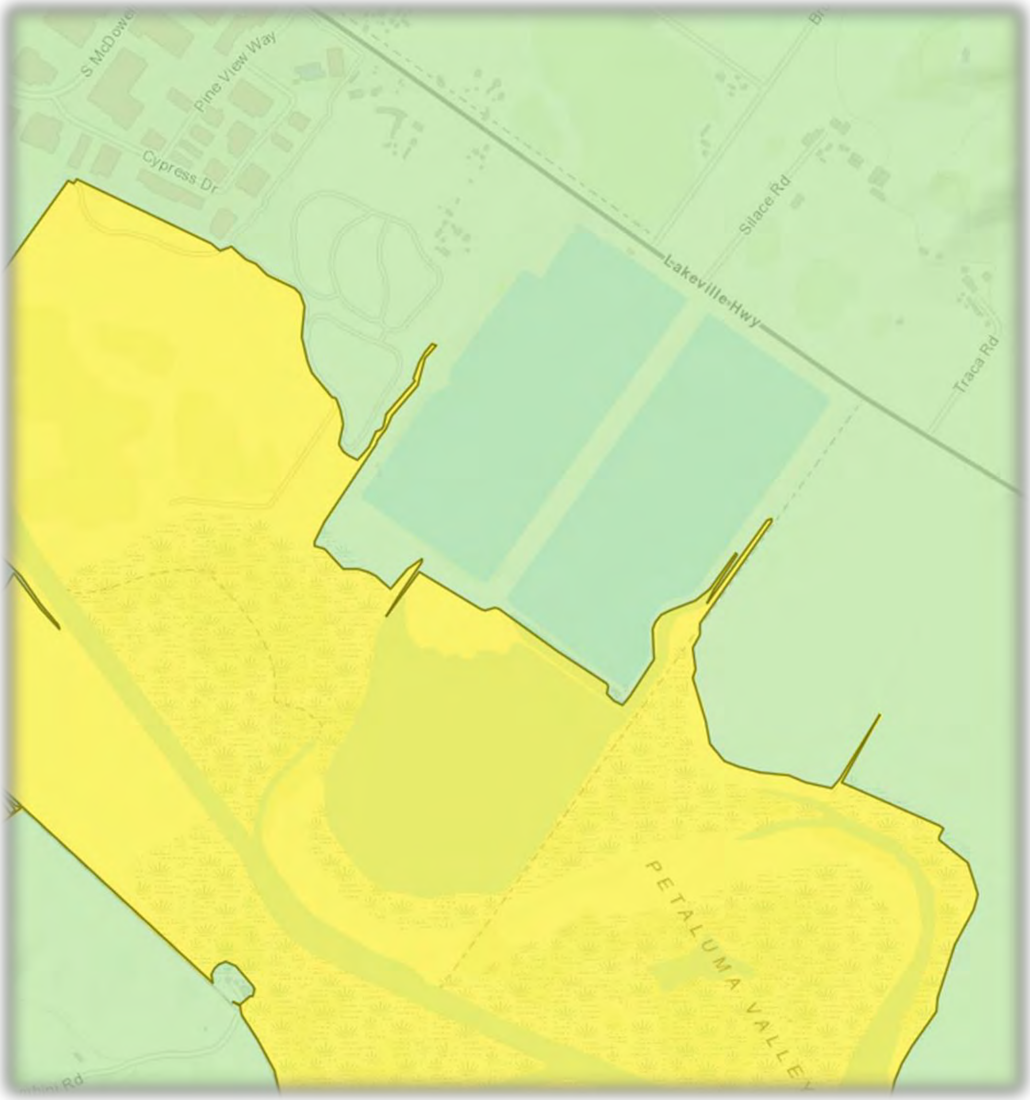


Exhibit 3 – Tsunami inundation map (yellow = tsunami hazard area)

7. RECOMMENDATIONS

Presented below are recommendations for foundations, lateral earth pressures and passive resistance, uplift loading due to buoyancy, seismic considerations, vertical loads on pipes, earthwork, site drainage, and construction considerations for this project.

7.1 Foundation Recommendations (Transfer Structures Only)

7.1.1 Spread Footings and Mat Foundations

We recommend the criteria presented in the tables below be incorporated into the design of new reinforced concrete basins for the existing transfer structures (if applicable). The low allowable bearing capacities provided below take into account the presence of Bay Mud underlying the Site.

SPREAD FOOTING DESIGN CRITERIA (ONLY APPLIES TO NEW TRANSFER STRUCTURES)	
Static Allowable Bearing Capacity[1]	1,000 psf
Seismic/Wind Allowable Bearing Capacity[1]	1,500 psf
Passive Resistance (Equivalent Fluid Pressure)[2,3]	300 pcf
Allowable Lateral Sliding Resistance Adhesion[3]	600 psf
Minimum Embedment Depth[4]	24 inches
Minimum Width	12 inches (continuous) 18 inches (isolated)

Notes:
1. Includes a factor of safety of at least 3 for static loading and at least 2 for transient loading (i.e., seismic or wind conditions).
2. Neglect upper 1 foot if surface is not confined by concrete slab or pavement. **For foundations located on or proximate to sloping ground, such as a levee, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 7 feet between the slope face and the nearest edge of the foundation.**
3. The sliding resistance and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading. Values include a factor of safety of at least 1½. The sliding resistance adhesion should be multiplied by the foundation area to obtain horizontal sliding resistance.
4. Below lowest adjacent grade.

MAT SLAB FOUNDATION CRITERIA[1] (ONLY APPLIES TO NEW TRANSFER STRUCTURES)	
Static Allowable Bearing Capacity[2]	500 psf
Seismic/Wind Allowable Bearing Capacity[2]	750 psf
Passive Resistance (Equivalent Fluid Pressure)[3, 4]	300 pcf
Allowable Friction Coefficient[4]	0.30
Modulus of Vertical Subgrade Reaction[5]	30 psi/in
Minimum Slab Thickness[6] at the Edges	12 inches

Notes:

1. Mat slab foundations should be supported on a minimum of 3 inches of CLSM (refer to the "Site Preparation and Grading" section of this report for CLSM recommendations).
2. Includes a factor of safety of at least 3 for static loading and at least 2 for transient loading (i.e., seismic or wind conditions).
3. Neglect upper 1 foot if surface is not confined by concrete slab or pavement. **For foundations located on or proximate to sloping ground, such as a levee, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 7 feet between the slope face and the nearest edge of the foundation.**
4. The sliding resistance and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading. Values include a factor of safety of at least 1½. The sliding resistance adhesion should be multiplied by the foundation area to obtain horizontal sliding resistance.
5. Based on a one square foot bearing plate. This unadjusted value needs to be adjusted for the actual size of the mat as follows:
    a. Multiply by [(m+0.5)/(1.5 x m)] where m is the ratio of the mat length divided by its width (unitless).
    b. If a computer program is used to design the mat for this project and it requires the input of a modulus of subgrade reaction for the Site, the designer should check whether the program requires input of the unadjusted or adjusted modulus of vertical subgrade reaction.
6. Below lowest adjacent finished grade. The thickened edge should be a minimum of 12 inches wide. The slab designer should determine the slab concrete thickness and reinforcing.

7.1.2 Additional Considerations for Shallow Foundations

Where foundations are located adjacent to below-grade structures (including existing footings) or near major underground utilities, the foundation should extend below a 1H:1V (horizontal to vertical) plane projected upward from the structure foundation or bottom of the underground utility to avoid surcharging the below grade structure and underground utility with foundation loads. Where this is not possible or feasible, we recommend that CLSM be used to backfill the portion of the utility trench that extends below the 1H:1V projection. Also, if a utility crosses perpendicular to a footing, if it is located within 2 x W of the bottom of the footing, where W = width of footing, the utility should be encased in CLSM or lean concrete. If a perpendicular utility is located below a depth of 2 x W below the footing, the utility does not need to be encased in CLSM or lean concrete.

Concrete for foundations should be placed neat against firm native soil or engineered fill. **It is critical that foundation excavations not be allowed to dry before placing concrete.** If shrinkage cracks appear in the foundation excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The foundation excavations should be monitored by a representative of BSK for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials.

7.1.3 Construction Monitoring

A BSK representative should observe the foundation excavations to confirm that subsurface conditions are similar to those encountered in the current and previous exploration points and to check if the contractor is properly dewatering the excavation for new transfer structures.

7.2 Uplift Loading Due to Buoyancy

New reinforced concrete basins and piping for the existing transfer structures should be designed to resist a buoyancy force based on a recommended design groundwater depth of 5 feet BGS (referenced to the crest of the existing oxidation pond levees). The weight of the reinforced concrete basins and piping (assume empty case) may be used to resist this uplift pressure as well as friction between the reinforced concrete basin walls and the surrounding backfill and the backfill above the piping. An allowable friction coefficient of 0.25 between the walls and surrounding backfill may be used. This value includes a factor of safety of about 1½. Normal pressures of 60D psf and 30D psf above and below the design groundwater depth, respectively, where D is the depth in feet of the reinforced concrete basin below-grade walls below the ground surface, may be used to compute the normal force to be used with the allowable friction coefficient.

If the mat foundation for the reinforced concrete basins extends beyond the outer reinforced concrete basin wall limits to form a “lip”, the weight of the backfill above the lip plus a soil wedge extending upward at a 65-degree angle from the horizontal from the edge of the lip may be used to resist uplift pressure in lieu of the wall friction discussed in the paragraph above. Effective soil unit weights of 120 and 58 pcf may be used above and below the design groundwater depth, respectively.

If additional resistance to buoyancy is required, this could be provided via use of thicker walls and a greater weight for the reinforced concrete basins, deadman anchors, or placing CLSM/lean concrete backfill above the lip of the mat foundation extending beyond the walls. Deadman anchors for new piping could consist of concrete slabs or ballast strapped to the piping.

7.3 Below-Grade Walls (Transfer Structures only)

Walls for new reinforced concrete basins for the existing transfer structures should be designed to resist the lateral earth pressures exerted by the retained soil or compacted backfill plus additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. An active earth pressure should be used where the walls are allowed to deflect and an at-rest pressure should be used for restrained walls. Fifty percent of a rectangularly distributed uniform surcharge placed at the top of a restrained wall may be assumed to act as a uniform horizontal pressure over the entire height of the wall. Thirty percent of a rectangularly distributed uniform surcharge placed at the top of an unrestrained wall may be assumed to act as a uniform horizontal pressure over the entire height of the wall. The surcharge pressure should be applied over the entire height of the wall. The active earth pressure condition will develop only when the wall is allowed to yield sufficiently. The amount of outward displacement at the top of the wall designed for active earth pressures may be up to 0.004H to 0.04H, where H is the height of the wall. Below-grade walls may be designed using the lateral earth pressures provided in the table below. A rectangularly distributed uniform surcharge pressure of 100 psf is typically applied over the upper 10 feet of below-grade walls to account for surcharge loading imposed by vehicular traffic, such as an HS-20 live load.

LATERAL EARTH PRESSURES		
Earth Pressures	Equivalent Fluid Pressures (pcf)[A]	
	Above Water[B]	Below Water[B]
Active (Flexible walls)	45	85[C]
At-Rest (Rigid walls)	60	90[C]
Seismic (Flexible walls)	27[D,E]	13[D,E]
Seismic (Rigid walls)	47[E,E]	23[D,E]

Notes:

A. The lateral earth pressures presented herein are applicable for level backfill up to 6H:1V.
B. Design groundwater is at a depth of 5 feet BGS (referenced to the crest of the existing oxidation pond levees).
C. Includes hydrostatic pressure.
D. Only applicable for walls retaining more than 6 feet of soil/backfill.
E. Section 1803.5.12 of the 2022 CBC requires that the design for foundation walls include seismic earth pressures and retaining walls supporting backfill heights greater than 6 feet include seismic earth pressures. These pressures are expressed as equivalent fluid pressures and should be added to the wall design in addition to the static active or at-rest pressures. The seismic earth pressure should be applied as a triangular distribution with the resultant force acting 1/3 times the wall height above the base of the wall. The seismic earth pressures presented herein are based on Agusti and Sitar (2013)[15] and the PGA value of 0.68g per Appendix F of this report.

7.4 Seismic Design Criteria

The Site is in located in a region of high seismic activity and will likely be subjected to moderate to intense ground shaking during the life of the project. As a result, structures to be constructed on the Site should be designed in accordance with applicable seismic provisions of the 2022 California Building Code (CBC).

[15] Agusti, G.C. and Sitar, N. (2013), Seismic Earth Pressures on Retaining Structures in Cohesive Soils, report submitted to the California Department of Transportation (Caltrans) under Contract No. 65A0367 and NSF-NEES-CR Grant No. CMI-0936376: Seismic Earth Pressures on Retaining Structures, Report No. UCB GT 13-02, August 2013.

7.4.1 Mapped 2022 CBC Seismic Design Parameters

Based on Section 1613.2.2 of the 2022 CBC, the Site shall be classified as Site Class A, B, C, D, E or F based on the Site soil properties and in accordance with Chapter 20 of ASCE 7-16. Based on the current and previous subsurface data for the Site, we recommend the Site be classified as a Site Class D. **A site-specific ground motion hazards analysis for this project is presented in Appendix F of this report and is discussed in the next section of this report.** However, as an option (if desired by the structural engineer), we have provided mapped 2022 CBC seismic design parameters in the table below, including increased values for S_{M1} and S_{D1} per the exception for Site Class D sites provided in ASCE 7-16, Supplement 3, Section 11.4.8, Item 1.

2022 CBC SEISMIC DESIGN PARAMETERS
(Lat: 38.222148, Lon: -121.568094)

Seismic Design Parameter	Value		Reference
Site Class	D		ASCE 7-16, Table 20.3-1
MCE_R Mapped Spectral Acceleration (g)	S_S = 1.847	S_1 = 0.704	USGS Mapped Values
Site Coefficients (Site Class D)	F_a = 1.0	F_v = 1.7[A]	ASCE 7-16, Table 11.4-1 & -2 **(Supplement 3)**
MCE_R Mapped Spectral Acceleration Adjusted for Site Class Effects (g)	S_{MS} = 1.847	S_{M1} = 1.795 **(See Note B below)**	ASCE 7-16, Eq. 11.4-1 & -2 **(Supplement 3)**
Design Spectral Acceleration (g)	S_{DS} = 1.231	S_{D1} = 1.197 **(See Note B below)**	ASCE 7-16, Eq. 11.4-3 & -4 **(Supplement 3)**
Site Short Period – T_S (Seconds)	T_S = 0.972		$T_S = S_{D1}/S_{DS}$
Site Long Period T_L (Seconds)	8		USGS Mapped Value
Seismic Design Category (SDC)	D		ASCE 7-16, Section 11.6
MCE_G peak ground acceleration adjusted for Site Class effects (g)	PGA_M = 0.854		ASCE 7-16, Section 11.8.3

Definitions:
MCE_R = Risk-Targeted Maximum Considered Earthquake
MCE_G = Maximum Considered Earthquake Geometric Mean

Notes:

A. See requirements for site-specific ground motions in ASCE 7-16, Section 11.4.8. This value of F_v shall be used only for calculation of T_S, determination of Seismic Design Category, linear interpolation for intermediate values of S_1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of S_{D1}.

B. S_{M1} and S_{D1} values with a 50% increase assuming the exception for Site Class D described in ASCE 7-16 Supplement 3, Section 11.4.8, Item 1 is taken. Otherwise, a site-specific ground motion analysis per ASCE 7-16 Section 21.2 is required.

7.4.2 Site-Specific Ground Hazard Analysis and 2022 CBC Seismic Design Parameters

A site-specific ground motion hazard analysis based on Section 21.2 of ASCE 7-16 for the Site is presented in Appendix F of this report. 2022 CBC seismic design parameters based on the site-specific ground motion hazard analysis are also presented in Appendix F.

7.5 Vertical Loads on Pipe

The pipe selected for the transfer structure should be capable of supporting vertical loads due to the soil overburden (trench backfill) and surcharge, including traffic loads. An in-place density of 130 pounds per cubic foot may be assumed for the trench backfill, and Marston's Formula[16] may be used. The table below presents the vertical pressure on the pipe due to an HS-20 live load as defined in the "American Iron and Steel Institute, Handbook of Steel Drainage and Highway Construction Products".

VERTICAL LOADS ON PIPE	
Height of Cover Over Pipe (Feet)	**Vertical Pressure on Pipe (psf)**
1	1,800
2	800
4	400
6	200
8	100
>8	Neglect live load

Additional surcharge loads on the pipe should be considered in the design if the loads are located above the pipe or within a 1H:1V plane projected upwards from the spring line of the pipe.

7.6 Foundation Support and Backfill for Transfer Structures

Removal of existing pipes, installation of new pipes, and removal/reconstruction of reinforced concrete basins (if applicable) for the transfer structures will occur within existing levees. Therefore, **typical pipe bedding and shading material consisting of granular soils should <u>not</u> be used**. Otherwise, adverse seepage conditions could lead to failure of the levees via internal erosion of the levee embankments, which is commonly referred to as "piping”[17]. After the existing pipes are removed, concrete ballast a minimum of 6 inches thick should be installed immediately below the new pipes. The purpose of the ballast is to provide pipe support and a gap below the new pipes to allow proper backfill under the new pipes. Backfill under and around the new pipes and extending at least 6 inches above the crown of the new pipes should consist of CLSM. The ballast should be installed in a manner that allows the CLSM to

[16] Marston, A, and Anderson, A.P., "The Theory of Loads on Pipes in Ditches and Tests of Cement and Clay Drain Tile and Sewer Pipe." Iowa Eng. Sta., Bull. No. 31 (1913).
[17] A condition where flowing water transports soil particles out of the inner core of an earthen dam/levee creating a hole within the dam/levee embankment.

flow freely to fill all voids under and around the new pipes. The new pipes should be secured to the ballast using straps or other means to avoid having the pipes float when they are being backfilled with CLSM.

Once the CLSM has sufficiently cured to allow soil backfill to be placed above it and mechanically compacted, the soil excavated from the levee fill may be used to backfill the remainder of the pipe excavation provided it is free of deleterious matter, organics, and Bay Mud. If imported fill is needed to backfill the zone above the pipe, it should meet the levee fill criteria provided in the “Site Preparation and Grading” section of this report.

Also, **the excavation bottom for new reinforced concrete basins (if applicable) should <u>not</u> be covered by crushed drain rock or similar material to create a stable base on which to construction the new foundation for such structures**. If the exposed surface at the bottom of the excavation is unstable, a layer of CLSM a minimum of 6 inches thick should be placed over the bottom of the excavation. Backfill around new or existing reinforced concrete basins should consist of the soil excavated from the levee fill provided it is free of deleterious matter, organics, and Bay Mud. If imported fill is needed for backfill, it should meet the levee fill criteria provided in the “Site Preparation and Grading” section of this report.

7.7 Demolition

7.7.1 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines that are 2 inches in diameter or less may be left in place beneath the planned structures provided they are cut off and capped at the structure perimeters. Pipelines larger than 2 inches in diameter within the planned structure footprint should be removed or filled with CLSM meeting the project specifications. Active utilities to be reused should be carefully located and protected during demolition and during construction.

7.7.2 Excavation and Backfill of Existing Foundations and Below-Grade Structures

If applicable, all existing foundations and below-grade structures to be abandoned should be demolished and removed. The resulting excavations should then be properly backfilled with compacted engineered fill per the requirements of the “Earthwork” section of this report. A BSK representative should observe and test the compaction for earthwork activities during construction.

7.7.3 Reuse of On-site Concrete, Asphalt Concrete, and Aggregate Base

Where applicable, existing asphalt concrete (AC) may be pulverized and mixed with the underlying gravel layer (i.e., aggregate base) for re-use in the lower 6 inches of the aggregate base layer for new gravel roadways and paved areas after the levees are raised 2 to 3 feet. The processing should be performed in such a manner that the pulverized AC meets the gradation, R-Value, durability index, and sand equivalent requirements of Section 26 of the 2018 Caltrans Standard Specifications, unless otherwise indicated by

BSK during construction. Also, **the contractor should exercise extreme care not to contaminate the pulverized AC and existing AB with the underlying clayey subgrade soils during removal or this could result in rejection of a portion or all the removed materials for use as aggregate base for new gravel roadways and paved areas.**

7.8 Earthwork

7.8.1 Site Preparation and Grading

Our general site preparation and grading recommendations are as follows:

1. The areas to be graded should be cleared of debris, significant surface vegetation and obstructions including abandoned underground pipes, foundations, and concrete slabs. Stripped surface organics should be disposed off-site.

2. **From a geotechnical standpoint only, the levee fill is generally suitable for re-use as general engineered fill[18] provided it is free of deleterious matter, organics, and Bay Mud and properly processed so that particle sizes are not greater than 3 inches in largest dimension.** At least 90 percent by weight of the general engineered fill/backfill materials should be passing the 1-inch sieve. Nesting (i.e., concentration) of larger particles should be avoided to reduce the potential that this could create voids and allow future settlement in the overlying fill/backfill or "piping" failure of the levee. All fill materials should be subject to evaluation and approval by a BSK representative prior to their use.

   If zones of loose/soft or saturated soils, including in existing fill areas, are encountered during excavation and compaction, deeper excavations may be required to expose firm soils. This should be evaluated in the field by a BSK representative. Where deleterious matter is encountered in excavations, this material should be overexcavated and disposed off-site.

3. Controlled Low Strength Material (CLSM) typically consists of a mixture of cement, fly ash, coarse and fine aggregate, an air entrainment admixture, and water. Where foundations will bear on CLSM, the CLSM should have a 28-day compressive strength of at least 50 pounds per square inch (psi) tested in conformance with ASTM D4832 and sampled in accordance with ASTM D5971. For future excavatability of the CLSM, its 28-day compressive strength should not exceed 1,000 psi. A minimum of one set of cylinders should be cast each day CLSM is placed. One flowability test should be conducted per ASTM D6103 each day CLSM is placed and should be at least 8 inches diameter prior to placement.

[18] "General engineered fill" is defined in this report as suitable **on-site soil** that is used to backfill excavations or raise site grade and is properly moisture conditioned and compacted per the requirements of this report. The requirements for the suitability of on-site soils are provided in the "Site Preparation and Grading" section of this report.

The CLSM mix design should be reviewed by the design team and BSK for approval at least 10 business days prior to its use. CLSM placement should be observed and tested by a qualified representative of BSK.

4. **Imported levee fill** material should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." The levee fill should meet the criteria presented in the California Code of Regulation, Title 23, Section 120, which is summarized in the table below (unless otherwise permitted by BSK). **Highly pervious materials such as pea gravel or clean sands should not be used.**

IMPORT LEVEE FILL CRITERIA	
Plasticity Index	8 or greater
Liquid Limit	Less than 50%
% Passing the 3-inch Sieve	100%
% Passing No. 200 Sieve	20% or greater

5. Following stripping and removal of deleterious materials in areas of the Site to receive fill, the Site should be scarified to a minimum depth of 12 inches, moisture conditioned to at least 2 percent above optimum moisture content, and re-compacted to a minimum of 90 percent relative compaction. **It is important to meet this minimum moisture conditioning due to the expansion potential of the near-surface soils.** Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density determined by ASTM D1557 compaction test procedures. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. Scarification and recompaction should extend laterally a minimum of 5 feet beyond the limits of the planned improvements, where achievable. **Per the "Grading of Levee Slopes" recommendations below, the bottom of keyways for levee slopes should not be scarified.**

6. We expect new fill to settle an amount equivalent to 1 percent of the fill thickness even if it is compacted to a minimum of 90 percent compaction. For instance, if the fill thickness is 8 feet, that would be equivalent to about 1 inch of settlement. Although most of this settlement is expected to occur during construction, a portion of this settlement could occur several months to 1+ year after grading for the project is completed. To address this potential settlement, the required compaction for deeper fills should be increased. Therefore, **where fills/backfills are greater than 7 feet in depth below finish grade, the zone below a depth of 7 feet should be compacted to a minimum of 95 percent relative compaction**. Note that increasing the compaction effort should reduce the amount of fill settlement, but it will not eliminate it.

7. **In areas to be exposed to vehicular traffic**, the upper 12 inches of the soil subgrade immediately below the aggregate base layer should be compacted to a minimum of 92 percent relative compaction at least 2 percent above optimum moisture content. Subgrade preparation should extend a minimum of 5 feet laterally beyond the edge of flatwork, pavers, and pavements, where achievable. The aggregate base layer underneath such flatwork, pavers, and pavement should be compacted to a minimum of 95 percent relative compaction at near optimum moisture content.

In addition to these compaction requirements, areas to be exposed to vehicular traffic should be firm and stable and should be proof rolled with a heavy piece of construction equipment, such as a loaded dump truck or water truck, to check for signs of subgrade instability.

8. Unless otherwise indicated above, all fill and backfill should be placed in thin lifts up to 8-inch maximum uncompacted thickness, properly moisture conditioned to at least 2 percent above optimum moisture content for clayey soils and to near optimum moisture content for granular soils, and compacted to at least 90 percent compaction per ASTM D1557. Aggregate base should be moisture conditioned to near-optimum moisture content.

9. **Grading of Levee Slopes**: Current levee slope gradients should be maintained as part of the rehabilitation of the existing transfer structures and raising of the levees unless BSK is consulted to evaluate the feasibility of steepening slope gradients. As previously discussed, existing levee slopes have gradients of about 3H:1V or flatter. If existing levee slope gradients are maintained while raising the levees by 2 to 3 feet vertically, this would require widening one or both sides of the levees a total width of at least 12 to 18 feet at the base depending on the thickness of new fill placed and the existing slope gradient. Our recommendations for widening the levees are discussed below.

    a. During widening of the levees, the new levee fill should be overbuilt a minimum of 2 feet laterally and then cut back to finished grade to allow proper compaction of the finished slope face. The widened portion of the levees should be supported on 18-inch-deep keyways that are a minimum of 3 to 5 feet wide or as indicated by a BSK representative during construction. A layer of Mirafi RS280i geotextile fabric or equivalent should be placed over the bottom of the keyways unless indicated otherwise by BSK during construction. The geotextile fabric should be overlapped a minimum of 2 feet at the seams. The contractor should exercise extreme care not to excavate the keyways any deeper than recommended herein. Otherwise, the integrity of the Bay Mud Crust layer could be compromised. For this reason, the bottom of the keyways should <u>not</u> be scarified. The backside (back cut) of the new levee fill should be benched into the existing levee fill at regular vertical intervals of about 2 to 3 feet as the new levee fill placement proceeds upslope of the keyway base. The bench width should be a minimum of 2 feet wide.

    b. Consideration should be given to installing rock slope protection (RSP) as part of the outer surface of the new levee fill slope to provide long-term protection against future surface erosion. The RSP layer should be a minimum of 1-foot thick and should consist of Class II rock gradation per Section 72-2.02B of the 2018 Caltrans Standard Specifications. The RSP layer should be underlain by Class 10 RSP fabric meeting the requirements of Section 96-1.02I of the 2018 Caltrans Standard Specifications, such as Mirafi® 1100NC or equivalent overlapped at minimum of 1 foot at the seams and fixed to the surface of the slope using staples per the manufacturer's requirements.

c. At the conclusion of construction operations, portions of the levee slopes that are not protected by RSP should be hydroseeded to help encourage growth of vegetation on the surface to serve as an additional long-term erosion control measure. Consideration should be given to covering these areas with a biodegradable woven coir erosion control blanket to help provide temporary erosion protection until vegetation is re-established over the area. If used, the woven coir erosion control blanket should meet the requirements of Section 21-2.02O(4), Type B of the 2018 Caltrans Standard specifications, such as North American Green BioNet® 125 (C125BN) or equivalent. The woven coir erosion control blanket should be overlapped a minimum of 1 foot at the seams and fixed to the surface of the slope using wooden stakes or staples per the manufacturer's requirements.

10. Observations and compaction testing should be carried out by a BSK representative during grading and backfill operations, especially during widening of the levees, to assist the contractor in obtaining the required degree of compaction and proper moisture content. Where the moisture content or compaction is outside the range required, additional compactive effort and adjustment of moisture content should be made until the specified compaction and moisture conditioning is achieved.
11. BSK should be notified at least 48 hours prior to any grading and backfill operations. The procedure and methods of grading may then be discussed between the contractor and BSK.

7.8.2 Excavation and Backfill

All excavations should conform to current OSHA requirements for work safety. Where trenches or other excavations extend deeper than 5 feet, the excavations may become unstable and should be evaluated by the contractor to monitor stability prior to personnel entering the trenches. Shoring or sloping of any trench wall may be necessary to protect personnel and to provide stability. It is the contractor's responsibility to follow OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be adjusted as necessary in the field to suit the actual conditions encountered, in order to protect personnel and equipment within excavations. Based on the subsurface conditions encountered in the current and previous exploration points, we expect the sidewalls of trenches that extend to depths of up to about 5 feet to remain relatively vertical for a period of several days. Nevertheless, the longer the trenches remain open the higher the potential for the sidewalls to start to slough off or cave.

As discussed in the "Subsurface Conditions" of this report, free groundwater was observed at depths ranging from about 7 to 25 feet BGS within the current and previous exploration points performed at the Site. However, the actual depth at which groundwater may be encountered in trenches and excavations may vary. We assume pertinent oxidation ponds will be drained during repair operations for the transfer structures. Due to the predominantly clayey nature of the levee fill and the underlying Bay Mud layer, we expect that dewatering can be conducted primarily via use of sumps and open pumping. However, the contractor should be responsible for the means and methods for dewatering the Site provided the design/construction management team is afforded an opportunity to review and comment on the proposed dewatering method(s) to be used. **Groundwater should be lowered and maintained at least 2 feet below the bottom of the planned excavations in order to maintain the undisturbed state of the supporting soils and to allow proper compaction of backfill after below-grade structures and utility lines are installed.**

Where new utility trenches extend from the exterior into the interior limits of pavement, CSLM or lean concrete should be used as backfill material for a distance of 2 feet laterally on each side of the pavement edge to reduce the potential for the trench to act as a conduit for exterior surface water. Utility trenches located in landscaped or unimproved areas of the Site should also be capped with a minimum of 12 inches of compacted on-site clayey soils.

7.8.3 Weather/Moisture Considerations

If earthwork operations and construction for this project are scheduled to be performed during the rainy season (usually November to May) or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures, such as quicklime treatment, to the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations.

7.9 Site Drainage

Proper site drainage is important for the long-term performance of the planned improvements. The Site should be graded to provide positive drainage towards ditches, drain inlets, catch basins, bioretention areas, and similar drainage collection facilities, and away from levee slope faces where possible. Water should not be allowed to pond anywhere along the crest of the Site levees.

7.10 Corrosion Potential

Soil samples were collected during our current subsurface investigation from boring B-3 from depths of about 0 to 5 and 15½ feet BGS and from boring B-5 from depths of about 0 to 5 feet BGS. These samples were submitted for corrosion testing. The samples were tested by CERCO Analytical, a State-certified laboratory in Concord, California, for redox potential, pH, resistivity, chloride content, and sulfate content in accordance with ASTM test methods. The test results are presented at the end of Appendix C. Also included is the evaluation by CERCO Analytical of the corrosion test results.

Based upon the resistivity measurements, the samples tested were classified as "corrosive” to “severely corrosive” by CERCO Analytical. The sulfate ion concentrations ranged from 27 to 390 mg/kg (ppm). These results are indicative of an exposure category S1 per Table 19.3.1.1 of ACI 318-19. For an S1 exposure class, Table 19.3.2.1 indicates that the minimum f’c of the concrete is 4,000 psi. CERCO Analytical concludes that the sulfate ion concentrations are sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel. They recommend that concrete that comes into contact with the soil should use sulfate resistant cement such as Type II with a maximum water-to-cement ratio of 0.55. CERCO Analytical also recommends that all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron be properly protected against corrosion depending upon the critical nature of the structure. They also recommend that all buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details.

The above are general discussions. A more detailed investigation may include more or fewer concerns and should be directed by a corrosion expert. BSK does not practice corrosion engineering. Consideration should also be given to soils in contact with concrete that will be imported to the Site during construction, such as topsoil and landscaping materials, which typically contain fertilizers and other chemicals that can be highly corrosive to metals and concrete. Any imported soil or landscaping materials should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

7.11 Plan Review and Construction Observation

We understand that BSK will be retained by the Client to review the geotechnical aspects of the plans and specifications before they go out to bid. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings of our recommendations prior to the start of construction.

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained and/or reviewed during this investigation and the actual soil conditions encountered during construction, we recommend that BSK be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in the current and previous exploration points performed at the Site and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by BSK during construction. BSK should be notified at least two weeks prior to the start of construction and prior to when observation and testing services are needed.

8. ADDITIONAL SERVICES AND LIMITATIONS

8.1 Additional Services

The review of plans and specifications, and field observation and testing during construction by BSK are an integral part of the conclusions and recommendations made in this report. If BSK is not retained for these services, the client will be assuming BSK's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein. The recommended tests, observations, and consultation by BSK during construction include, but are not limited to:

- review of plans and specifications;
- observations of site grading, including stripping and engineered fill construction;
- observation of foundation and below-grade wall excavations;
- observation of levee widening operations, including keyway excavations and levee fill placement; and
- in-place density testing of fills, backfills, and finished subgrades.

8.2 Limitations

The recommendations contained in this report are based on our field observations and current and previous subsurface exploration, limited laboratory tests, review of available geologic maps and publications, and our present knowledge of the proposed construction. It is possible that soil conditions could vary between or beyond the points explored. If soil conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the Site area at the time of our study. No warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by BSK during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference", as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the Site have changed. If this report is used beyond this period, BSK should be contacted to evaluate whether site conditions have changed since the report was issued.

Also, land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, BSK may recommend that additional work be performed and that an updated report be issued.

The scope of services for this subsurface investigation and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this Site.

BSK conducted subsurface exploration and provided recommendations for this project. We understand that BSK will be given the opportunity to perform a formal geotechnical review of the final project plans and specifications. In the event BSK is not retained to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted, we will assume no responsibility for misinterpretation of our recommendations.

We recommend that all earthwork during construction be monitored by a representative from BSK, including site preparation, foundation excavation, placement of engineered fill, levee fill widening operations, and trench/wall backfill. The purpose of these services would be to provide BSK the opportunity to observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

FIGURES

References: 1. https://www.arcgis.com/apps/mapviewer/index.html, 2023

Note: Location is approximate

BSK ASSOCIATES

PROJECT NO.	**G00000075**
DRAWN:	05/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

VICINITY MAP

Oxidation Pond Transfer Structure Rehab. and Oxidation Pond Storage Expansion Ellis Creek Water Recycling Facility (WRF) Petaluma, California

FIGURE 1

References: 1. http://earth.google.com, 2023

Legend

- B-1 — Approximate Boring Location (BSK, 2023)
- CPT-1 — Approximate Cone Penetration Test Locations (BSK, 2023)
- B-1 — Approximate Boring Location (RGH Consultants, 2012)
- EB-3 — Approximate Boring Location (Harza, 2001)
- EB-1 — Approximate Bloring Location (Harza, 2001) Converted into Stand Pipe Piezometers
- HB-1 — Approximate Boring Location (Harding Lawson, 1995)
- EB-24 — Approximate Boring Location (Fugro West, 2002)
- CPT-1 — Approximate CPT Location (Harza, 2001)
- MT71-10 — Approximate Boring Locations (Moore and Taber, 1971)
- Approximate Extent of Bay Mud (inferred from exploration points)

BSK ASSOCIATES

PROJECT NO.	G00000075
DRAWN:	04/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	SitePlan.indd

SITE PLAN

Oxidation Pond Transfer Structure Rehab. and Oxidation Pond Storage Expansion Ellis Creek Water Recycling Facility (WRF) Petaluma, California

FIGURE 2

APPENDIX A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487/2488)

MAJOR DIVISIONS				*GRAPHIC LOG*	*TYPICAL DESCRIPTIONS*
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH <5% FINES	Cu ≥4 and 1≤Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/or 1>Cc>3	GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5 to 12% FINES	Cu ≥4 and 1≤Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/or 1>Cc>3	GP-GM	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH >12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤Cc≤3	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/or 1>Cc>3	SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5 to 12% FINES	Cu ≥6 and 1≤Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/or 1>Cc>3	SP-SM	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH >12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)			ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY,
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

BSK®

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487/2488)

FIGURE

A-1

SOIL DESCRIPTION KEY

MOISTURE CONTENT

DESCRIPTION	ABBR	FIELD TEST
Dry	D	Absence of moisture, dusty, dry to the touch
Moist	M	Damp but no visible water
Wet	W	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

PLASTICITY

DESCRIPTION	ABBR	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm) thread cannot be rolled at any water content.
Low (L)	LP	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	MP	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	HP	It takes considerable time rolling and kneeding to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

STRUCTURE

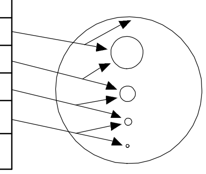
DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4 in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4 in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	ABBR	FIELD TEST
Very Soft	VS	Thumb will penetrate soil more than 1 in. (25 mm)
Soft	S	Thumb will penetrate soil about 1 in. (25 mm)
Firm	F	Thumb will indent soil about 1/4 in. (6 mm)
Hard	H	Thumb wil not indent soil but readily indented with thumbnail
Very Hard	VH	Thumbnail will not indent soil

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12"	>12"	Larger than basketball-sized
Cobbles		3 - 12'	3 - 12"	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3"	3/4 -3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029	Flour-sized and smaller

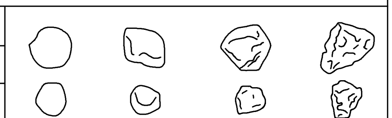


REACTION WITH HCL

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

ANGULARITY

DESCRIPTION	ABBR	CRITERIA
Angular	A	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	SA	Particles are similar to angular description but have rounded edges
Subrounded	SR	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	R	Particles have smoothly curved sides and no edges



APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	ABBR	SPT (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
Very Loose	VL	<4	<4	<5	0 - 15	Easily penetrated with 1/2-inch reinforcing rod by hand
Loose	L	4 - 10	5 - 12	5 - 15	15 - 35	Difficult to penetrate with 1/2-inch reinforcing rod pushed by hand
Medium Dense	MD	10 - 30	12- 35	15 - 40	35 - 65	Easily penetrated a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Dense	D	30 - 50	35 - 60	40 - 70	65 - 85	Difficult to penetrate a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Very Dense	VD	>50	>60	>70	85 - 100	Penetrated only a few inches with 1/2-inch reinforcing rod driven with 5-lb. hammer

LOG SYMBOLS

BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
SPLIT BARREL SAMPLER (2-1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
SPLIT BARREL SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
CONTINUOUS CORE	TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (EM 1110-1-1906)/ASTM Test Method D 2850
SHELBY TUBE	EI	EXPANSION INDEX (UBC STANDARD 18-2)
ROCK CORE	COL	COLLAPSE POTENTIAL
GROUNDWATER LEVEL (encountered at time of drilling)		
GROUNDWATER LEVEL (measured after drilling)	UC	UNCONFINED COMPRESSION (ASTM Test Method D 2166)
SEEPAGE	MC	MOISTURE CONTENT (ASTM Test Method D 2216)

GENERAL NOTES

Boring log data represents a data snapshot.

This data represents subsurface characteristics only to the extent encountered at the location of the boring.

The data inherently cannot accurately predict the entire subsurface conditions to be encountered at the project site relative to construction or other subsurface activities.

Lines between soil layers and/or rock units are approximate and may be gradual transitions.

The information provided should be used only for the purposes intended as described in the accompanying documents.

In general, Unified Soil Classification System designations presented on the logs were evaluated by visual methods.

Where laboratory tests were performed, the designations reflect the laboratory test results.

The Responsible Geotechnical Engineer, Professional Engineer, or Professional Geologist uses professional judgement and visual-manual procedures in general conformance with ASTM D2488 to classify soil when the full classification suite of tests per ASTM D2487 is not conducted.

LOG KEY

FIGURE

A-3

LOG OF BORING NO. B-1

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

Project Name: **Oxidation Ponds**
Project Number: **G00000075**
Project Location: **Ellis Creek WRF**
Logged by: **O. Khan**
Checked by: **M. Romero**

Surface El.: **22 feet**
Location: **Approximately: 38.224750, -122.577197**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0	**Sandy Lean CLAY with Gravel (CL):** dark yellowish brown, moist, fine to medium sand, fine to coarse gravel (FILL)									
2.5	**Sandy Fat CLAY (CH):** dark olive brown, moist, hard, medium to high plasticity, fine sand (FILL)	1A 1B 1C	9 13 16	4.0		105	17			
4.5	**Sandy Lean CLAY (CL):** dark olive gray, moist, firm to hard, medium plasticity, fine to medium sand (FILL) TXUU (see figure C-2) c= 2,225 psf	2A 2B 2C	7 10 14	3.5 4.0 2.0		105	20			
6.5	**Sandy Lean CLAY (CL):** dark greenish gray, moist, firm, medium plasticity, fine to medium sand									
10	decreased sand content, interbedded with sandy silt, medium to high plasticity	3A 3B 3C	4 8 11	2.0 3.0		103	23			
14.5	**Fat CLAY (CH):** light greenish gray to light olive gray, moist, firm, high plasticity, trace fine sand and gravel TXUU (see figure C-2) c= 1,940 psf	4A 4B 4C	6 9 12	2.0 2.5		96	28			
16.5	Boring terminated at approximately 16.5 feet. No free groundwater was observed. Boring was backfilled with cement grout.									

Depth scale: 5, 10, 15, 20

Completion Depth: 16.5
Date Started: 2/16/23
Date Completed: 2/16/23
California Sampler: 2.5-in inner diameter
SPT Sampler: 1.4-in inner diameter

Drilling Equipment: Taber Drilling CME 55
Drilling Method: Hollow Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 8-in
Drop: 30-in
Remarks: Automatic Hammer

BSK ASSOCIATES

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-2

Project Name: **Oxidation Ponds**
Project Number: **G00000075**
Project Location: **Ellis Creek WRF**
Logged by: **O. Khan**
Checked by: **M. Romero**

Surface El.: **21 feet**
Location: **Approximately: 38.223688, -122.572618**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0	**Sandy Lean CLAY (CL):** dark yellowish brown to olive brown, moist, fine to medium sand, trace fine to coarse gravel (FILL)									
5		1A 1B 1C	5 7 9			104	21			
	Clayey SAND (SC): olive brown, moist, loose to medium dense, low plasticity, fine to medium sand (FILL)									
10	**Fat CLAY with Sand (CH):** dark bluish gray, moist, fine sand, soft to firm, high plasticity TXUU (see figure C-2) c= 1,260 psf	2A 2B 2C	5 6 10	1.0 1.5		98	25			
15	olive brown, firm, medium to high plasticity, mottled with calcium carbonate TXUU (see figure C-2) c= 1,680 psf	3A 3B 3C	4 6 9	2.0		100	24			
	Boring terminated at approximately 16.5 feet. No free groundwater was observed. Boring was backfilled with cement grout.									
20										

Completion Depth: 16.5
Date Started: 4/13/23
Date Completed: 4/13/23
California Sampler: 2.5-in inner diameter
SPT Sampler: 1.4-in inner diameter

Drilling Equipment: Taber Drilling CME 55
Drilling Method: Solid Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 6-in
Drop: 30-in
Remarks: Automatic Hammer

LOG OF BORING NO. B-3

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

Project Name: **Oxidation Ponds**
Project Number: **G00000075**
Project Location: **Ellis Creek WRF**
Logged by: **O. Khan**
Checked by: **M. Romero**

Surface El.: **15 feet**
Location: **Approximately: 38.220190, -122.582302**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Clayey GRAVEL with Sand (GC): dark olive gray, dry to moist, fine to medium sand, fine angular gravel (FILL)										
		Sandy Fat CLAY (CL): dark greenish gray, moist, hard, medium plasticity, fine sand, trace fine gravel (FILL)										
		TXUU (see figure C-3) c= 1,350 psf		1A 1B 1C	8 12 12	3.0 3.5		93	25	61	20	41
5		**Clayey SAND with Gravel (SC):** olive brown, moist, very loose, low plasticity, fine to coarse sand, fine gravel (FILL)		2A 2B 2C	3 3 3			92	23			
		Lean CLAY (CL): dark bluish gray, moist to wet, very soft, medium to high plasticity, interbedded with clayey sand to silty sand lenses (Bay Mud)										
10		∇										
		TXUU (see figure C-3) c= 740 psf		3A 3B 3C	1 1 2	1.0		80	40	40	17	23
15		soft, organic odor Consolidation Test (see figure C-5)		4A 4B 4C	2 1 3	0.5		49	89			
		Fat CLAY (CH): dark bluish gray, moist to wet, firm, high plasticity 0 to 250 psi		5		2.0						
20												

Completion Depth: 31.5
Date Started: 2/16/23
Date Completed: 4/13/23
California Sampler: 2.5-in inner diameter
SPT Sampler: 1.4-in inner diameter

Drilling Equipment: Taber Drilling CME 55
Drilling Method: Hollow Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 8-in
Drop: 30-in
Remarks: Automatic Hammer

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-3

Project Name:	**Oxidation Ponds**
Project Number:	**G00000075**
Project Location:	**Ellis Creek WRF**
Logged by:	**O. Khan**
Checked by:	**M. Romero**

Surface El.: **15 feet**
Location: **Approximately: 38.220190, -122.582302**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
	Fat CLAY (CH): dark bluish gray, moist to wet, firm, high plasticity *(continued)*									
	Sandy Lean CLAY (CL): light olive gray, moist, firm to hard, medium plasticity, high sand content, fine to medium sand, manganese oxide staining									
25		6A 6B 6C	7 8 13	2.5 4.5		115	17			
	Clayey SAND (SC): olive, moist to wet, loose, low plasticity, fine to coarse sand									
30		7A	9							
	Lean CLAY with Sand (CL): olive to green, moist, firm, medium plasticity, fine sand, iron and manganese oxide staining	7B 7C	4 6	1.5 3.0						
	Boring terminated at approximately 31.5 feet. Free groundwater was observed at approximately 10 feet. Boring was backfilled with cement grout.									
35										
40										

Completion Depth:	31.5	**Drilling Equipment:**	Taber Drilling CME 55
Date Started:	2/16/23	**Drilling Method:**	Hollow Stem Auger
Date Completed:	4/13/23	**Drive Weight:**	140 lbs
California Sampler:	2.5-in inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-in inner diameter	**Drop:**	30-in
		Remarks:	Automatic Hammer

LOG OF BORING NO. B-4

BSK ASSOCIATES

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

Project Name:	**Oxidation Ponds**
Project Number:	**G00000075**
Project Location:	**Ellis Creek WRF**
Logged by:	**O. Khan**
Checked by:	**M. Romero**

Surface El.: **14 feet**

Location: **Approximately: 38.217479, -122.578757**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Poorly Graded GRAVEL with Sand (GP): yellowish brown, dry, fine to coarse sand, fine to coarse gravel (FILL)										
		Fat CLAY (CH): dark grayish brown, moist, firm to hard, high plasticity, trace organics, trace fine sand (FILL)		1A 1B 1C	5 7 11	3.5 2.5		83	36			
5		**Fat CLAY with Sand (CH):** dark greenish gray, moist, firm to hard, high plasticity, fine to medium sand, interbedded with silty sand lens (possibly FILL)		2A 2B 2C	5 8 12	2.5		101	23			
10		**Fat CLAY (CH):** dark olive brown, moist, soft to firm, high plasticity, manganese oxide staining, iron oxide staining (Bay Mud) TXUU (see figure C-3) c= 765 psf		3A 3B 3C	2 4 4	0.5 1.5		84	38			
15		**Organic CLAY (OH):** dark olive brown, moist, soft, high plasticity, high organic content (Bay Mud) Organic content= 13%		4A 4B 4C	2 2 3	0.0 0.5		38	123			
20		**Fat CLAY (CH):** dark gray to dark greenish gray, moist, firm, high plasticity		5A 5B	4 6	2.0						
		Sandy Lean CLAY (CL): grayish green, moist, hard, medium plasticity, fine to medium sand		5C	11	3.5						

Completion Depth:	31.5	**Drilling Equipment:**	Taber Drilling CME 55
Date Started:	2/16/23	**Drilling Method:**	Hollow Stem Auger
Date Completed:	4/13/23	**Drive Weight:**	140 lbs
California Sampler:	2.5-in inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-in inner diameter	**Drop:**	30-in
		Remarks:	Automatic Hammer

LOG OF BORING NO. B-4

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

Project Name:	**Oxidation Ponds**
Project Number:	**G00000075**
Project Location:	**Ellis Creek WRF**
Logged by:	**O. Khan**
Checked by:	**M. Romero**

Surface El.: **14 feet**

Location: **Approximately: 38.217479, -122.578757**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Sandy Lean CLAY (CL): grayish green, moist, hard, medium plasticity, fine to medium sand *(continued)*										
		∇										
25		light olive brown, increased sand content, trace fine gravel, iron and manganese oxide staining		6A 6B 6C	4 5 6	2.0		106	23			
30		**Poorly Graded SAND (SP):** olive gray, wet, loose, fine to medium sand		7A	6							
		Clayey SAND (SC): pale olive, moist, medium dense, low plasticity, fine sand		7B 7C	9 11							
		Boring terminated at approximately 31.5 feet. Free groundwater was observed at approximately 23 feet. Boring was backfilled with cement grout.										
35												
40												

Completion Depth:	31.5	**Drilling Equipment:**	Taber Drilling CME 55
Date Started:	2/16/23	**Drilling Method:**	Hollow Stem Auger
Date Completed:	4/13/23	**Drive Weight:**	140 lbs
California Sampler:	2.5-in inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-in inner diameter	**Drop:**	30-in
		Remarks:	Automatic Hammer

BSK ASSOCIATES

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-5

Project Name: **Oxidation Ponds**
Project Number: **G00000075**
Project Location: **Ellis Creek WRF**
Logged by: **O. Khan**
Checked by: **M. Romero**

Surface El.: **13 feet**
Location: **Approximately: 38.214985, -122.574695**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		ASPHALT: approximately 4 inches of asphalt										
		FILL: approximately 8 inches of sandy/gravelly soil (possibly aggregate base)										
		Lean CLAY with Sand (CL): dark greenish gray, moist, high plasticity, fine sand, trace gravel (FILL)		1						41	15	26
		dark gray, trace organics										
		increased sand content										
5		firm		2	6 9 11							
10		very dark greenish gray, firm to hard, decreased sand content TXUU (see figure C-3) c= 2,650 psf		3A 3B 3C	5 8 11	3.0 4.5		101	25			
15		**PEAT:** black to dark yellowish brown, soft to firm, low to medium plasticity, high organic content (Bay Mud) Organic content = 37%		4A 4B 4C	2 2 3			24	208			
		Fat CLAY (CH): dark bluish gray, moist, soft to firm, high plasticity, trace fine sand, trace organics (Bay Mud)										
20				5A 5B	4 5	1.0						
		Fat Clay (CH): dark bluish gray, moist, firm, high plasticity TXUU (see figure C-4) c= 1,205 psf		5C	6	2.0		92	31			

Completion Depth: 31.5
Date Started: 4/13/23
Date Completed: 4/13/23
California Sampler: 2.5-in inner diameter
SPT Sampler: 1.4-in inner diameter

Drilling Equipment: Taber Drilling CME 55
Drilling Method: Solid Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 6-in
Drop: 30-in
Remarks: Automatic Hammer

BSK ASSOCIATES

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-5

Project Name: **Oxidation Ponds**
Project Number: **G00000075**
Project Location: **Ellis Creek WRF**
Logged by: **O. Khan**
Checked by: **M. Romero**

Surface El.: **13 feet**
Location: **Approximately: 38.214985, -122.574695**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		Fat Clay (CH): dark bluish gray, moist, firm, high plasticity *(continued)*										
25		▽ **Lean CLAY (CL):** dark olive gray to dark olive green, moist, firm to hard, medium to high plasticity, mottled with calcium carbonate		6A 6B 6C	7 9 12	3.0 4.0		102	24			
30		mottled with manganese oxide staining		7A 7B 7C	8 9 12	4.0						
		Boring terminated at approximately 31.5 feet. Free groundwater was observed at approximately 25 feet. Boring was backfilled with cement grout and patched with rapid set concrete.										
35												
40												

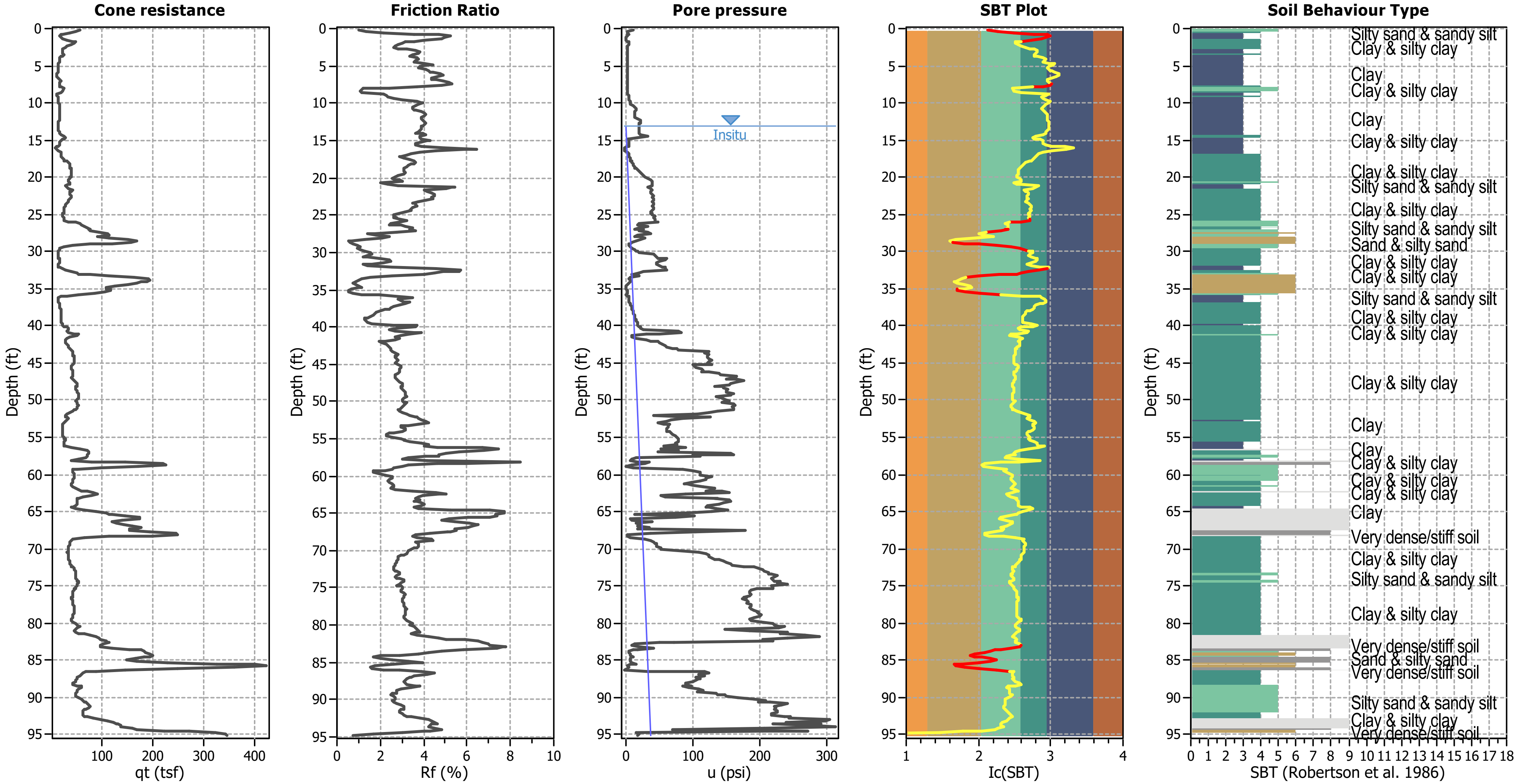
Completion Depth: 31.5
Date Started: 4/13/23
Date Completed: 4/13/23
California Sampler: 2.5-in inner diameter
SPT Sampler: 1.4-in inner diameter

Drilling Equipment: Taber Drilling CME 55
Drilling Method: Solid Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 6-in
Drop: 30-in
Remarks: Automatic Hammer

APPENDIX B

CPT Logs and Liquefaction Analysis

CPT basic interpretation plots



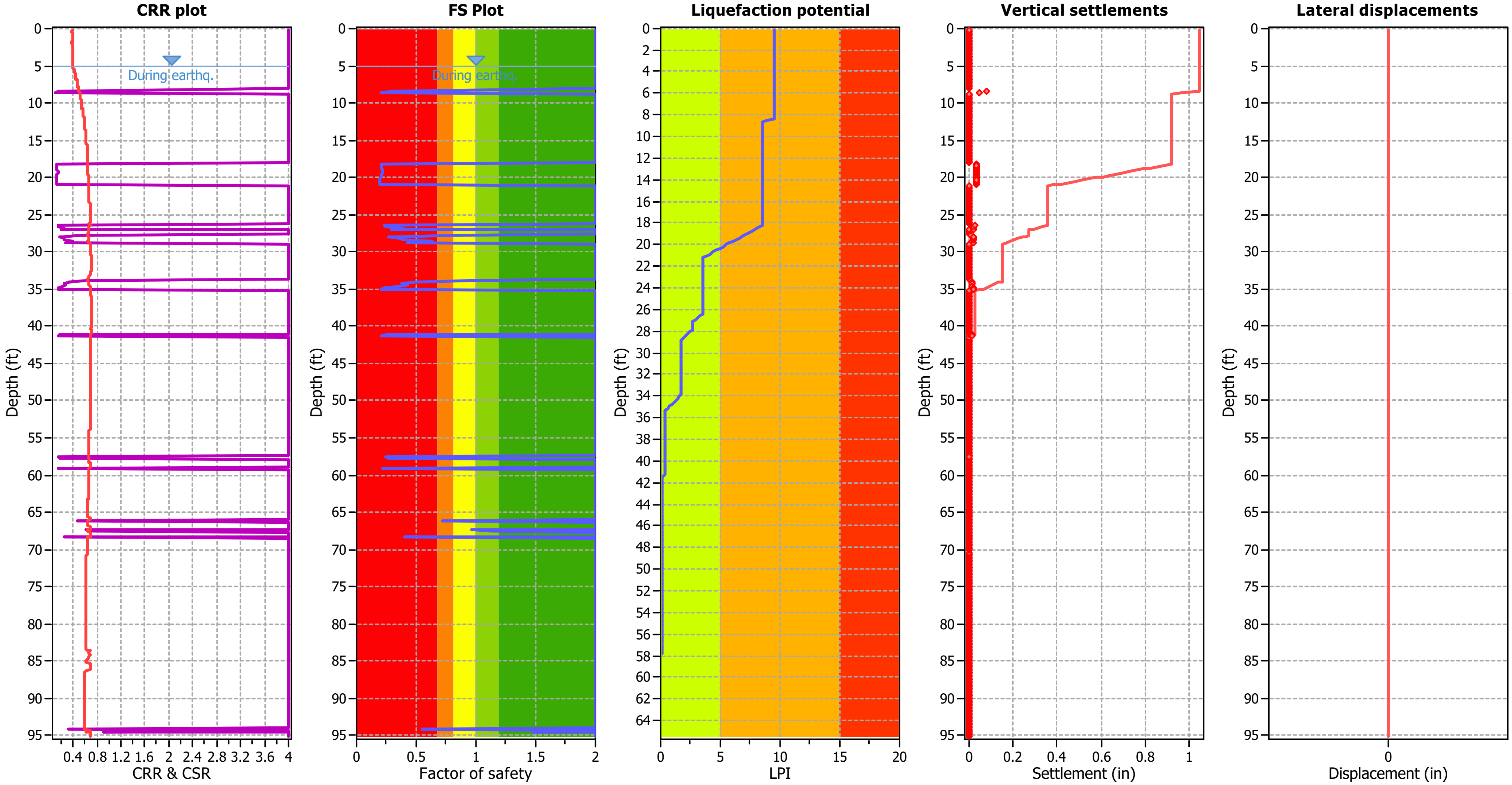
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

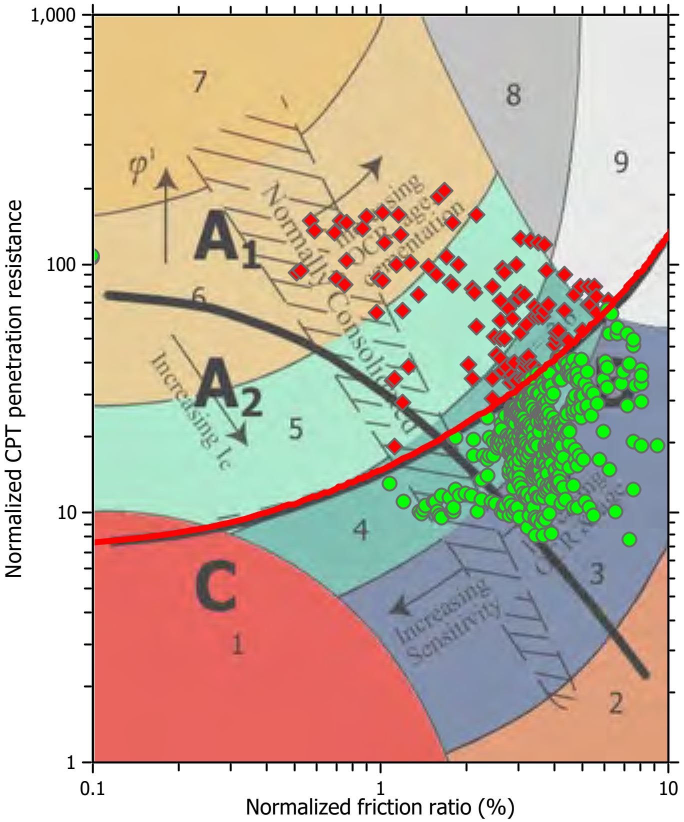
F.S. color scheme

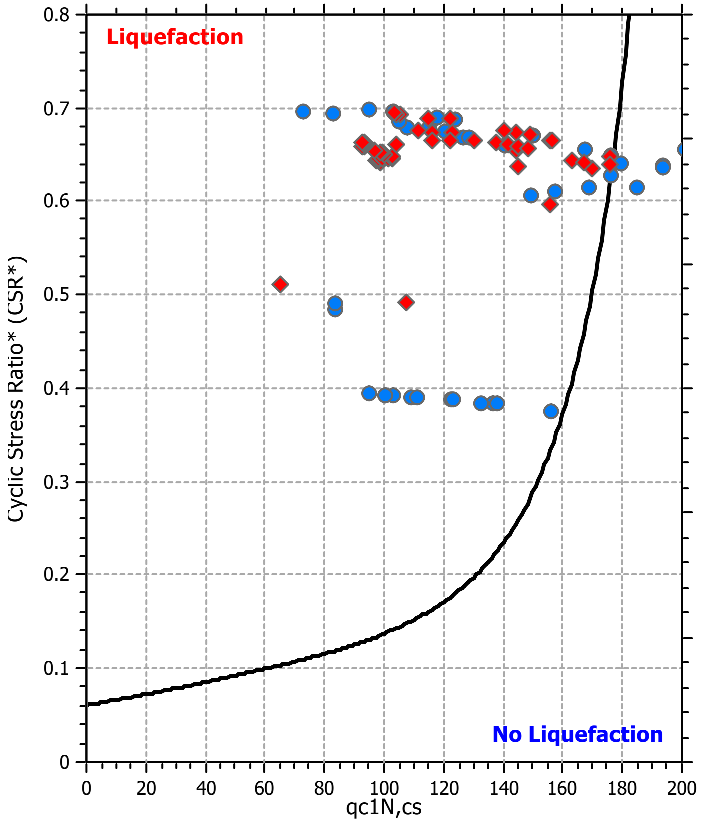
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

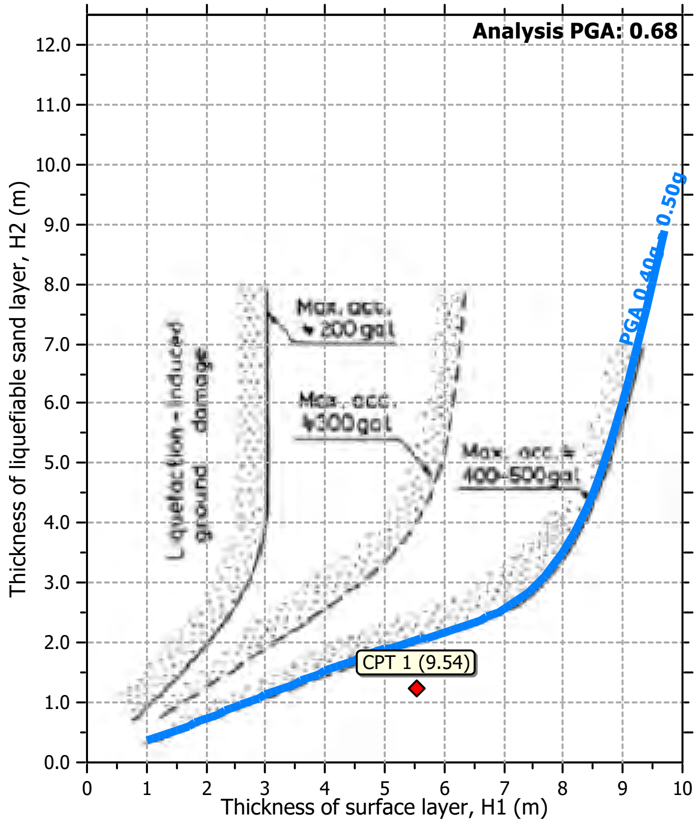
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots





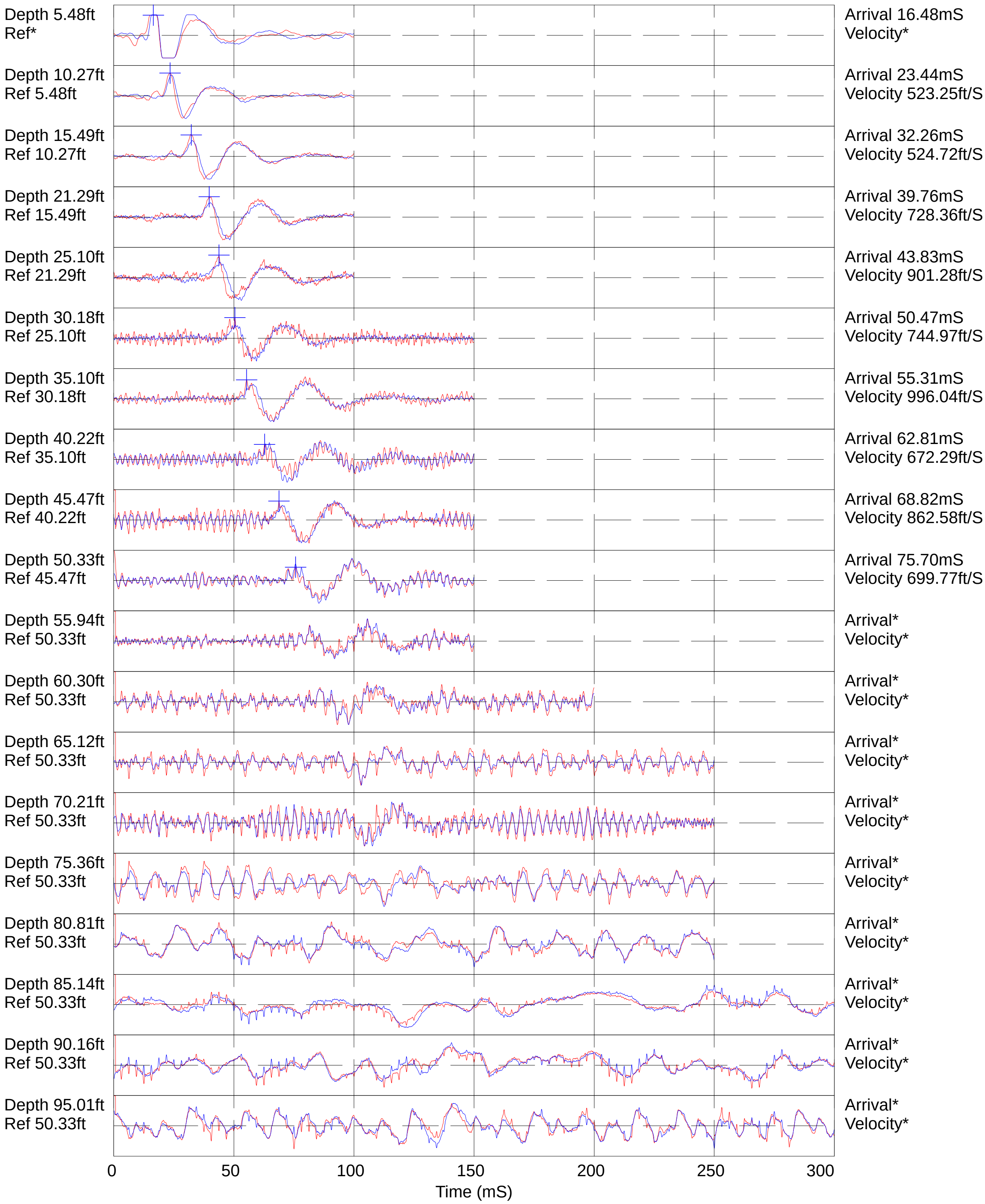


Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT-1

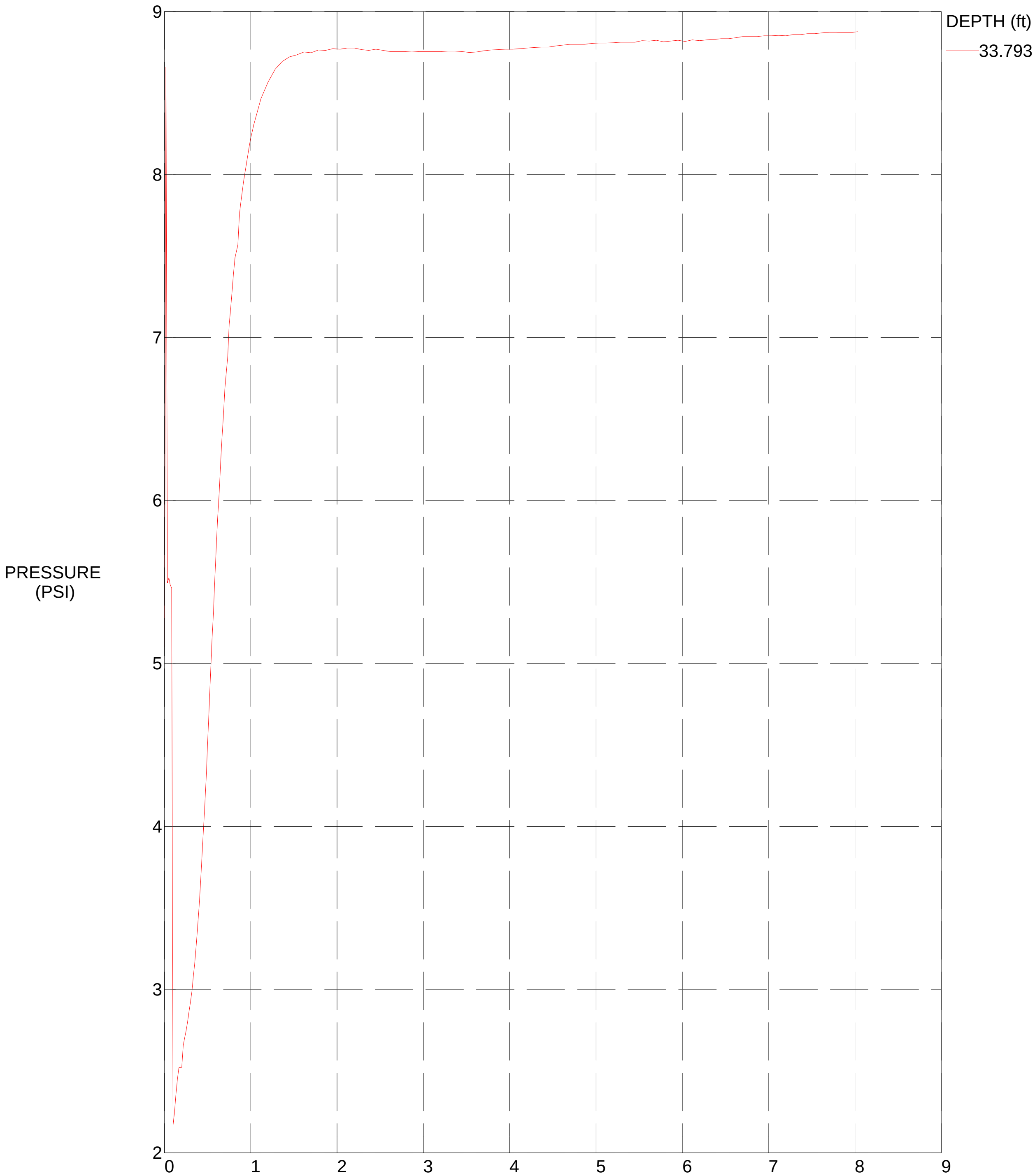
SEISMIC TEST



Hammer to Rod String Distance (ft): 6.56
* = Not Determined

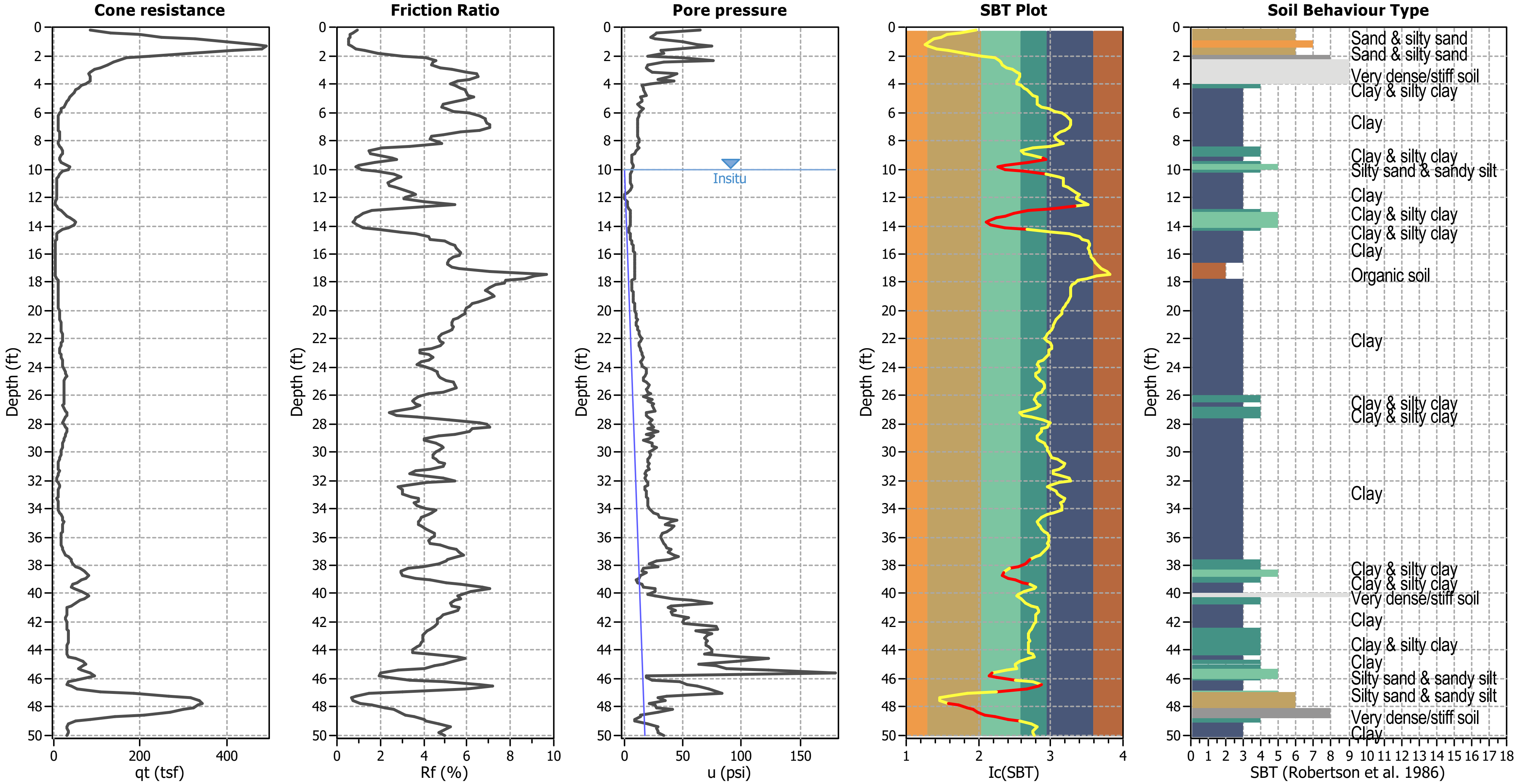
COMMENT:

CPT-1



MAXIMUM PRESSURE = 8.876 (PSI) TIME: (MINUTES)
HYDROSTATIC PRESSURE = 14.645 (PSI), WATER TABLE: 0.00 ft

CPT basic interpretation plots



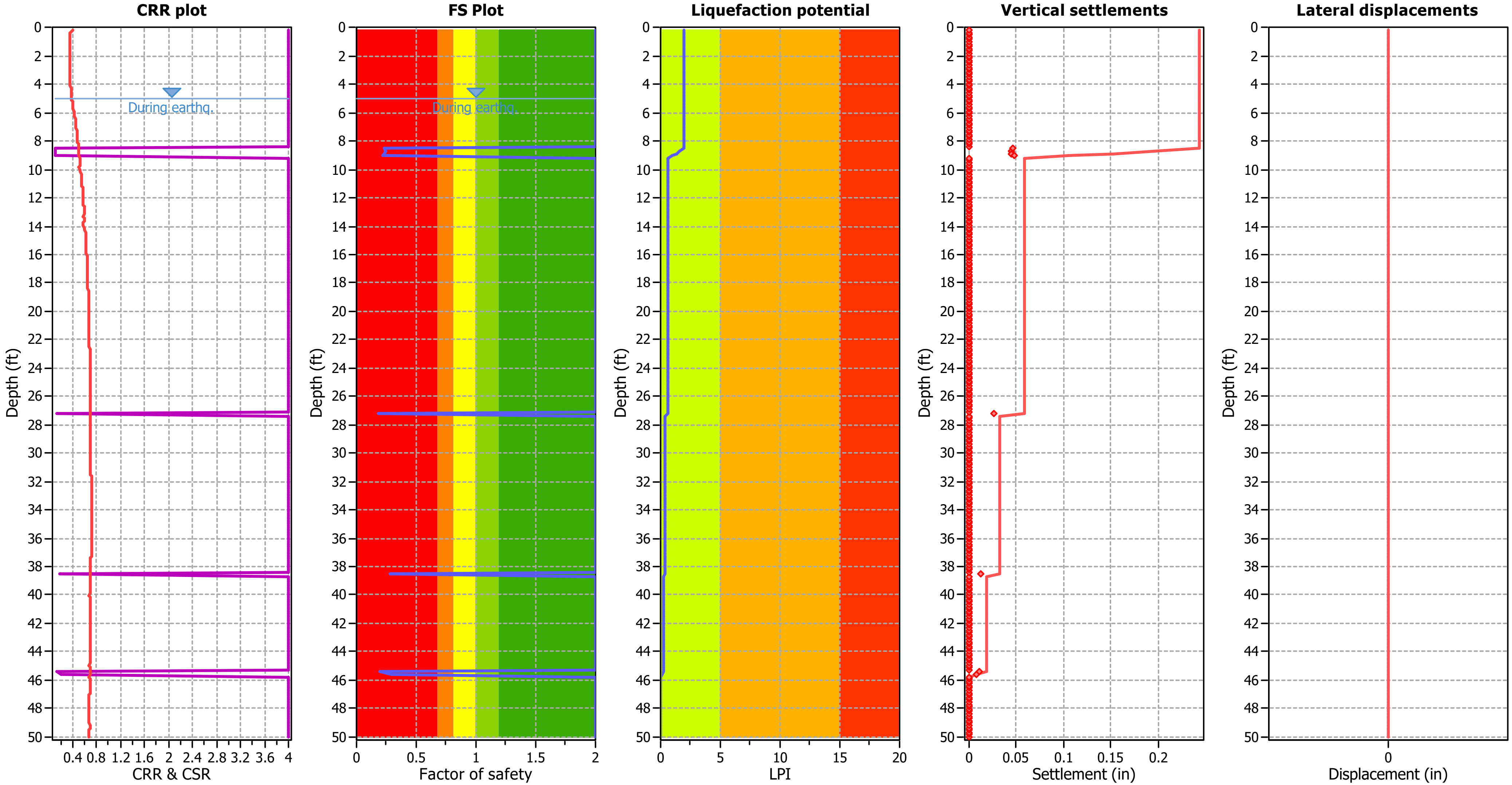
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

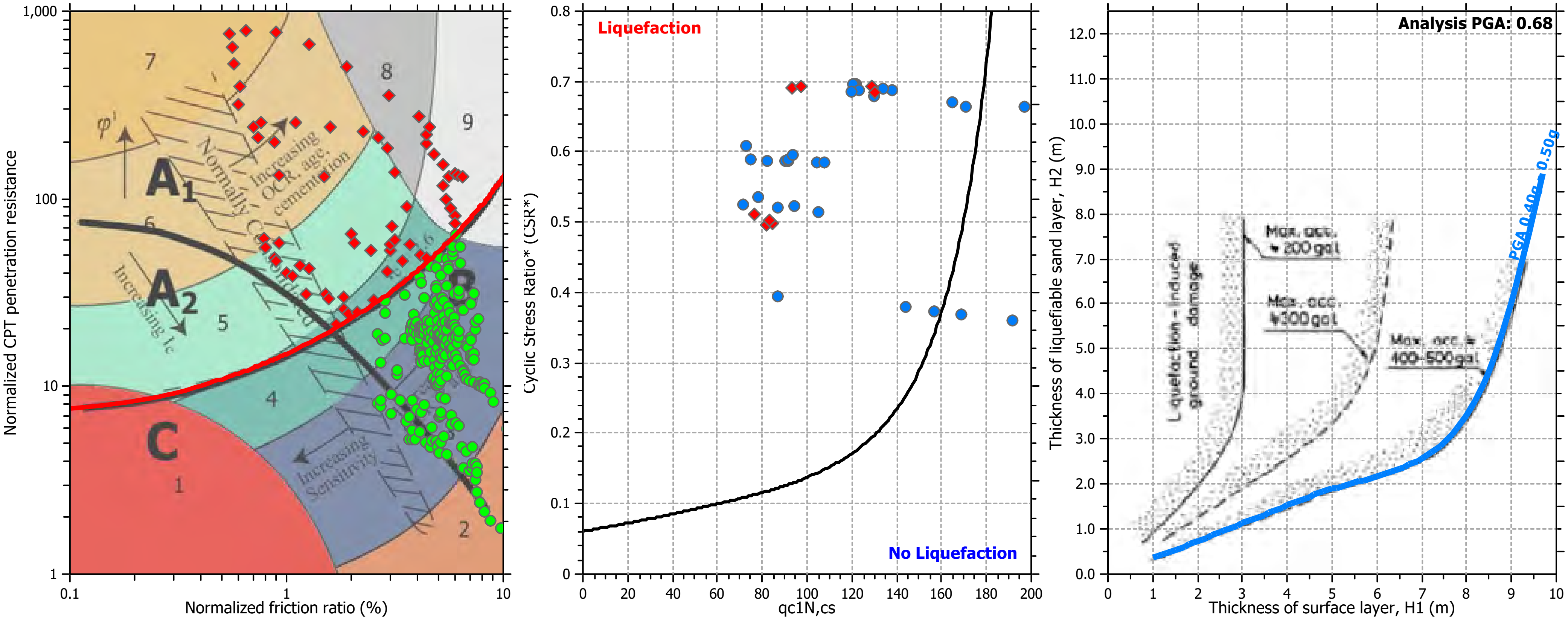
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

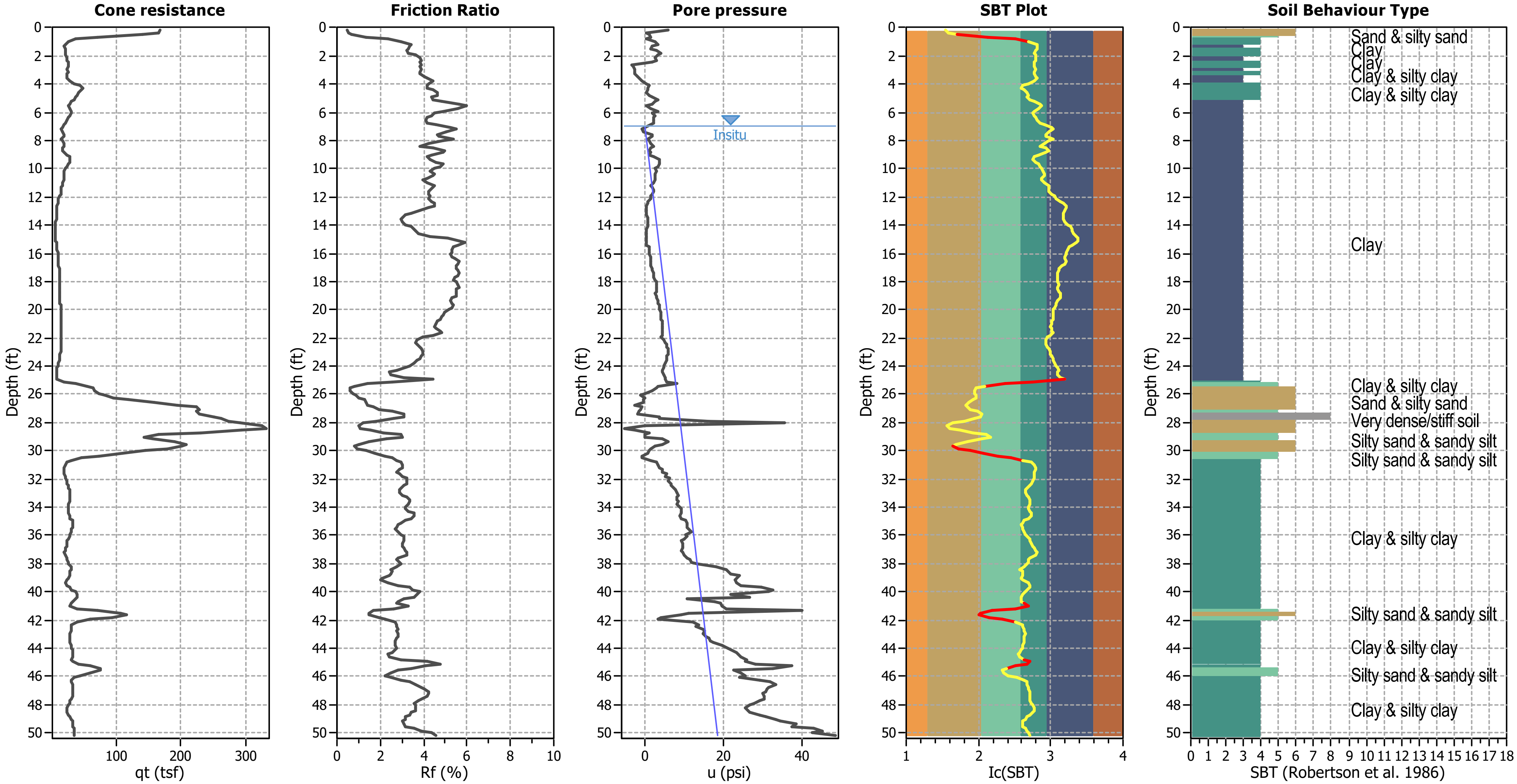
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT basic interpretation plots



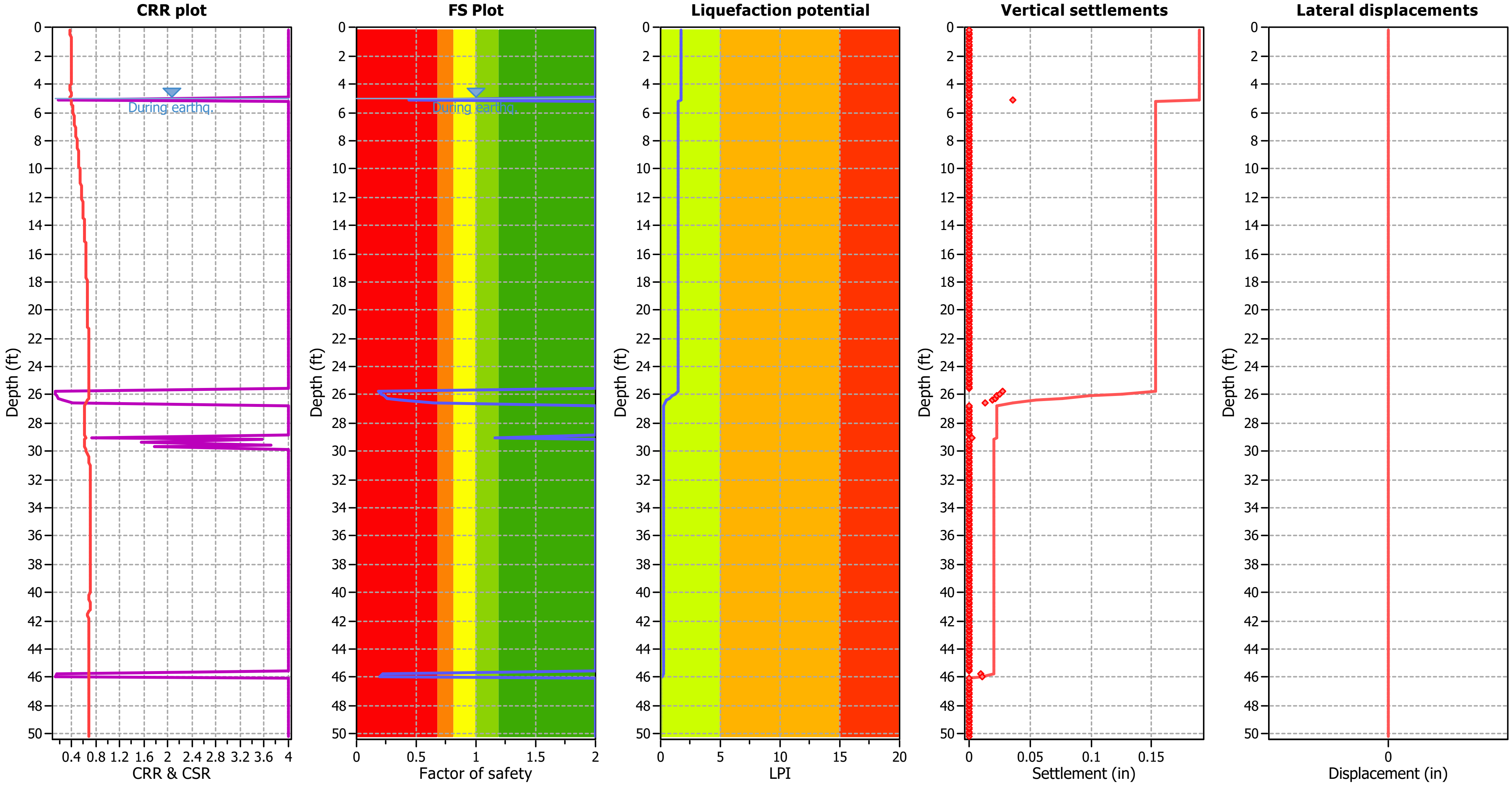
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 ft	Fill height:	N/A	Limit depth:	N/A

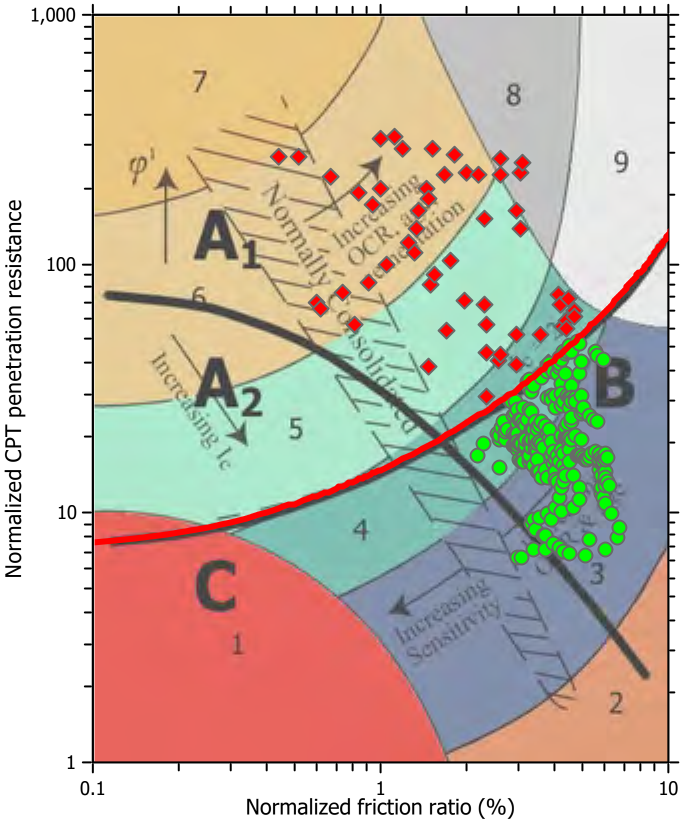
F.S. color scheme

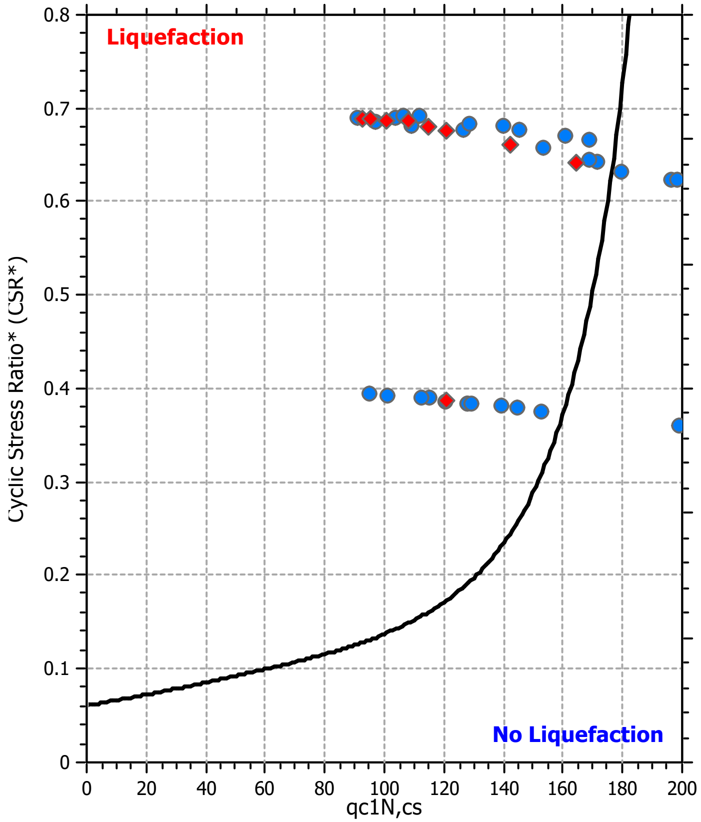
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

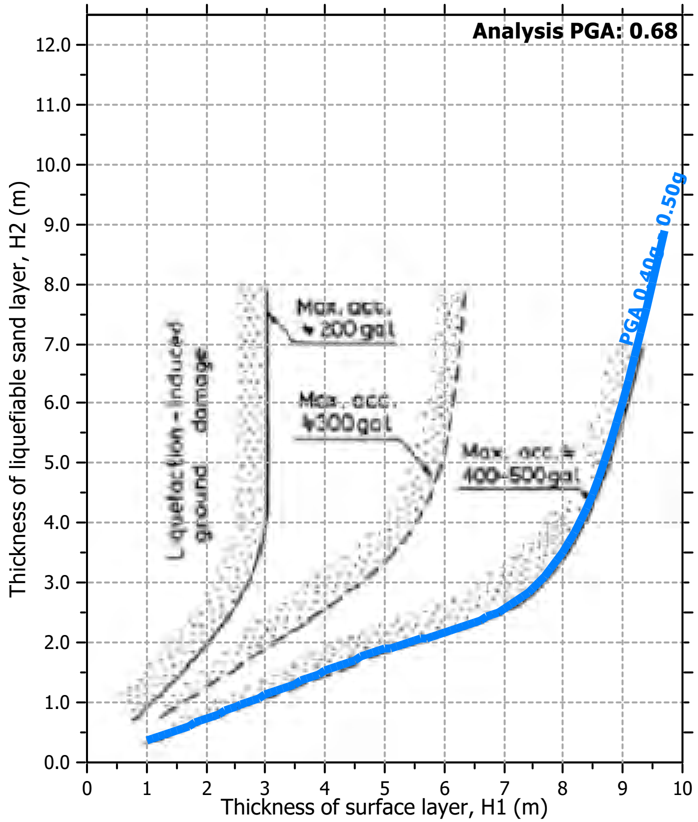
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



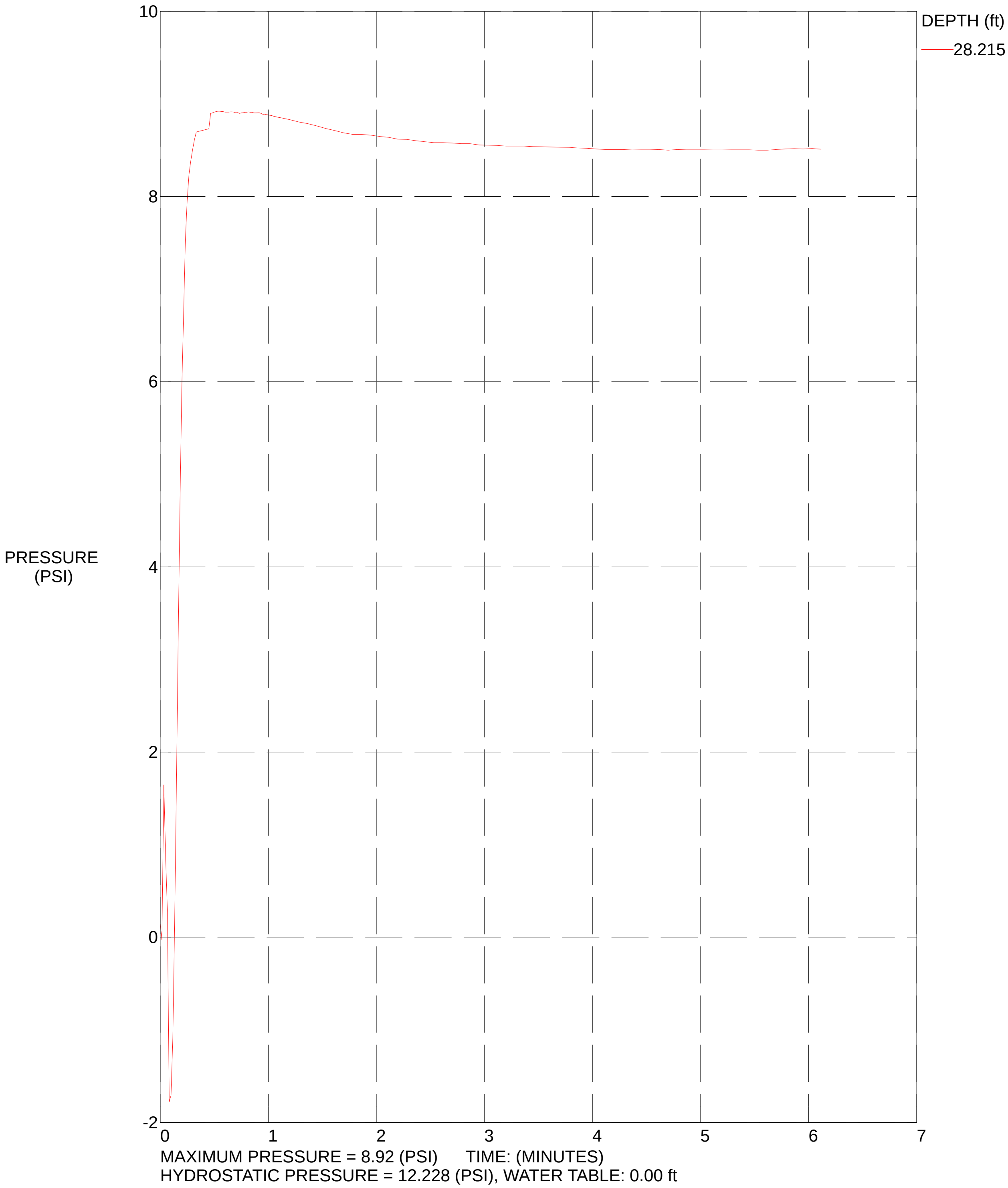




Input parameters and analysis data

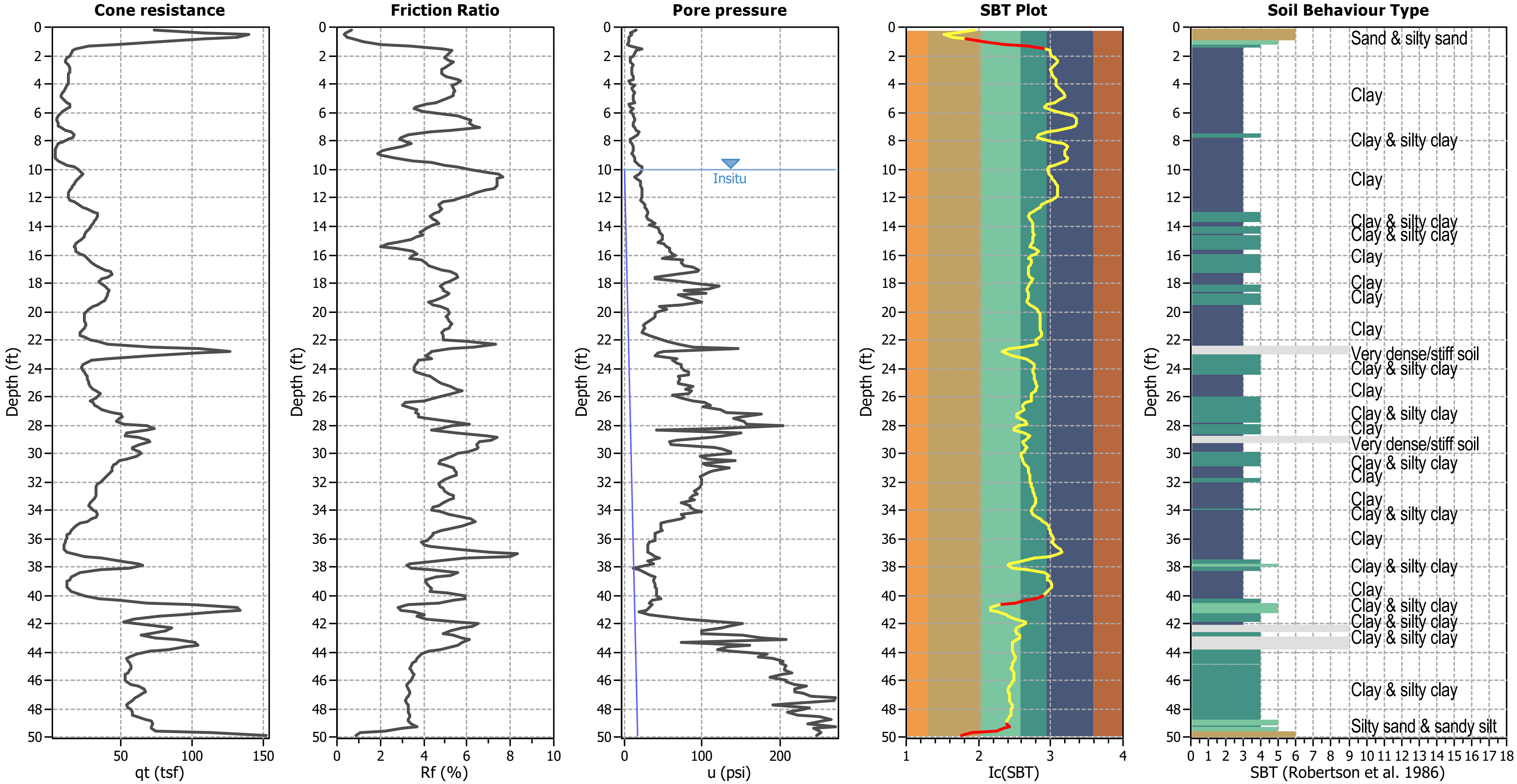
Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT-3



PRESSURE (PSI)
10
8
6
4
2
0
-2
0
1
2
3
4
5
6
7
DEPTH (ft)
28.215
MAXIMUM PRESSURE = 8.92 (PSI)
TIME: (MINUTES)
HYDROSTATIC PRESSURE = 12.228 (PSI), WATER TABLE: 0.00 ft

CPT basic interpretation plots



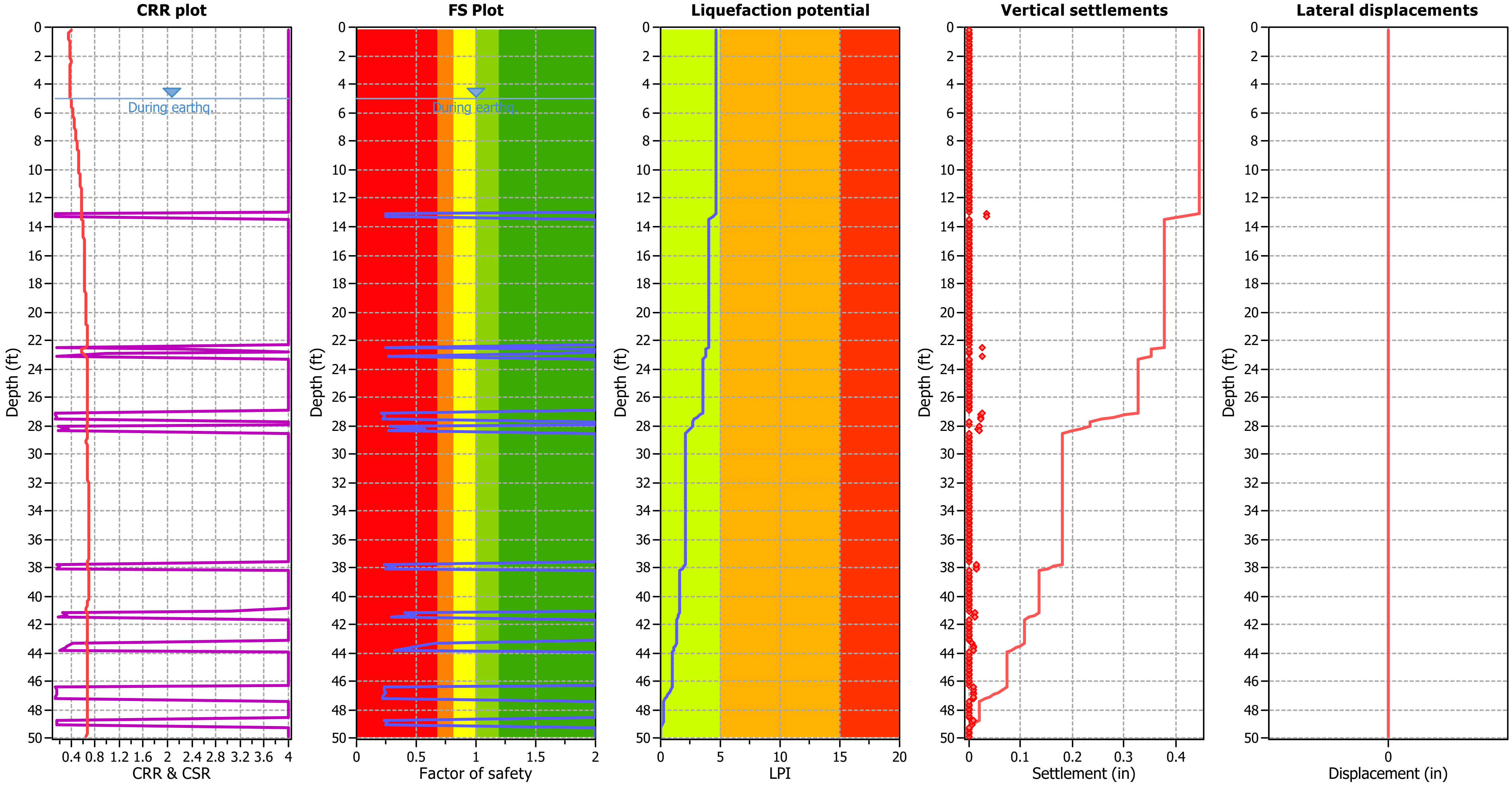
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

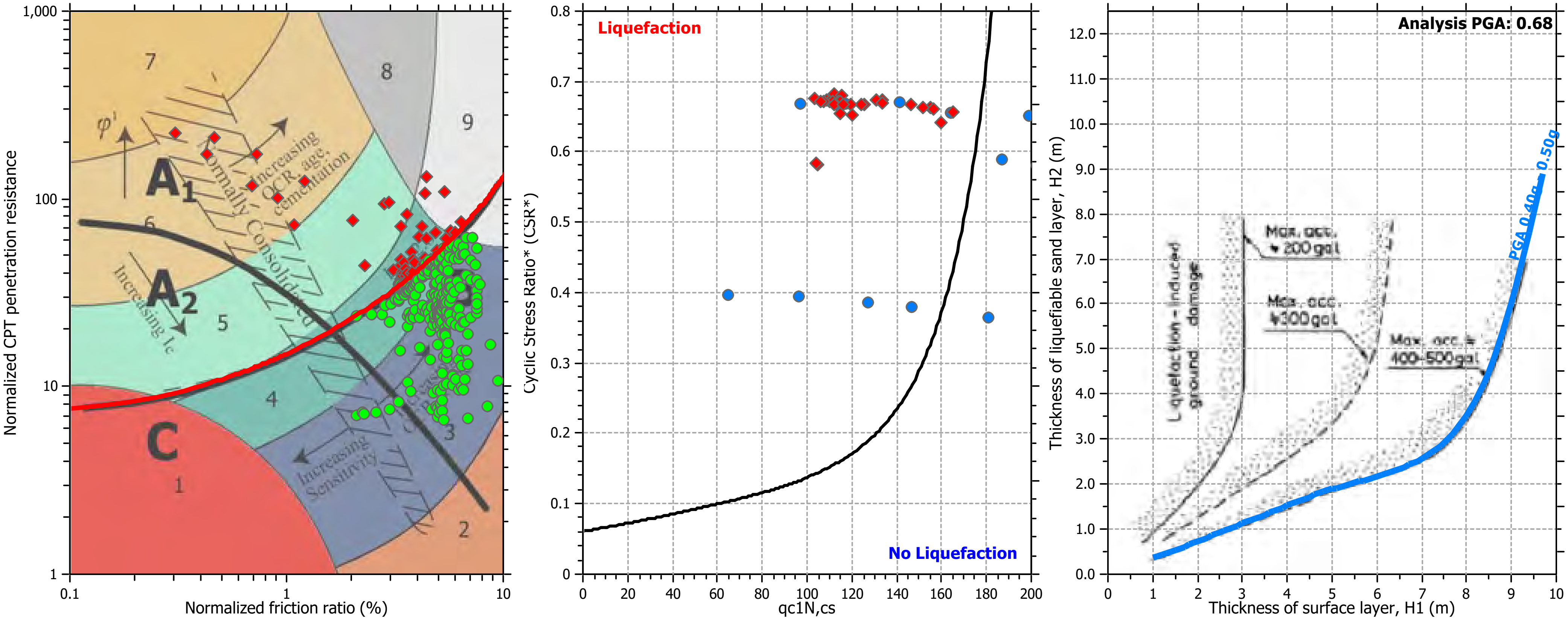
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

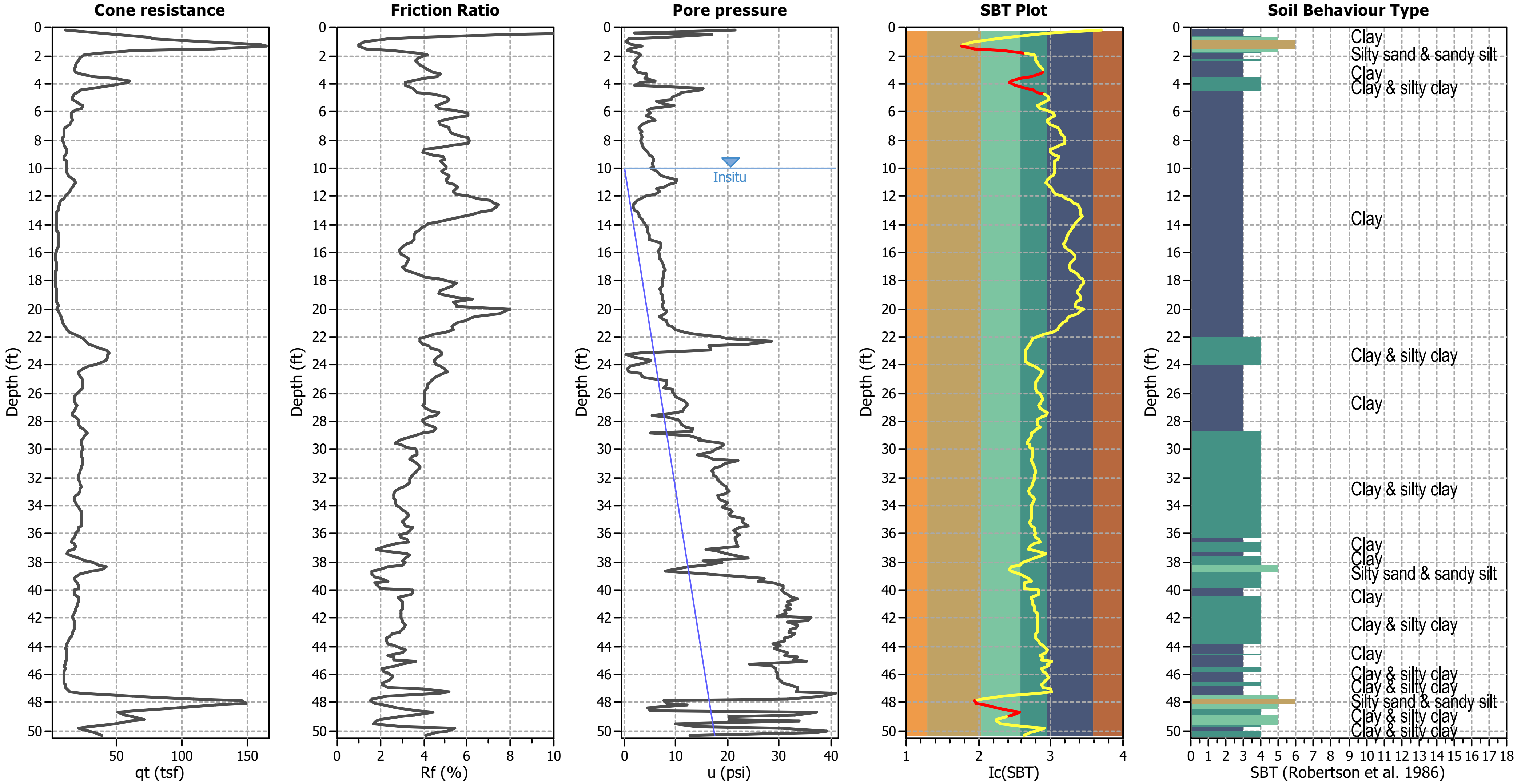
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT basic interpretation plots



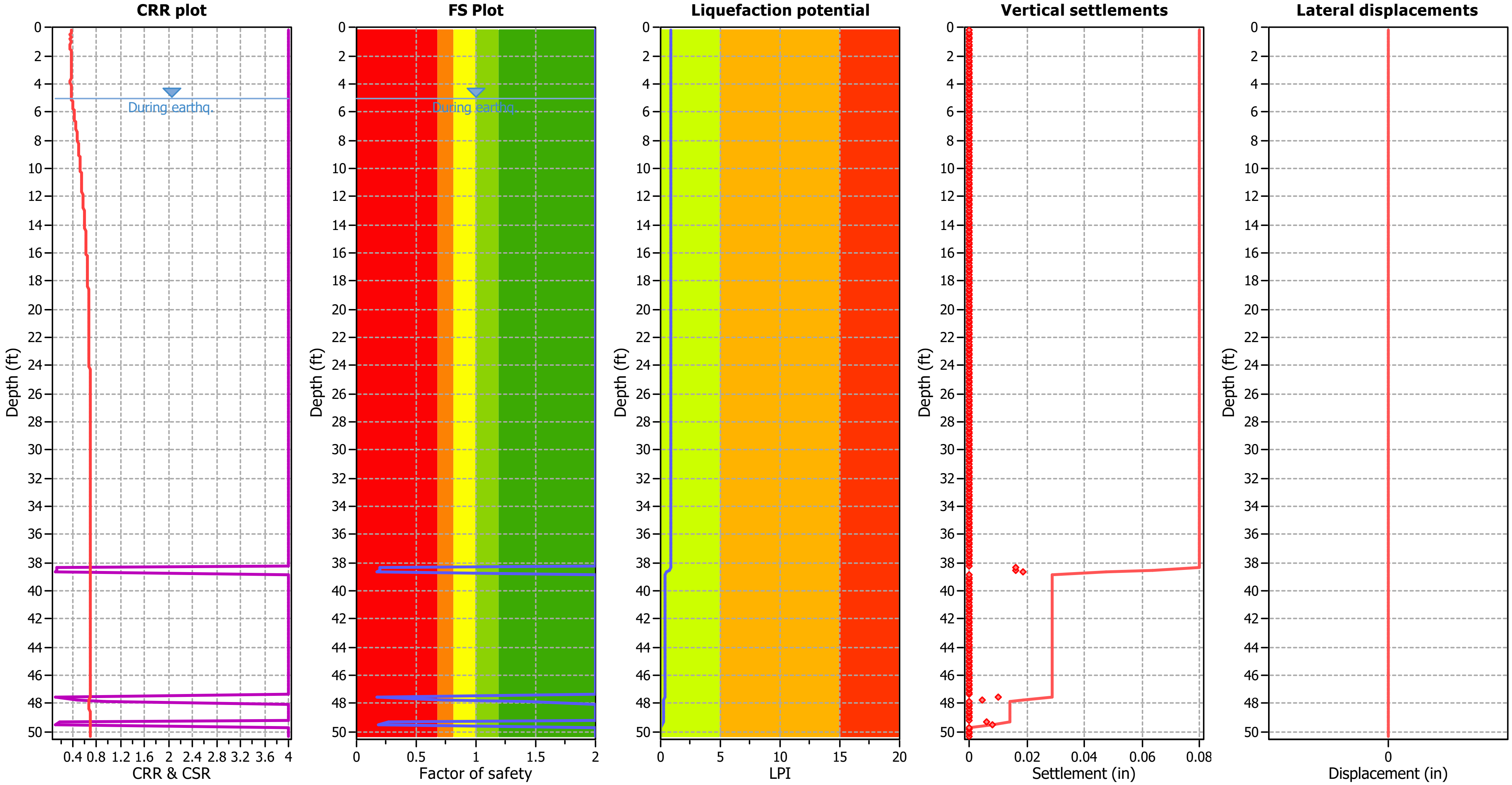
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

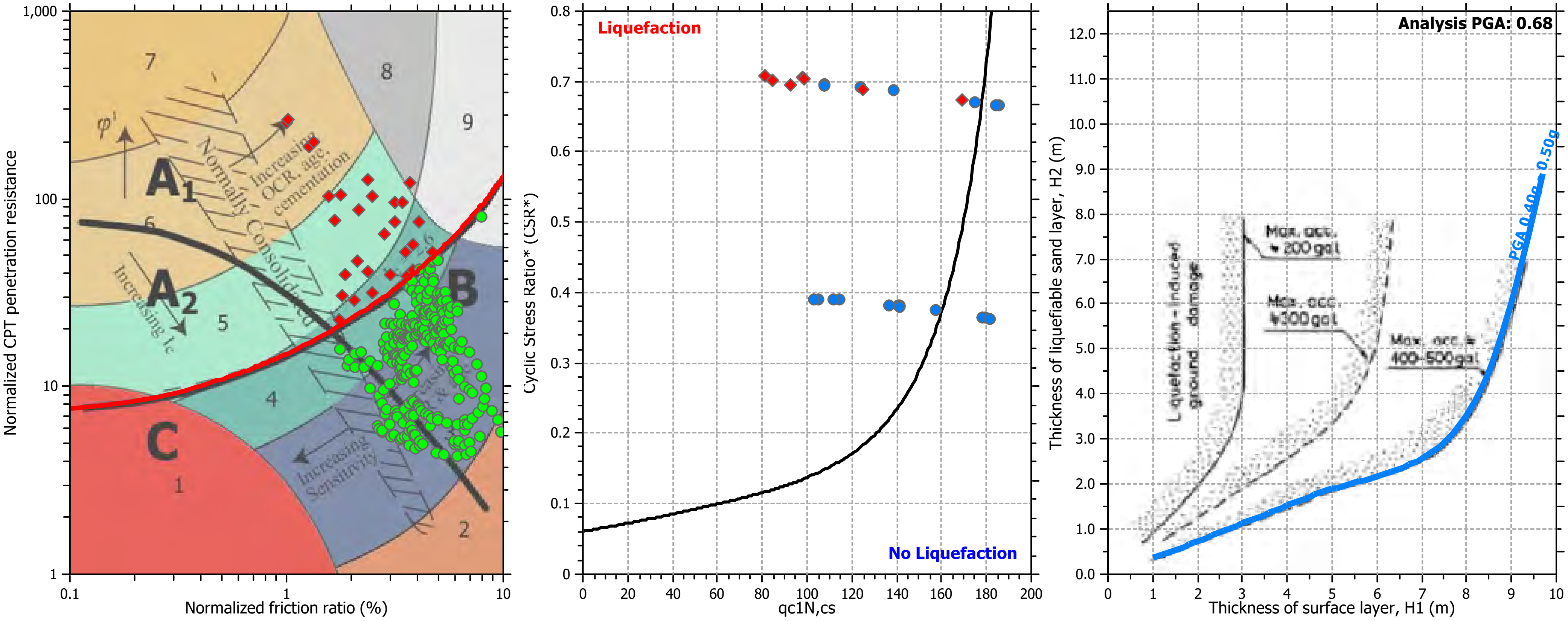
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots

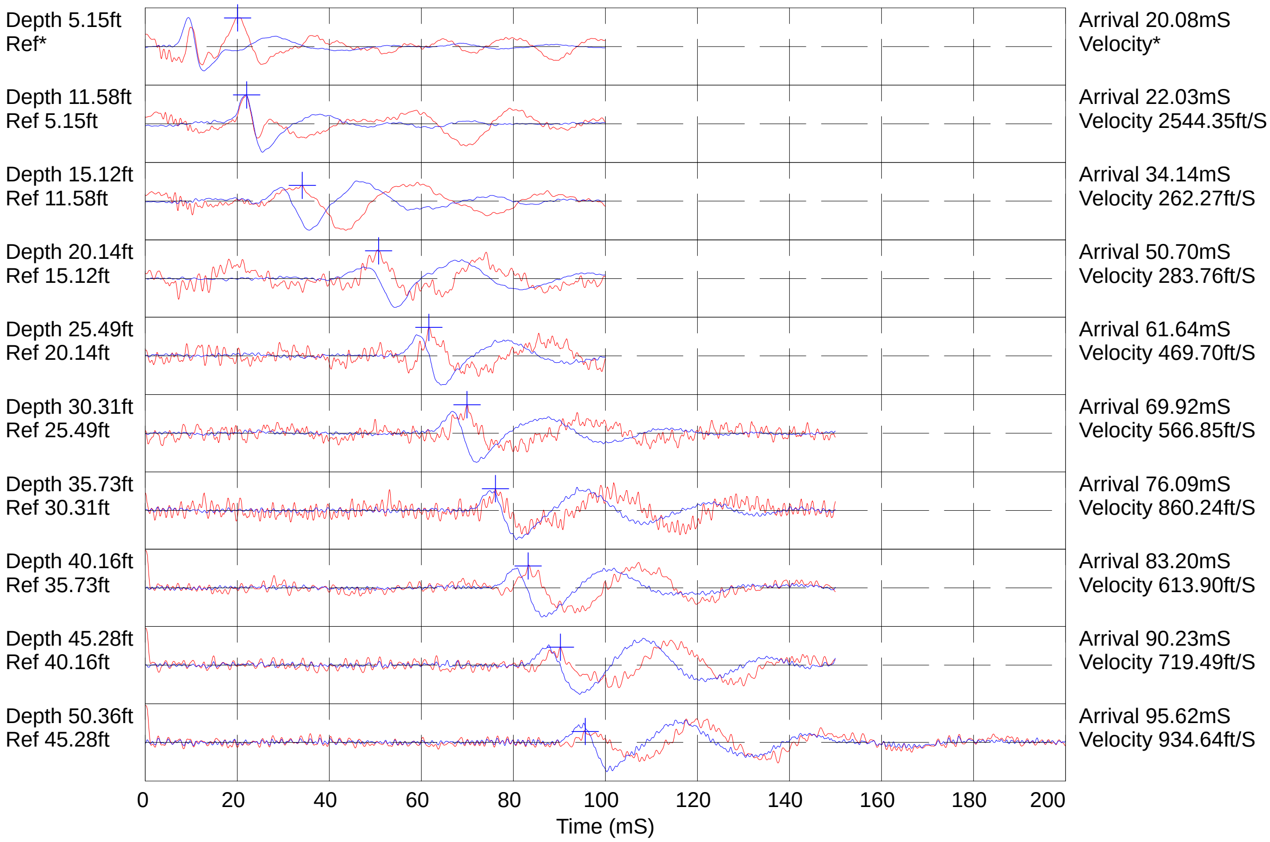


Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.22	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT-5

SEISMIC TEST

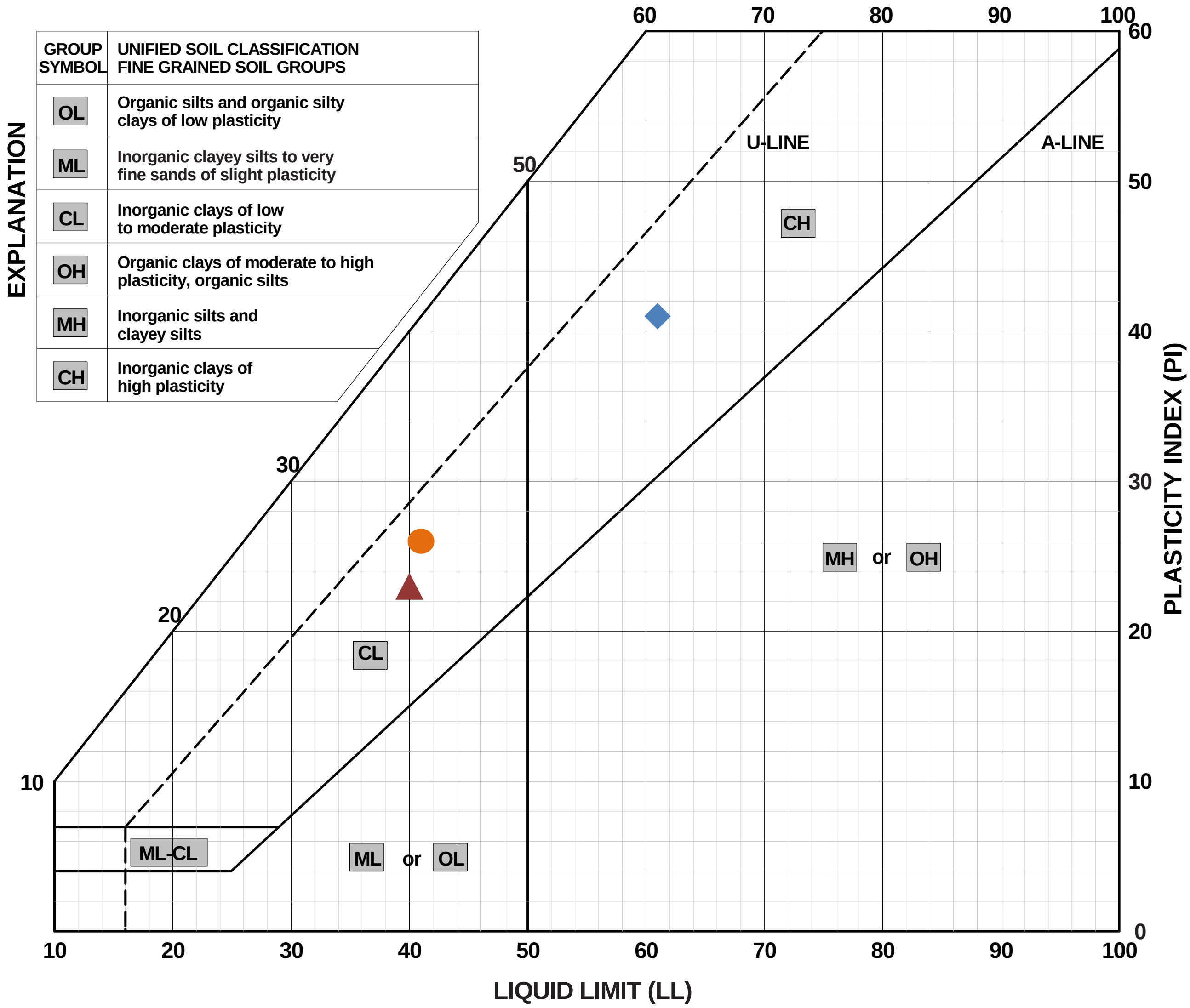


Hammer to Rod String Distance (ft): 6.56
* = Not Determined

COMMENT:

APPENDIX C

Laboratory Test Results



LEGEND:	SOURCE	DEPTH (ft)	LL	PL	PI	DESCRIPTION
◆	B-3	3.0	61	20	41	Sandy Fat Clay (CH)
▲	B-3	10.5	40	17	23	Lean Clay (CL) - Bay Mud
●	B-5	0-5	41	15	26	Lean Clay with Sand (CL)

BSK ASSOCIATES

PROJECT NO.	G00000075
DRAWN:	5/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

ATTERBERG LIMITS

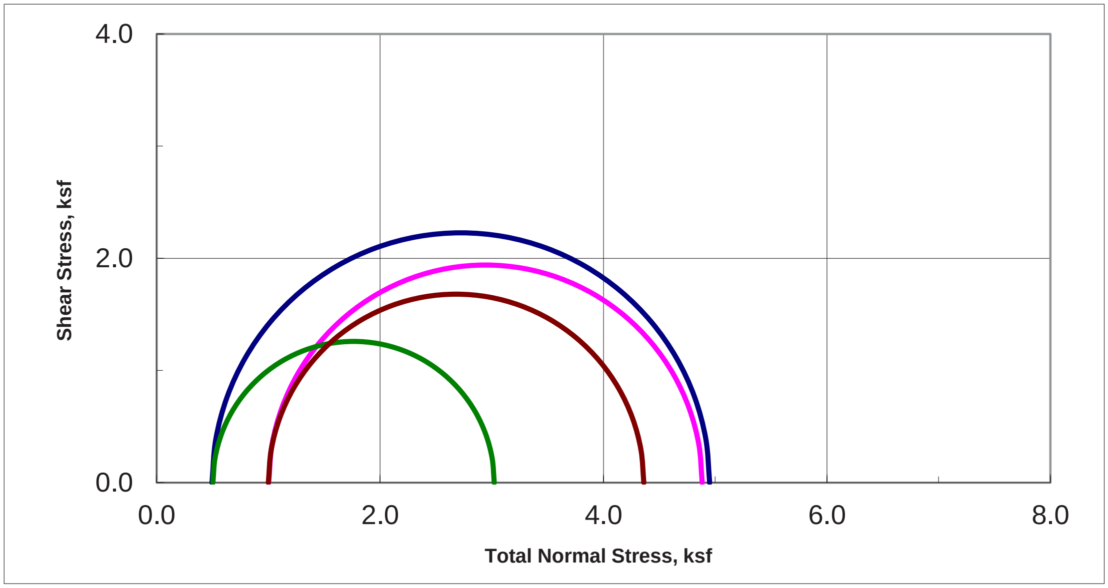
Oxidation Pond Transfer Structure
Rehab. and Oxidation Pond Storage Expansion
Ellis Creek Water Recycling Facility (WRF)
Petaluma, California

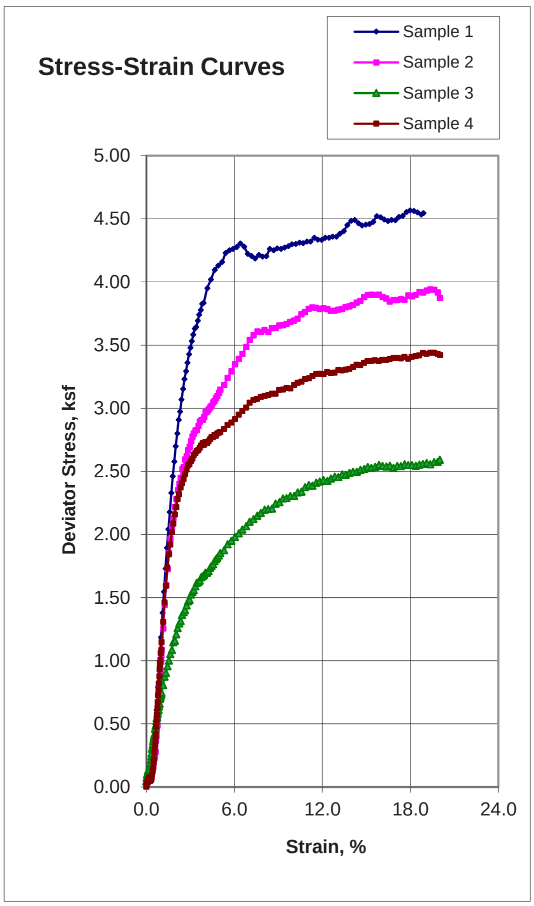
FIGURE C-1

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data	1	2	3	4
Moisture %	20.0	28.1	25.1	24.4
Dry Den,pcf	104.6	96.0	98.2	99.9
Void Ratio	0.641	0.788	0.747	0.718
Saturation %	85.5	98.0	92.3	93.5
Height in	5.03	4.97	4.98	4.98
Diameter in	2.38	2.36	2.39	2.41
Cell psi	3.4	7.0	3.5	6.9
Strain %	15.00	15.00	15.00	15.00
Deviator, ksf	4.455	3.881	2.519	3.361
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.050	0.050	0.050
Job No.:	664-482a			
Client:	BSK Associates			
Project:	G00000075			
Boring:	B-1	B-1	B-2	B-2
Sample:	2C	4C	2C	3C
Depth ft:	6.0	16.0	11.0	16.0

Visual Soil Description

Sample #	
1	Sandy Lean Clay (CL)
2	Fat Clay (CH)
3	Fat Clay with Sand (CH)
4	Fat Clay with Sand (CH)

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

BSK ASSOCIATES

PROJECT NO.	G00000075
DRAWN:	05/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

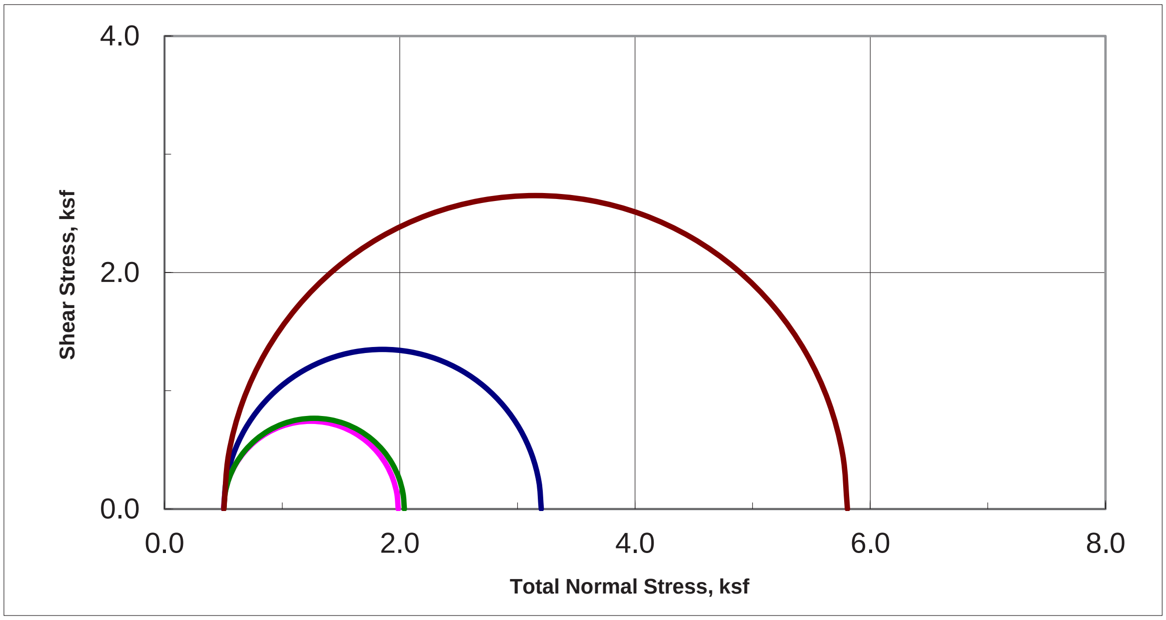
Oxidation Pond Transfer Structure Rehab. and Oxidation Pond Storage Expansion Ellis Creek Water Recycling Facility (WRF) Petaluma, California

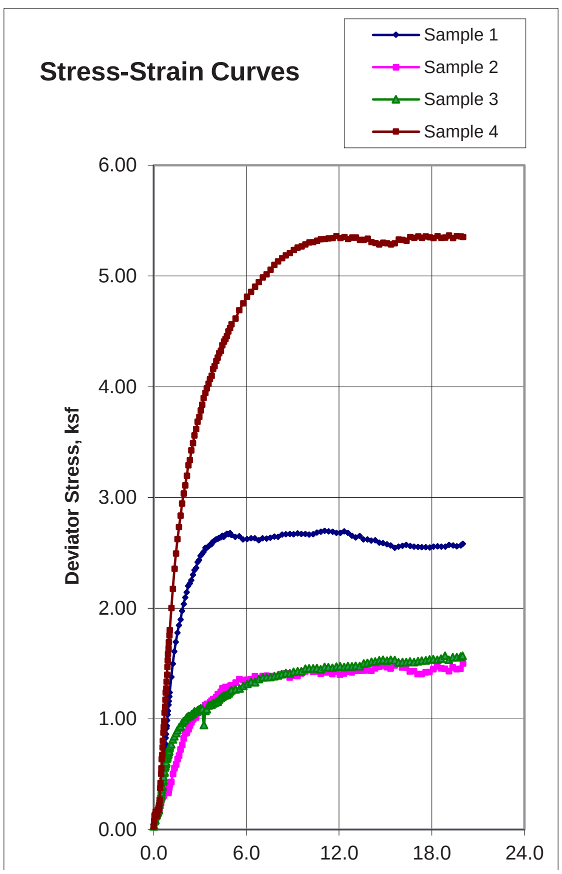
FIGURE C-2

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	24.5	39.9	37.7	25.3
Dry Den,pcf	92.6	80.0	83.9	100.8
Void Ratio	0.854	1.145	1.047	0.703
Saturation %	79.0	95.9	99.1	99.1
Height in	5.03	4.94	5.00	4.99
Diameter in	2.37	2.38	2.41	2.41
Cell psi	3.5	3.5	3.5	3.5
Strain %	11.07	15.00	15.00	15.00
Deviator, ksf	2.700	1.485	1.535	5.302
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.049	0.050	0.050
Job No.:	**664-482b**			
Client:	**BSK Associates**			
Project:	**G00000075**			
Boring:	B-3	B-3	B-4	B-5
Sample:	1C	3C	3B	3C
Depth ft:	3.5	11.0	10.5	11.0

Visual Soil Description

Sample #	
1	Sandy Fat Clay (CH)
2	Lean Clay (CL)
3	Fat Clay (CH)
4	Lean Clay with Sand (CL)

Remarks:

BSK ASSOCIATES

PROJECT NO.	**G00000075**
DRAWN:	05/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

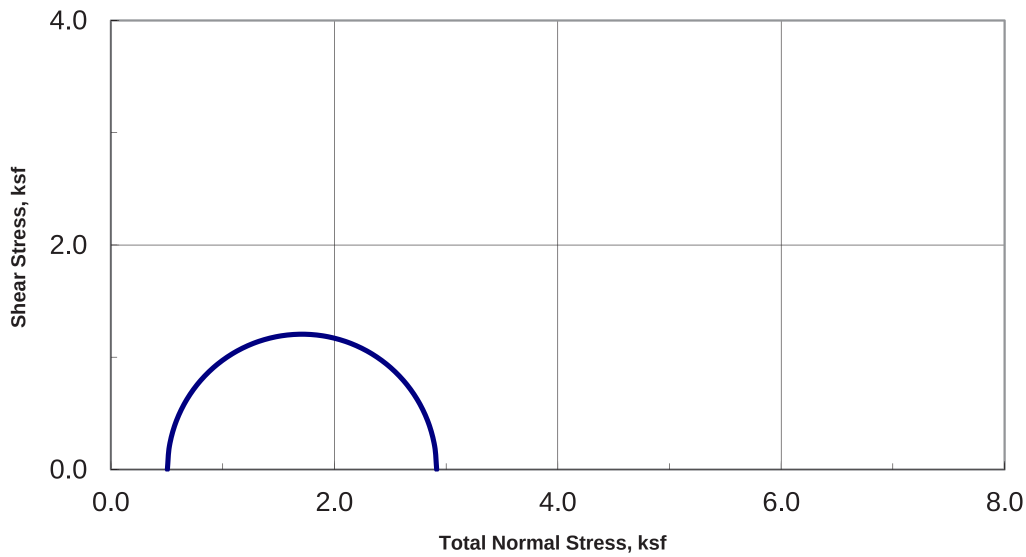
Oxidation Pond Transfer Structure Rehab. and Oxidation Pond Storage Expansion Ellis Creek Water Recycling Facility (WRF) Petaluma, California

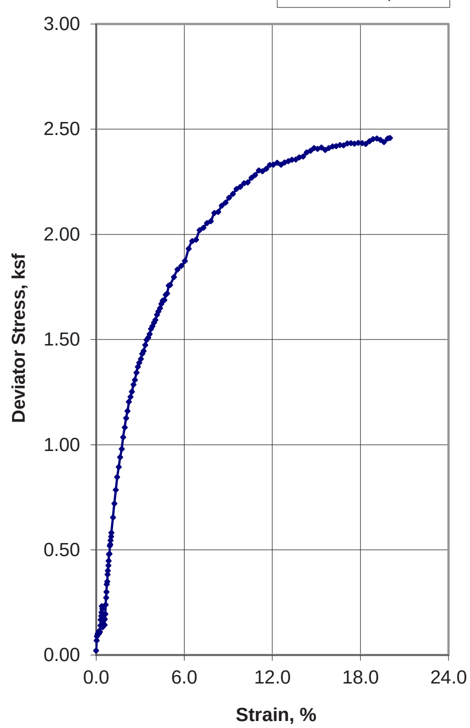
FIGURE

C-3

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	31.3			
Dry Den,pcf	92.1			
Void Ratio	0.864			
Saturation %	99.5			
Height in	4.94			
Diameter in	2.40			
Cell psi	3.5			
Strain %	15.00			
Deviator, ksf	2.411			
Rate %/min	1.00			
in/min	0.049			
Job No.:	664-482c			
Client:	BSK Associates			
Project:	G00000075			
Boring:	B-5			
Sample:	5C			
Depth ft:	21.0			

Visual Soil Description	
Sample #	
1	Fat Clay (CH)
2	
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

BSK ASSOCIATES

PROJECT NO.	G00000075
DRAWN:	05/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

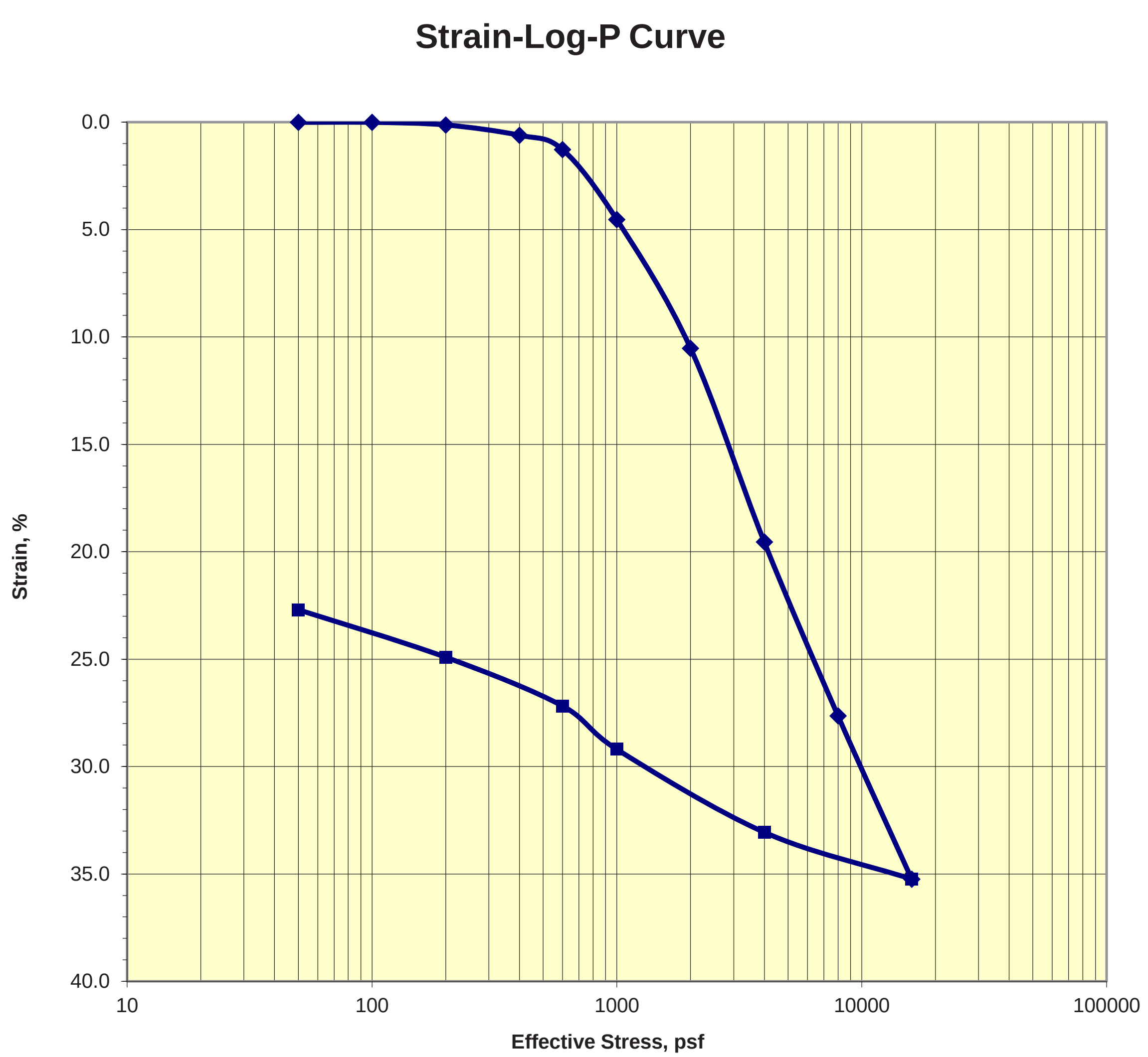
UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

Oxidation Pond Transfer Structure Rehab. and Oxidation Pond Storage Expansion Ellis Creek Water Recycling Facility (WRF) Petaluma, California

FIGURE C-4

Consolidation Test
ASTM D2435

Job No.:	664-482	**Boring:**	B-3	**Run By:**	HM
Client:	BSK Associates	**Sample:**	4C	**Reduced:**	RU
Project:	G00000075	**Depth, ft.:**	16	**Checked:**	PJ
Soil Type:	Lean Clay (CL) - Bay Mud			**Date:**	5/16/2023



Assumed Gs 2.75	Initial	Final
Moisture %:	88.8	61.5
Dry Density, pcf:	49.2	63.8
Void Ratio:	2.488	1.692
% Saturation:	98.1	100.0

Remarks:

BSK ASSOCIATES

PROJECT NO.	G00000075
DRAWN:	05/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

CONSOLIDATION TEST

Oxidation Pond Transfer Structure
Rehab. and Oxidation Pond Storage Expansion
Ellis Creek Water Recycling Facility (WRF)
Petaluma, California

FIGURE C-5

Moisture-Density Relationship

Report #: MDRS-000001

Client:	**Project:**
Dudek 605 Third Street Encinitas, CA 92024	G00000075 ECWRF Oxidation Pond Transfer Structure Rehab. & Oxidation Pond Storage Expansio... 3890 Cypress Drive Petaluma, CA 94954

Sample Details

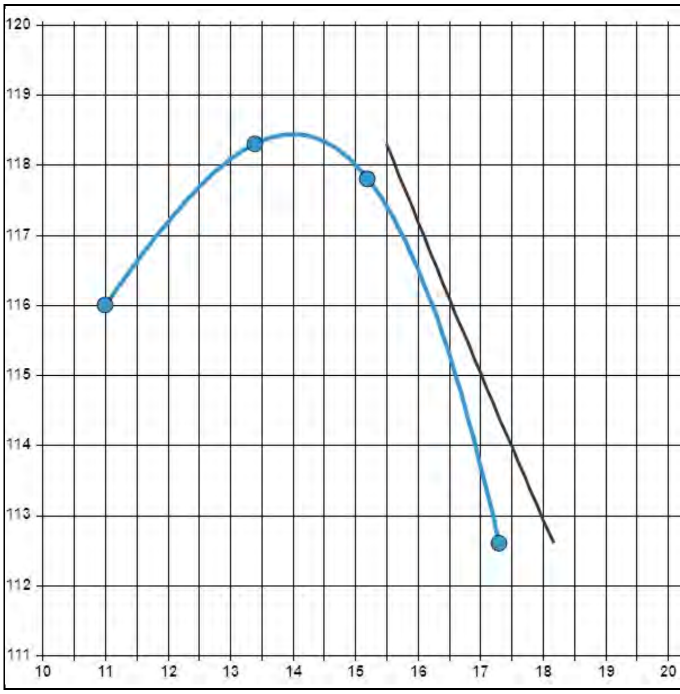
Technician: Omar Khan

Sample Date: 04/14/2023

Sample Location: B-2: Bulk @ 0-5'

Sample Number: 25747

ASTM D1557



Method: B (ASTM D1557)

Preparation Method: Moist

Rammer Type: Mechanical Round

Specific Gravity: 2.69

Maximum Dry Density (pcf): 118.4

Optimum Moisture (%): 14.0

Soil Classification: Sandy Lean Clay (CL)

Test Notes: B-2: Bulk @ 0-5'

Test Completed By: David Ordaz

Test Completed Date: 04/25/2023

Approved By: Nicholas Shelly

Approved Date: 04/26/2023

BSK ASSOCIATES

PROJECT NO. G00000075

DRAWN: 05/22/23

DRAWN BY: D. Tower

CHECKED BY: C. Melo

FILE NAME: Figures.indd

MOISTURE DENSITY RELATIONSHIP

Oxidation Pond Transfer Structure Rehab. and Oxidation Pond Storage Expansion Ellis Creek Water Recycling Facility (WRF) Petaluma, California

FIGURE

C-6

Moisture-Density Relationship

Report #: MDRS-000002

Client:	**Project:**
Dudek 605 Third Street Encinitas, CA 92024	G00000075 ECWRF Oxidation Pond Transfer Structure Rehab. & Oxidation Pond Storage Expansio... 3890 Cypress Drive Petaluma, CA 94954

Sample Details

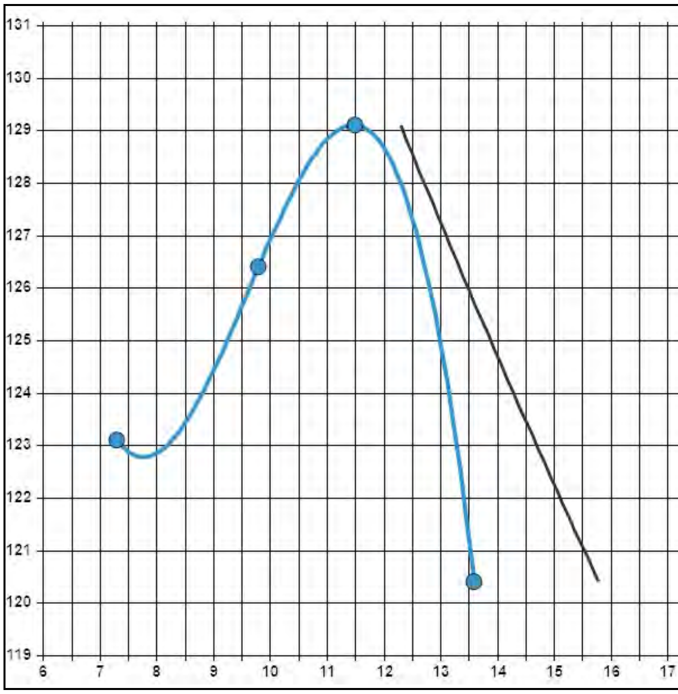
Technician: Omar Khan

Sample Date: 04/14/2023

Sample Location: B-3: Bulk @ 0-5"

Sample Number: 25749

ASTM D1557



Method: B (ASTM D1557)

Preparation Method: Moist

Rammer Type: Mechanical Round

Specific Gravity: 2.78

Maximum Dry Density (pcf): 129.1

Optimum Moisture (%): 11.5

Soil Classification: Sandy Fat Clay (CH)

Test Notes: B-3: Bulk @ 0-5'

Test Completed By: David Ordaz

Test Completed Date: 04/26/2023

Approved By: Nicholas Shelly

Approved Date: 04/27/2023

BSK ASSOCIATES

PROJECT NO.	G00000075
DRAWN:	05/22/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	Figures.indd

MOISTURE DENSITY RELATIONSHIP

Oxidation Pond Transfer Structure Rehab. and Oxidation Pond Storage Expansion
Ellis Creek Water Recycling Facility (WRF)
Petaluma, California

FIGURE C-7

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

12 May, 2023

Job No. 2304036
Cust. No. 12667

Mr. Michael Romero
BSK Associates Engineers & Laboratories
399 Lindbergh Avenue
Livermore, CA 94551

Subject: Project No.: G00000075
Project Name: Ellis Creek WRF Pond
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Romero:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 26, 2023. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Samples No.002 and No.003 are classified as "severely corrosive" and Sample No.001 is classified as "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 ranged from 200 mg/kg to 2300 mg/kg and determined to be sufficient to attack steel embedded in a concrete mortar coating. Chloride ion concentrations greater than 300 mg/kg are considered corrosive to embedded reinforcing steel; and, as such, the concrete mix design shall be adjusted accordingly by a qualified corrosion engineer.

The sulfate ion concentrations ranged 27 mg/kg to 390 mg/kg and are determined to be sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, with a maximum water-to-cement ratio of 0.55.

The pH of the soils ranged from 8.11 to 8.34, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 150-mV to 210-mV. Sample No.002 is indicative of potentially "moderately corrosive" soils and the remaining 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.

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

JDH/jdl
Enclosure

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

Client: BSK Associates Engineers & Laboratories
Client's Project No.: G00000075
Client's Project Name: Ellis Creek WRF Pond
Date Sampled: 02/16 - 04/14/23
Date Received: 26-Apr-23
Matrix: Soil
Authorization: Signed Chain of Custody

B-3

5'

Date of Report: 12-May-2023

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)*
2304036-001	B-3/Bulk @ 0-~~6~~'	210	8.28	-	530	-	200	390
2304036-002	~~B~~-4/4B @ 15.5'	150	8.34	-	110	-	2,300	280
2304036-003	B-5/Bulk @ 0-5'	200	8.11	-	240	-	810	27

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	3-May-2023	3-May-2023	-	4-May-2023	-	10-May-2023	10-May-2023

* Results Reported on "As Received" Basis

N.D. - None Detected

Sherri Moore
Chemist

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

APPENDIX D

Previous Subsurface and Laboratory Test Data (by Others)

DATE DRILLED: November 8, 2012	**NOTES:**
DRILLING CONTRACTOR: Pearson Drilling	*Equivalent Standard Penetration Test (SPT) blow count.
DRILLING METHOD: 6 inch Solid Flight	
HAMMER WEIGHT: 140 lbs. / **DROP:** 30 inches	
LOGGED BY: KSG / **ELEVATION:** Levee Surface	

FIELD					LABORATORY							
DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0				Crushed Gravel Surface								0
				GRAY GREEN SILTY CLAY (CH), stiff, wet (Fill)								
		11										
4												4
8												8
		8										
12												12
				GRAY GREEN SILTY CLAY (CH), stiff, wet								
		6										
16												16
20				Becomes medium stiff								20
		3										
				Bottom of Boring B-1 at 21 1/2 feet Water encountered at 13 1/2 feet Converted to 2 inch piezometer								
24												24

RGH CONSULTANTS

Job No: 2553.08.04.1 Date: 12/4/2012

LOG OF BORING B-1

Ellis Creek Oxidation Ponds 7 and 10
Sheet Pile Levee Project
Petaluma, California

PLATE

3

DATE DRILLED: November 8, 2012		NOTES:
DRILLING CONTRACTOR: Pearson Drilling		*Equivalent Standard Penetration Test (SPT) blow count.
DRILLING METHOD: 6 inch Solid Flight		
HAMMER WEIGHT: 140 lbs.	DROP: 30 inches	
LOGGED BY: KSG	ELEVATION: Levee Surface	

FIELD					LABORATORY							
DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0				Crushed Gravel Surface								0
				GRAY GREEN SILTY CLAY (CH), stiff, wet (Fill)								
4												4
8												8
12		10										12
				GRAY BROWN SILTY CLAY (CH), stiff, wet								
		10										
				BLACK SILTY CLAY (CH), stiff, wet								
		7										
16												16
				GRAY GREEN SILTY CLAY (CH), stiff, wet								
20												20
24												24
		7										
				Bottom of Boring B-2 at 25 feet No free water encountered								

RGH
CONSULTANTS

Job No: 2553.08.04.1 | Date: 12/4/2012

LOG OF BORING B-2

Ellis Creek Oxidation Ponds 7 and 10
Sheet Pile Levee Project
Petaluma, California

PLATE
4

DATE DRILLED: November 8, 2012		NOTES:
DRILLING CONTRACTOR: Pearson Drilling		*Equivalent Standard Penetration Test (SPT) blow count.
DRILLING METHOD: 6 inch Solid Flight		
HAMMER WEIGHT: 140 lbs.	DROP: 30 inches	
LOGGED BY: KSG	ELEVATION: Levee Surface	

DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0				Crushed Gravel Surface								0
				GRAY GREEN SILTY CLAY (CH), stiff, wet (Fill)								
4												4
8												8
12				With some fine sand from 12 to 13 feet								12
		10		BLACK SILTY CLAY (CH), stiff, wet, with occasional thin peat layers from 14 to 17 feet								
		8										
16												16
				GRAY GREEN SILTY CLAY (CH), stiff, wet								
20		10										20
				Bottom of Boring B-3 at 20 feet No free water encountered								
24												24

RGH CONSULTANTS

Job No: 2553.08.04.1 | Date: 12/4/2012

LOG OF BORING B-3

Ellis Creek Oxidation Ponds 7 and 10
Sheet Pile Levee Project
Petaluma, California

PLATE 5

DATE DRILLED: November 8, 2012		NOTES:
DRILLING CONTRACTOR: Pearson Drilling		*Equivalent Standard Penetration Test (SPT) blow count.
DRILLING METHOD: 6 inch Solid Flight		
HAMMER WEIGHT: 140 lbs.	DROP: 30 inches	
LOGGED BY: KSG	ELEVATION: Levee Surface	

DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
FIELD					LABORATORY							
0				Crushed Gravel Surface								0
				GRAY GREEN SILTY CLAY (CH), stiff, wet (Fill)								
4												4
8												8
12												12
				GRAY GREEN SILTY CLAY (CH), medium stiff, wet								
		4										
16				Thin peat lense at 16 feet								16
		4										
20				Bottom of Boring B-3 at 20 feet No free water encountered Converted to 2 inch piezometer								20
24												24

RGH CONSULTANTS

Job No: 2553.08.04.1 | Date: 12/4/2012

LOG OF BORING B-4

Ellis Creek Oxidation Ponds 7 and 10
Sheet Pile Levee Project
Petaluma, California

PLATE
6

DATE DRILLED: November 8, 2012	**NOTES:**
DRILLING CONTRACTOR: Pearson Drilling	*Equivalent Standard Penetration Test (SPT) blow count.
DRILLING METHOD: 6 inch Solid Flight	
HAMMER WEIGHT: 140 lbs. **DROP:** 30 inches	
LOGGED BY: KSG **ELEVATION:** Levee Surface	

FIELD					LABORATORY							
DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0				Crushed Gravel Surface								0
				GRAY GREEN SILTY CLAY (CH), stiff, wet (Fill)								
4												4
8												8
12		3										12
		6		GRAY GREEN SILTY CLAY (CH), medium stiff, wet								
		6		GRAY CLAYEY SAND (SC), loose, saturated								
16		3		BLUE GRAY SILTY CLAY (ch), stiff, saturated								16
		7										
20				Bottom of Boring B-5 at 21 1/2 feet Water encountered at 15 feet								20
24												24

RGH CONSULTANTS

Job No: 2553.08.04.1 Date: 12/4/2012

LOG OF BORING B-5

Ellis Creek Oxidation Ponds 7 and 10
Sheet Pile Levee Project
Petaluma, California

PLATE 7

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL (LITTLE OR FINES)		GW	WELL-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES (OVER 12% OF FINES)		GM	WELL-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GC	CLAYEY GRAVEL, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SAND, GRAVELLY SAND, LITTLE OR NO FINES
				SP	POORLY-GRADED SAND, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (OVER 12% OF FINES)		SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
				SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50			ML	INORGANICS SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50			MH	ORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS AND OTHER SOILS WITH HIGH ORGANIC-CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

- ■ - "Undisturbed" Sample
- ⊠ - Bulk or Disturbed Sample
- ◥ - Standard Penetration Test
- ⧄ - Sample Attempt With No Recovery
- ◫ - Sample Recovered But Not Retained
- ▽ - Groundwater First Encountered
- ▼ - Groundwater Level at End of Exploration
- ⚲ - Seepage Observed

	Shear Strength, psf	Confining Pressure, psf	
Tx	320	(2600)	- Unconsolidated Undrained Traixial
TxCU	320	(2600)	- Consolidated Undrained Triaxial
DS	2750	(2600)	- Consolidated Drained Direct Shear
UC	2000		- Unconfined Compression
FVS	470		- Field Vane Shear
LVS	700		- Laboratory Vane Shear
SS			- Shrink Swell
EXP			- Expansion
P			- Permeability

Note: All strength tests on 2.8-in. or 2.4-in. diameter sample, unless otherwise indicated.

RGH CONSULTANTS

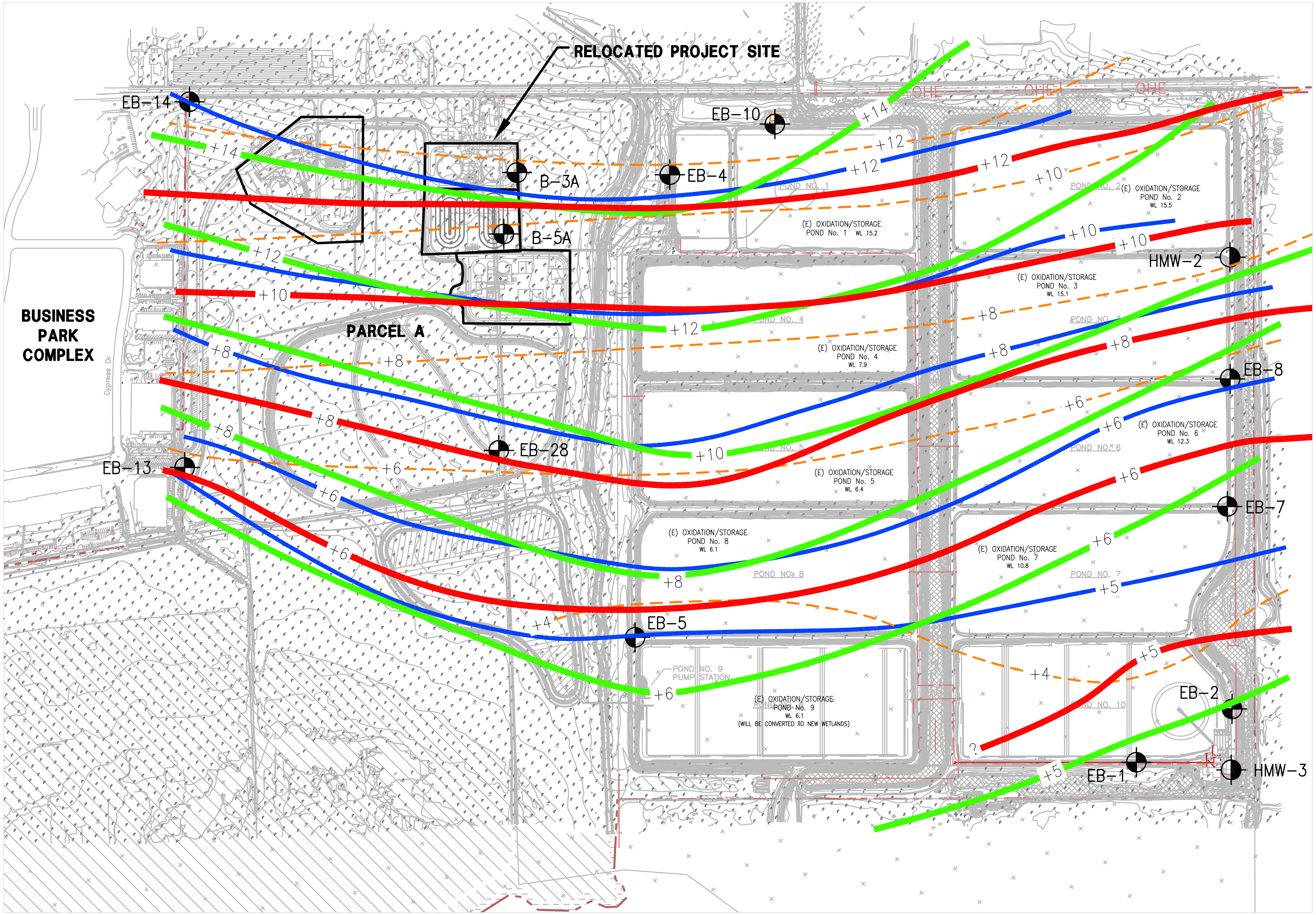
Job No: 2553.08.04.1 | Date: DEC 2012

SOIL CLASSIFICATION AND KEY TO TEST DATA

Ellis Creek Oxidation Ponds 7 and 10
Sheet Pile Levee Project
Petaluma, California

PLATE 8

April 2005
Project No. 3045.022



PIEZOMETER	MEASURED GROUNDWATER LEVEL ELEVATIONS				
	JULY 2001	JUNE 2002	JANUARY 2003	JULY 2003	MARCH 2004
EB-1	3.8	4.7	4.9	4.6	0.9
EB-2	3.5	4.8	5.0	4.8	4.6
EB-4	11.6	12.4	14.7	12.4	12.9
EB-5	3.7	4.9	6.9	4.7	5.4
EB-7	4.4	5.3	5.8	4.4	5.6
EB-8	5.1	6.2	6.6	5.9	6.4
EB-10	12.5	12.7	14.4	12.1	13.4
EB-13	5.8	5.0	6.4	5.9	5.8
EB-14	12.5	12.2	14.7	12.6	14.2
HMW-2	8.1	9.4	10.4	8.9	9.2
HMW-3	6.0	6.2	6.6	N/A	6.3
EB-28	–	–	9.2	6.4	8.0
B-3A	–	–	–	10.2	12.3
B-5A	–	–	–	8.9	10.7

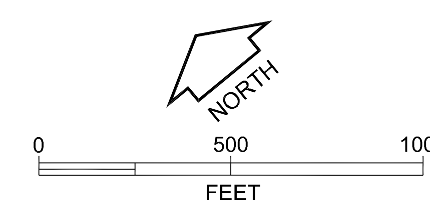
LEGEND

+12 ESTIMATED PIEZOMETRIC LINE AT ELEVATION +12 MEASURED IN MARCH, 2004

+12 ESTIMATED PIEZOMETRIC LINE AT ELEVATION +12 MEASURED IN JULY, 2003

+14 ESTIMATED PIEZOMETRIC LINE AT ELEVATION +14 MEASURED IN JANUARY, 2003

+12 ESTIMATED PIEZOMETRIC LINE AT ELEVATION +12 MEASURED IN JULY, 2001



PIEZOMETRIC CONTOUR LINES
Lakeville Highway WRF, Parcel A
Petaluma, California

G:\jobdocs\3045\3045.022\drawings\B1679.001-08.dwg 5-02-05 11:55:05 AM jbreyes

BASE MAP SOURCE: This Site Plan provided by Corollo Engineers, titled "00C200.dwg", dated 3/11/04.

PLATE 8

APPENDIX A
FIELD EXPLORATION – OXIDATION PONDS

The field investigation around the Oxidation Ponds consisted of a surface reconnaissance and a subsurface exploration program using a drill rig equipped with a truck-mounted, continuous flight, hollow stem auger and a truck mounted cone penetration test (CPT) rig. Twenty-six exploratory borings of 8-inch diameter were drilled on April 2 through 6, 2001, June 18 through 19, 2001, and May 2 through 3, 2002, to a maximum depth of about 70 feet. Ten CPTs were advanced between April 16 and 17, 2001, to a maximum depth of about 80 feet. Location of our Borings EB-1 through EB-19 was re-surveyed after the completion of our investigation. Location of the remaining borings and CPTs, including the previous borings by others were located approximately on the Site Plan, Figure 2. The soils encountered in the borings were continuously logged in the field by our representative. The soils are described in accordance with the Unified Soil Classification System (ASTM D-2487.) The logs of the borings, a key for the classification of the soil (Figure A-1), and a key for the CPTs (Figure A-2) are included as part of this appendix.

Representative samples were obtained from the exploratory borings at selected depths appropriate to the investigation. Undisturbed samples were obtained using a 3-inch O.D. Modified California sampler and disturbed samples were obtained using the 2-inch O.D. split-spoon sampler. All samples were transmitted to our laboratory for evaluation and appropriate testing. Both sampler types are indicated in the "Sampler" column of the boring logs as designated in Figure A-1.

Resistance blow counts were obtained with the samplers by dropping a 140-pound hammer through a 30-inch free fall. The sampler was 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, or the number of inches indicated where hard resistance was encountered. When the split-spoon sampler was used, these blow counts are the standard penetration resistance 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. In order to convert these values to approximate standard penetration resistance values, the indicated blow counts should be multiplied by a factor of about 0.65.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

Table A-1. Summary of Previous Boring Logs by Others

Boring	Ground Elevation (feet)	Depth (feet)	Material
MT71-3	+ 14	0-5 5-12	Soft, Silty CLAY Stiff Sandy CLAY
MT71-10	+10.2	0-4 4-9 9-13.5 13.5-23 23-27	Stiff, Silty CLAY Soft Silty CLAY Soft , Sandy CLAY Very loose Clayey to Silty SAND Dense gravelly SAND
MT71-24	+18.3	0-3 3-10 10-17	Stiff Silty CLAY Stiff Sandy CLAY Very stiff Sandy CLAY
MT71-25	+17	0-4 4-7 7-13 13-20	Stiff Silty CLAY Soft Clayey SILT Soft Silty CLAY Stiff Sandy CLAY
HB-1	+20	0-13 13-18 18-23 23-28.5 28.5-31.5	Stiff lean CLAY Medium Stiff fat CLAY Loose clayey SAND Very Stiff fat CLAY Stiff sandy fat CLAY
HB-2	+20	0-16 16-28 28-34.5 34.5-41.5	Stiff to Very Stiff lean CLAY Very Stiff sandy fat CLAY Medium Dense clayey SAND Very Stiff sandy fat CLAY
HB-3	+17	0-17 7-29 29-30	Medium Stiff to Stiff lean CLAY Very Stiff sandy lean CLAY Poorly-Graded SAND
HB-4	+17	0-23.5 23.5-25 25-26	Medium Stiff to Very Stiff lean CLAY Medium Dense poorly-graded SAND Very Stiff lean CLAY
HB-5	+19	0-13 13-26.5	Stiff sandy lean CLAY Stiff sandy fat CLAY
HB-6	+19	0-16 16-41.5	Stiff to Very Stiff sandy lean CLAY Medium Dense clayey SAND
HB-7	+14	0-14 14-24 24-29 29-31.5	Stiff to Very Stiff sandy CLAY Soft to Medium Stiff fat CLAY Medium Stiff to Stiff sandy lean CLAY Very Stiff sandy fat CLAY
HB-8	+11.5	0-9.5 9.5-18.5 18.5-20 20-23 23-25 25-27 27-49 49-50	Medium Stiff to Stiff lean CLAY Medium Stiff fat CLAY Very Stiff sandy CLAY Dense clayey SAND Medium Stiff lean CLAY Loose to Medium Dense clayey SAND Stiff to Very Stiff fat CLAY Clayey Gravel

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel And Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines
			SP	Poorly-graded sands or gravelly sands, little or no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, and-clay mixtures

Major Divisions		grf	ltr	Description
Fine Grained Soils	Silts And Clays LL < 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silt-clays of low plasticity
	Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat and other highly organic soils

GRAIN SIZES

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

	200	40	10	4	3/4"	3"	12"
Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.

**Unconfined compressive strength.

SYMBOLS

Standard Penetration sample

Modified California sample

Shelby Tube sample

Rock Core sample

Ground Water level during drilling

Stabilized Ground Water level

Increasing Visual Moisture Content

Dry
Damp
Moist
Wet
Saturated

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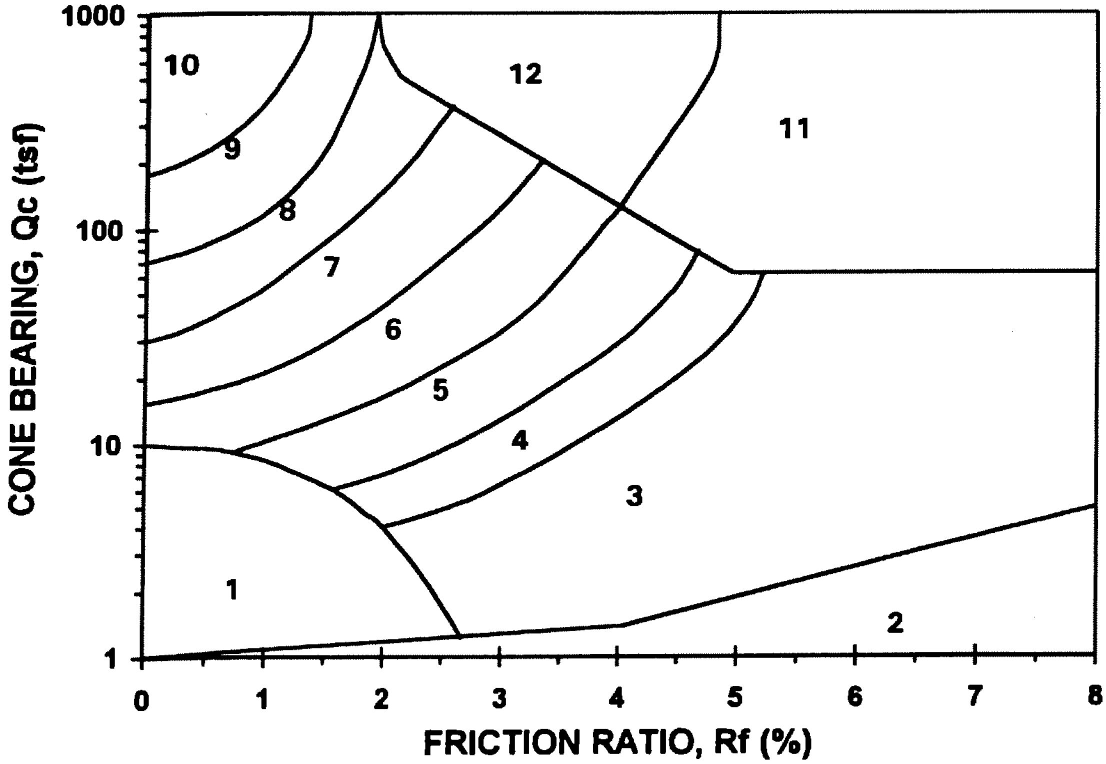
PREP'D BY: JND	KEY TO EXPLORATORY BORING LOGS	FIGURE A-1
APP'D BY: SR	LAKEVILLE HIGHWAY WRF PROJECT	
DATE: 9/17/02	Petaluma, California	PROJECT No.
DWG FILE: 19639-GI.GPJ		3045.006

SIMPLIFIED SOIL BEHAVIOR TYPE CLASSIFICATION FOR STANDARD ELECTRONIC CONE PENETROMETER



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for Qc < = 9 tsf)	Sensitive Fine Grained
2	1	15 (10 for Qc < = 9 tsf)	Organic Material
3	1	15 (10 for Qc < = 9 tsf)	CLAY
4	1.5	15	Silty CLAY to CLAY
5	2	15	Clayey SILT to Silty CLAY
6	2.5	15	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

Referances: [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

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DRAWN BY:	ROC
PREP'D BY:	JH
APP'D BY:	SR
SCALE:	NONE
DATE:	24JUL02
DWG FILE:	3045.006-A2

KEY TO CONE PENETRATION TEST

LAKEVILLE HIGHWAY WRF PROJECT
PETALUMA, CALIFORNIA

FIGURE
A-2

PROJECT No.
3045.006

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	10.3 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	23 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/6/01

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
EMBANKMENT FILL: CLAY (CL), green gray, with silt, moist	Stiff								
			5	X	21	23	100	2.4	
(silty, trace fine-grained sand)			10	X	19				
BAY MUD: CLAY (CH/MH), dark green gray, silty, trace organics, wet to saturated	Firm								
			15	X	9	81	52	0.9	
CLAY (CL/CH), blue green, silty, moist	Very Stiff		20	X	34	30	93	1.8	
						∇			
CLAY (CL), blue gray, with sand lenses, trace gravel (fine, subangular to subrounded), moist	Stiff								

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-1

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	10.3 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	23 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/6/01

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
CLAY (CL), continued	Stiff				15				
SAND (SW), brown, fine- to coarse-grained, trace gravel (fine, subangular), trace clay, saturated	Very Dense		30		80/9"				
CLAY (CL), light yellow brown, silty, trace sand (fine-grained), moist	Very Stiff								
			35		28				
(yellow brown, with sand inclusions)			40		42	25	101		
			45						
					33				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-1

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	10.3 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	23 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/6/01

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), continued	Very Stiff								
			55						
	Hard				60				
			60						
(some fine-grained sand at 64 feet)					36				
			65						

Bottom of Boring = 65 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. A piezometer was installed upon completion of drilling (See Figure A-2 for typical piezometer detail).
5. The initial groundwater level was encountered at 23 feet at the time of drilling. The stabilized groundwater level was monitored in the piezometer in the following months after drilling was completed.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-1

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**10.3 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**3 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/5/01**

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
PAVEMENT: 2 inches AC over 8 inches AB									
EMBANKMENT FILL: CLAY (CL), dark olive gray, with silt, trace sand (fine-grained), damp to moist	Stiff					▼			
			5		19				
(dark gray, moist)					17	30	92	1.7	
			10						
BAY MUD: CLAY (CH/MH), dark olive gray, silty, trace organics, saturated	Soft								
					6	109 95	43 47		Tx = 0.45 (1.0) Tx = 0.40 (2.0)
			15						
					Push	82 94	52 47		Tx = 0.45 (1.5) Tx = 0.45 (1.0)
			20		4	65	60	0.8	
SAND (SC), gray brown, fine- to coarse-grained, trace gravel (fine,	Dense					▽			

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-2**

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**10.3 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**3 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/5/01**

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
subangular), trace to some clay, saturated	Dense				60				
CLAY (CL), dark blue green, silty, some sand (fine-grained), moist	Firm		30		6				
			35		33				
SAND (SW), gray brown, fine- to coarse-grained, trace silt and clay, saturated	Medium Dense								
SAND (SC), brown, fine-grained, with clay, wet	Medium Dense				27				
			40						
CLAY (CL), yellow brown, silty, trace sand (fine-grained), moist	Very Stiff		45						
					31				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-2**

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	10.3 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	3 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/5/01

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)						
CLAY (CL), continued	Very Stiff								
			55						
				X	52				
			60						
	Hard				38				
			65						

Bottom of Boring = 65 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. A piezometer was installed upon completion of drilling (See Figure A-2 for typical piezometer detail).
5. The initial groundwater level was encountered at 24 feet at the time of drilling. The stabilized groundwater level was monitored in the piezometer in the following months after drilling was completed.
6. Tx = undrained shear strength from UU triaxial test (ksf) and confinement pressure (ksf).

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	12.6 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	24.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/3/01

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
EMBANKMENT FILL: CLAY(CL), dark gray, silty, trace sand (fine-grained), moist	Stiff								
			5		23				
BAY MUD: CLAY (CH), dark gray, silty, trace organics, saturated	Firm		10		14	32	89	1.1	
	Soft		15		4	87	46	0.8	
CLAY (CL), green gray, silty, trace pockets of calcium carbonate, moist	Stiff								
			20		16	25	100	2.6	
(yellow brown, no calcium carbonate, damp to moist)					12	∇			

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-3

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	12.6 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	24.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/3/01

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), continued	Stiff								
SILT (ML), brown, sandy (fine- to medium grained), trace gravel (fine), moist	Stiff								
			30		12				
(silty sand to sandy silt)			35		15				Passing #200 Sieve = 40%
			40		11				
	Very Stiff		45		45				

Bottom of Boring = 45 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. The initial groundwater level was encountered at 23½ feet at the time of drilling.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-3**

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**19.6 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**14 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/4/01**

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
EMBANKMENT FILL: CLAY (CL), dark blue gray, silty, trace to some sand (fine-grained), moist	Firm								
			5		13				
	Very Stiff		10		37				
GRAVEL (GC), gray brown, fine, subrounded, sandy (fine-grained), some clay, wet	Medium Dense		15		14	▽			
									Passing #200 Sieve = 24%
			20		38				
SAND (SW), brown, fine-grained, trace silt, saturated	Very Dense				56				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-4**

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	19.6 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	14 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/4/01

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
SAND (SW), continued	Very Dense								
SAND (SC), brown, fine- to coarse-grained, clayey, trace gravel (fine, subangular to subrounded), wet to saturated	Dense		30		73				
SILT (ML), yellow brown, clayey, sandy (fine-grained), moist	Very Stiff		35		27				
CLAY (CL), yellow brown, silty, trace sand (fine- to coarse-grained), damp to moist	Very Stiff		40		39				

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. A piezometer was installed upon completion of drilling (See Figure A-2 for typical piezometer detail).
5. The initial groundwater level was encountered at 14 feet at the time of drilling. The stabilized groundwater level was monitored in the piezometer in the following months after drilling was completed.

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	13.8 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	8 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/4/01

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
EMBANKMENT FILL: CLAY(CL), dark gray-brown, silty, trace sand (fine-grained), moist	Firm								
			5		10				
						▽			
			10		10	31	92	0.5	
CLAY (CH), dark gray black, silty, wet to saturated	Firm				4				
	Stiff		15		15	39	84	1.1	
(dark gray, moist to wet)			20		18				
CLAY (CL), yellow brown, silty, moist	Very Stiff				25				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-5

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	13.8 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	8 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/4/01

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)						
CLAY (CL), continued									
SAND (SC/SP), yellow brown, fine- to coarse-grained, trace gravel (fine, subangular to subrounded), trace clay, wet to saturated	Medium Dense				50				
			30						Passing #200 Sieve = 11%
					23				
CLAY (CL), yellow brown, silty, trace sand (fine-grained), damp to moist	Very Stiff		35						
					28				
			40						

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. A piezometer was installed upon completion of drilling (See Figure A-2 for typical piezometer detail).
5. The initial groundwater level was encountered at 8 feet at the time of drilling. The stabilized groundwater level was monitored in the piezometer in the following months after drilling was completed.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-5

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**15.8 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**7.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/3/01**

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
EMBANKMENT FILL: CLAY(CL), dark gray, silty, trace sand (fine-grained), damp	Very Stiff								
(blue gray, with fine-grained sand)			5		34				
SAND (SP/SC), dark blue gray, fine-grained, trace clay, saturated	Loose					▽			
			10		14				
					10				
CLAY (CH), dark gray, silty, wet	Firm								
			15		9	48	73	0.7	PI = 47, LL = 64, Passing #200 Sieve = 92%
CLAY (CL), yellow brown, sandy (fine-grained), with silt, moist	Stiff								
			20		22				
					23				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-6**

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**15.8 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**7.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/3/01**

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
CLAY (CL), continued	Stiff								
(trace fine-grained sand)			30		9				
(trace to some fine-grained sand, trace fine, subangular to subrounded gravel)			35		14				
GRAVEL (GC), brown, fine, subangular to subrounded, with sand (fine- to medium-grained), some clay, moist to wet	Medium Dense				19				
CLAY (CL), brown, silty, trace sand (fine-grained), moist	Very Stiff		40						

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. The initial groundwater level was encountered at 8.5 feet at the time of drilling.
5. PI = Plasticity Index; LL = Liquid Limit

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**14.2 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**21.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/5/01**

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
PAVEMENT: 2 inches AC over 6 inches AB									
EMBANKMENT FILL: CLAY (CL), dark gray green, silty, trace sand (fine-grained), moist	Stiff				21				
	Very Stiff		5		33				
			10		13	22	104	1.6	
BAY MUD: CLAY (CH), dark olive gray, with silt, trace to some organics, wet to saturated	Firm								
			15		11	99	42	0.9	
CLAY (CL/CH), blue gray, silty, trace gravel (fine, subangular to subrounded), damp to moist	Very Stiff								
			20		38	∇			
GRAVEL (GC), gray brown, fine, subangular to subrounded, sandy (fine- to coarse-grained), some clay	Medium Dense								
CLAY (CL), yellow brown, silty, some sand (fine-grained), moist	Very Stiff								

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-7**

File Name: G:\ENGINEER\GINT\WPROJECTS\19639-GI.GPJ Report Template: FUGRO 440 Output Date: 9/17/02

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	14.2 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	21.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/5/01

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
CLAY (CL), continued	Very Stiff				42	17	116		Tx = 2.0 (1.5)
(trace fine-grained sand)	Stiff		30		9				
(fine to medium-grained sandy, trace fine, subangular to subrounded gravel, moist to wet)	Very Stiff		35		35				
(some fine-grained sand, no gravel, moist)					43				
			40						

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. A piezometer was installed upon completion of drilling (See Figure A-2 for typical piezometer detail).
5. The initial groundwater level was encountered at 21½ feet at the time of drilling. The stabilized groundwater level was monitored in the piezometer in the following months after drilling was completed.
6. Tx = undrained shear strength from UU triaxial test (ksf) and confinement pressure (ksf).

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	16.3 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	23 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/5/01

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
PAVEMENT, 2 inches AC over 8 inches AB									
EMBANKMENT FILL: CLAY(CL), dark gray, with silt, damp to moist	Very Stiff								
			5	X	32				
	Stiff		10	X	27	26	98	3.2	
CLAY (CL/CH), dark gray black, silty, trace organics, wet to saturated	Firm								
			15	X	9	34	87	0.8	
(light blue-green, with silt, moist)	Stiff		20	X	26	30	94	2.3	
					∇				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-8

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**16.3 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**23 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/5/01**

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
CLAY (CL), yellow brown, silty, trace sand (fine-grained), damp to moist	Very Stiff				33				
(some sand)	Hard		30		55				
			35		26				
SAND (SC), brown, fine-grained, some gravel (fine, subangular to subrounded), some clay, wet	Dense								Passing #200 Sieve = 22%
					33				
CLAY (CL), yellow brown, silty, trace sand (fine-grained), damp	Very Stiff		40						

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. A piezometer was installed upon completion of drilling (See Figure A-2 for typical piezometer detail).
5. The initial groundwater level was encountered at 23 feet at the time of drilling. The stabilized groundwater level was monitored in the piezometer in the following months after drilling was completed.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-8**

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**19.8 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**8 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/2/01**

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
EMBANKMENT FILL: CLAY (CL), dark gray, silty, trace sand (fine-grained), damp to moist	Stiff								
			5		11				
						▽			
(dark gray blue, no sand, wet to saturated)	Firm				11	30	93	1.7	
			10						
					26	24	101	1.4	
SAND (SC), dark blue green, fine- to coarse-grained, some gravel (fine, subrounded to rounded), some clay, damp to moist	Medium Dense		15						
CLAY (CL), mottled blue green and yellow brown, silty, trace sand (fine-grained), damp to moist	Stiff								
			20		11				
(yellow brown, some sand (fine-grained),	Firm				10	24	101	1.4	

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-9**

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	19.8 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	8 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/2/01

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)						
moist at 24 feet) **CLAY (CL)**, continued	Firm								
SAND (SC), brown, fine- to coarse-grained, trace clay and silt, saturated	Loose		30		13				Passing #200 Sieve = 12%
GRAVEL (GW/GC), brown, fine to coarse, subangular to subrounded, with sand (fine-grained), trace silt and clay, saturated	Dense		35		41				
CLAY (CL), brown, silty, with sand (fine-grained), moist	Stiff		40		14				

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. The initial groundwater level was encountered at 8 feet at the time of drilling.

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**19.9 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**18 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/4/01**

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
PAVEMENT, 2 inches AC over 8 inches AB									
EMBANKMENT FILL: CLAY (CL), dark blue-gray, silty, damp	Stiff								
			5		21	20	105	4.9	PI = 30, LL = 43, Passing #200 Sieve = 74%
(trace fine-grained sand, trace fine and subrounded gravel, damp to moist)			10		10				
CLAY (CL), yellow brown, silty, trace sand (fine-grained), damp to moist	Very Stiff		15		29	18	113	6.6	
						▽			
			20		35				
SAND (SC), yellow brown, fine- to medium-grained, with clay, saturated	Very Dense				60				

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**19.9 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**18 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/4/01**

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
SAND (SC), continued	Very Dense								
CLAY (CL), yellow brown, silty, trace sand (fine-grained), damp to moist	Very Stiff		30		53				
			35						
(with sand)	Hard		40		56				

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. A piezometer was installed upon completion of drilling (See Figure A-2 for typical piezometer detail).
5. The initial groundwater level was encountered at 8 feet during drilling. The stabilized groundwater level was monitored in the piezometer in the following months after drilling was completed.
6. PI = Plasticity Index; LL = Liquid Limit

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-10**

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**19.6 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**18.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**4/2/01**

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
EMBANKMENT FILL: CLAY (CL), dark brown, silty, trace sand (fine- to medium-grained), damp to moist	Stiff								
(green brown)			5		17				
(dark blue gray, increasing plasticity)			10		24	23	102	2.2	
CLAY (CL), yellow brown, sandy (fine-grained), damp to moist	Very Stiff		15		37	19	109	5.9	
	Stiff		20		12	∇			
					9				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-11**

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	19.6 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	18.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	4/2/01

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), continued	Stiff								
(fine-grained sandy)	Firm				12	23	103	0.4	
CLAY (CL/CH), yellow brown, with silt, damp to moist	Very Stiff		30						
					23				
			35						
					39				
			40						

Bottom of Boring = 40 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. The initial groundwater level was encountered at 18½ feet at the time of drilling.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-11

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	16.2 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	4/3/01

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
CLAY (CL), dark blue-gray, silty, trace sand (fine-grained), damp to moist	Stiff								R-Value = 13
					21				
(dark gray, trace fine, subangular to subrounded gravel at 2½ feet)					13				
(dark blue-green at 4½ feet)			5		14				

Bottom of Boring = 5½ 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. The groundwater level was not encountered at the time of drilling.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-12

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	19.9 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	6/18/01

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
CLAY (CL), dark gray, silty, trace sand (fine-grained), moist	Firm								
			5						
				X	11				
			10						
	Stiff			X	22				
				X	20				
			15						
			20						

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	19.9 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	6/18/01

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
GRAVEL (GW/GC), gray, fine to coarse, subangular to subrounded, sandy (fine- to coarse-grained), trace clay, saturated	Dense				41				
CLAY (CL), brown, some silt, sandy (fined-grained), moist (1" SP lens)	Firm		30		13	25			
(silty, some sand)	Stiff		35		21				

Bottom of Boring = 35 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. No groundwater level was encountered at the time of drilling.
5. Sampling at 9 to 13 feet for environmental investigating purposes.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-17

DRILL RIG	**Mobile B-53, HSA**	SURFACE ELEVATION	**15.4 Feet**	LOGGED BY	**JND**
DEPTH TO GROUND WATER	**Not Encountered**	BORING DIAMETER	**8-inch**	DATE DRILLED	**6/18/01**

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
CLAY (CL), dark gray, silty, moist	Stiff								
					11				
(1"-2" thick lenses of SC/CL with sand from 4-½ to 9 feet)			5						
	Firm				8				
					7				
(no sandy lenses)			10		6				
					6				
			15						

Bottom of Boring = 15 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. No groundwater level was encountered at the time of drilling.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**June 2001**	**EB-18**

DRILL RIG	Mobile B-53, HSA	SURFACE ELEVATION	10.8 Feet	LOGGED BY	JND
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	6/18/01

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
EMBANKMENT FILL: CLAY (CL), dark gray, silty, trace to some sand (fine-grained), damp to moist	Stiff								
			5						
			10	X	20				
	Very Stiff								
BAYMUD: CLAY (CH/MH)	Very Stiff			X	30				

Bottom of Boring = 12 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 the first page of Appendix A.
3. A 140-lb wire trip hammer falling 30 inches was used to drive the samplers.
4. No groundwater level was encountered at the time of drilling.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	June 2001	EB-19

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	16.8 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	14 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/3/02

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
EMBANKMENT FILL: CLAY (CL), dark brown, silty, some sand (fine- grained), moist	Firm								
grading blue-gray, sandy (fine- grained), trace silt at 3½ feet			5		14				
	Stiff		10		17	29	91		
SAND (SC), gray, fine- to coarse-grained, some clay, wet	Loose				9	▼ 35			Passing #200 Sieve = 20%
CLAY (CL), dark brown, silty, trace sand (fine- to coarse- grained), damp	Firm to Stiff		15						Passing #200 Sieve = 86%, LL=46, PI=32
			20		8	28			
grading sandy at 23½ feet	Stiff				23	22	102		

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**May 2002**	**EB-20**

File Name: G:\ENGINEER\GINTW\PROJECTS\3045_006.GPJ Report Template: FUGRO 440 Output Date: 9/17/02

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	16.8 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	14 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/3/02

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
CLAY (CL), gray-brown, mottled with blue, trace sand (fine- to coarse- grained), trace gravel (fine to coarse, subrounded to subangular), moist	Stiff								
			30		11				
CLAY (CL), light gray, silty, trace sand (fine- grained), some wood pieces (up to 1"), moist	Stiff				15	36	86		
			35						
grading sandy silt at 38½ feet	Hard				60	29	95		
CLAY (CL), brown, with sand (fine- to coarse- grained), damp	Very Stiff		40						
					38	19	112	4.2	
			45						
CLAY (CL), light brown, silty, trace sand	Very Stiff				41	20	110		

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-20

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**16.8 Feet**	LOGGED BY	**JCH**
DEPTH TO GROUND WATER	**14 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**5/3/02**

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)						
(fine- to coarse- grained), damp									
Sand (SC), brown, fine- to coarse-grained, gravelly (fine, subrounded to subangular), trace clay, wet			55						Passing #200 Sieve = 12%, #4 Sieve = 75%
CLAY (CL), light gray, with sand (fine- to coarse- grained), silty, moist	Dense				40	16			
			60						

Bottom of Boring = 60 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with neat cement grout immediately upon completion.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**May 2002**	**EB-20**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	19.0 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	15 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/2/02

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)						
EMBANKMENT FILL: CLAY (CL), brown, silty, some sand (fine- to coarse-grained), damp	Firm								
grading trace sand at 3½ feet					13				
			5						
grading dark brown, trace sand at 7 feet									
grading with sand at 9½ feet					14	23	100		
			10						
SAND (SC), brown, fine- to coarse-grained, clayey, moist	Loose				13				
			15			▼			Passing #200 Sieve = 34%
CLAY (CL), light brown, silty, trace sand (fine- grained), trace gravel (coarse, angular), damp	Stiff				9				
			20						
grading sandy at 23½ feet					19	25			

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-21

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	19.0 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	15 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/2/02

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 (SC), brown, fine- to coarse-grained, clayey, moist	Medium Dense								Passing #200 Sieve = 34%, LL=32, PI=18
CLAY (CL), gray-brown, silty, trace sand (fine- grained), damp	Firm to Stiff								
grading gray brown, some silt at 29 feet					8	27			
			30						
Grading blue-gray, pieces of wood (up to 1"), wet at 34 feet	Very Stiff				43				
			35						
grading brown, some clay, coarse- grained at 39 feet					11				
			40						
					22	23			
			45						
SAND (SP), brown, fine- to coarse-	Dense				73				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-21

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**19.0 Feet**	LOGGED BY	**JCH**
DEPTH TO GROUND WATER	**15 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**5/2/02**

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)						
grained, some gravel (fine, subrounded to subangular), wet									Passing #200 Sieve = 4%, #4 Sieve = 84%
CLAY (CL), brown, silty, trace sand (fine- grained), moist	Very Stiff		55		19				
grading with sand at 59 feet	Hard		60		80	17	114	3.0	

Bottom of Boring = 60 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with neat cement grout immediately upon completion.

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EXPLORATORY BORING LOG		
LAKEVILLE HIGHWAY WRF PROJECT **Petaluma, California**		
PROJECT NO.	DATE	BORING NO.
3045.006	**May 2002**	**EB-21**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	19.5 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	14 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/2/02

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
EMBANKMENT FILL: CLAY (CL), dark brown, silty, with sand (fine- grained), damp	Firm								
			5		12				
grading dark gray-brown, moist			10		8	27	92		
					8	▼			
CLAY (CL), light gray, silty, trace sand (fine- to coarse- grained), some gravel (coarse, angular), moist	Firm		15						
	Stiff		20		18	23	99	2.3	
grading light brown, some sand (fine- to coarse- grained), moist at 24½ feet					20	23	98		

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-22

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	19.5 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	14 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/2/02

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
SILT (ML), light gray-brown, clayey, trace sand, moist	Stiff								
			30		12	27			
SAND (SC), light brown, fine- to coarse-grained, clayey, wet	Medium Dense				11				Passing #200 Sieve = 32%
			35						
CLAY (CL), light brown, silty, some sand (fine- to coarse- grained), damp	Very Stiff				19	23			
			40						
					34	19	109		
			45						
grading sandy at 47½ feet									
					34				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-22

File Name: G:\ENGINEER\GINTW\PROJECTS\3045_006.GPJ Report Template: FUGRO 440 Output Date: 9/17/02

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**19.5 Feet**	LOGGED BY	**JCH**
DEPTH TO GROUND WATER	**14 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**5/2/02**

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
grading trace sand (fine- grained), trace silt at 53½ feet				X	38				
			55						
				X	40				
			60						

Bottom of Boring = 60 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with neat cement grout immediately upon completion.

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**24.6 Feet**	LOGGED BY	**JCH**
DEPTH TO GROUND WATER	**18.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**5/2/02**

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SLUDGE: CLAY (CL), gray, silty, moist									
inclusion of gravel (GP), light brown, coarse, angular, sandy (fine- to coarse-grained), damp from 4 to 6 feet	Medium Dense		5	X	24				
grading CL/CH, dark gray, sandy (fine- to coarse- grained), moist at 6 feet	Firm								
			10	X	11				
grading blue-gray, silty, trace sand (fine-grained), damp									
			15	X	11				Consolidation Test Performed at 14 feet
CLAY (CL), dark brown, silty, trace sand (fine- grained), moist	Stiff			X	16	21	107	2.7	
grading sandy at 20 feet			20						
SILT (ML), light brown, sandy (fine- to coarse- grained), some clay, wet	Stiff			X	15	29			

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**May 2002**	**EB-23**

File Name: G:\ENGINEER\GINTW\PROJECTS\3045_006.GPJ Report Template: FUGRO 440 Output Date: 9/17/02

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**24.6 Feet**	LOGGED BY	**JCH**
DEPTH TO GROUND WATER	**18.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**5/2/02**

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
			30		13	28			Passing #200 Sieve = 54%, LL=32, PI=19
SAND (SC), gray-brown, fine- to coarese-grained, trace clay, moist	Loose		35		8				Passing #200 Sieve = 36%
CLAY (CL), light brown, silty, trace sand (fine- grained), moist	Firm		40		9	21			
SAND (SC), brown, black specks, fine- to coarse- grained, trace clay, trace silt, damp	Medium Dense		45		44				
					23				Passing #200 Sieve = 48%

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**May 2002**	**EB-23**

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**24.6 Feet**	LOGGED BY	**JCH**
DEPTH TO GROUND WATER	**18.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**5/2/02**

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
grading clayey at 49½ feet	Medium Dense								
					27				
grading some gravel (fine to coarse, subangular) at 54½ feet			55						
CLAY (CL), light brown, silty, some sand (fine- to coarse- grained), wet	Very Stiff								
					31	26	110		
			60						
grading sandy at 61½ feet									
					23				
			65						Passing #200 Sieve = 51%
					46				
			70						

Bottom of Boring = 70 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with neat cement grout immediately upon completion.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**May 2002**	**EB-23**

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**16.2 Feet**	LOGGED BY	**JCH**
DEPTH TO GROUND WATER	**13.5 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**5/3/02**

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
CLAY (CH), dark gray-brown, silty, trace sand (fine- grained), moist	Firm								
			5	X	13	25			Passing #200 Sieve = 95%, LL=53, PI=32
grading dark gray at 9½ feet	Stiff		10	X	20				
grading trace sand (fine- grained) at 13 feet						▼			
			15	X	18	30			Passing #200 Sieve = 91%, LL=45, PI=30
SAND (SC), light gray, fine- to coarse-grained, some clay, wet	Medium Dense								Passing #200 Sieve = 24%, #4 Sieve = 100%
			20	X	29	20			Passing #200 Sieve = 41%, LL=29, PI=16
CLAY (CL), light brown, silty, trace to some sand, damp	Very Stiff				27				

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	**May 2002**	**EB-24**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	16.2 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	13.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/3/02

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
grading sandy at 28½ feet	Stiff				22	21	109	3.4	
			30						
SAND (SC), brown, fine- to coarse-grained, with clay, some gravel (fine to coarse, subrounded to subangular), wet	Medium Dense				40				Passing #200 Sieve = 34%
			35						
	Dense				32	18			
CLAY (CL), light brown, sandy (fine- to coarse- grained), moist	Very Stiff		40						
grading silty, some sand, some gravel (fine to coarse, subrounded to subangular) at 44 feet					19	23			
			45						
CLAY (CL), brown, silty, some sand (fine- grained), wet	Stiff				13				Passing #200 Sieve = 80%

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-24

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	16.2 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	13.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/3/02

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
grading light brown, no sand at 52 feet	Hard		55		36				
	Very Stiff		60		36	27	96	3.0	

Bottom of Boring = 60 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with neat cement grout immediately upon completion.

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EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-24

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	18.5 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	13.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/3/02

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
EMBANKMENT FILL: CLAY (CL), brown, silty, some sand (fine- grained), damp	Stiff								
			5		15				
					14	26			
CLAY (CH), brown, silty, trace sand (fine- grained), trace organics, moist	Stiff		10						
						▼			
lens of gravelly clay (coarse, subrounded to subangular) at 14½ feet	Very Stiff				34	21	100	1.6	
CLAY (CL), brown, mottled blue-gray, silty, some sand (fine- to coarse-grained), moist	Very Stiff		15		21				
					49	17	117		
SAND (SC), brown, mottled black, fine- to coarse- grained, clayey, moist	Medium Dense		20						
					25				Passing #200 Sieve = 34%

File Name: G:\ENGINEER\GINT\WPROJECTS\3045_006.GPJ Report Template: FUGRO 440 Output Date: 9/17/02

FUGRO WEST, INC.

1000 Broadway, Suite 200
Oakland, CA 94607

EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-25

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	18.5 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	13.5 feet	BORING DIAMETER	8-inch	DATE DRILLED	5/3/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), light brown, sandy (fine- to coarse- grained), moist	Very Stiff								
			30		17	17			
grading silty, some sand at 32½ feet									
			35		33	21	105	4.2	
			40		35				

Bottom of Boring = 40 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with neat cement grout immediately upon completion.

FUGRO

FUGRO WEST, INC.

1000 Broadway, Suite 200
Oakland, CA 94607

EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-25

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	12.9 Feet	LOGGED BY	JCH
DEPTH TO GROUND WATER	Not Encountered	BORING DIAMETER	8-inch	DATE DRILLED	5/2/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
CLAY (CL), brown, with sand (fine- to coarse- grained), trace silt, trace gravel (fine to coarse, subrounded to subangular), damp	Stiff to Very Stiff								
			5	X	27				
grading gravelly, with sand at 9 feet	Very Stiff		10	X	35	15	122	7.3	
				X	36	16	115		
SAND (SC), brown, fine- to coarse-grained, some clay, moist	Medium Dense		15						
grading clayey, some gravel at 17 feet									
			20	X	38	15	120		

Bottom of Boring = 20 feet

Notes:
1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of the penetration resistance values, see the first page of Appendix A.
3. A 140-lb, down-hole, wire-line, safety hammer falling 30 inches was used to advance the sampler.
4. The boring was backfilled with neat cement grout immediately upon completion.

FUGRO

FUGRO WEST, INC.

1000 Broadway, Suite 200
Oakland, CA 94607

EXPLORATORY BORING LOG

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

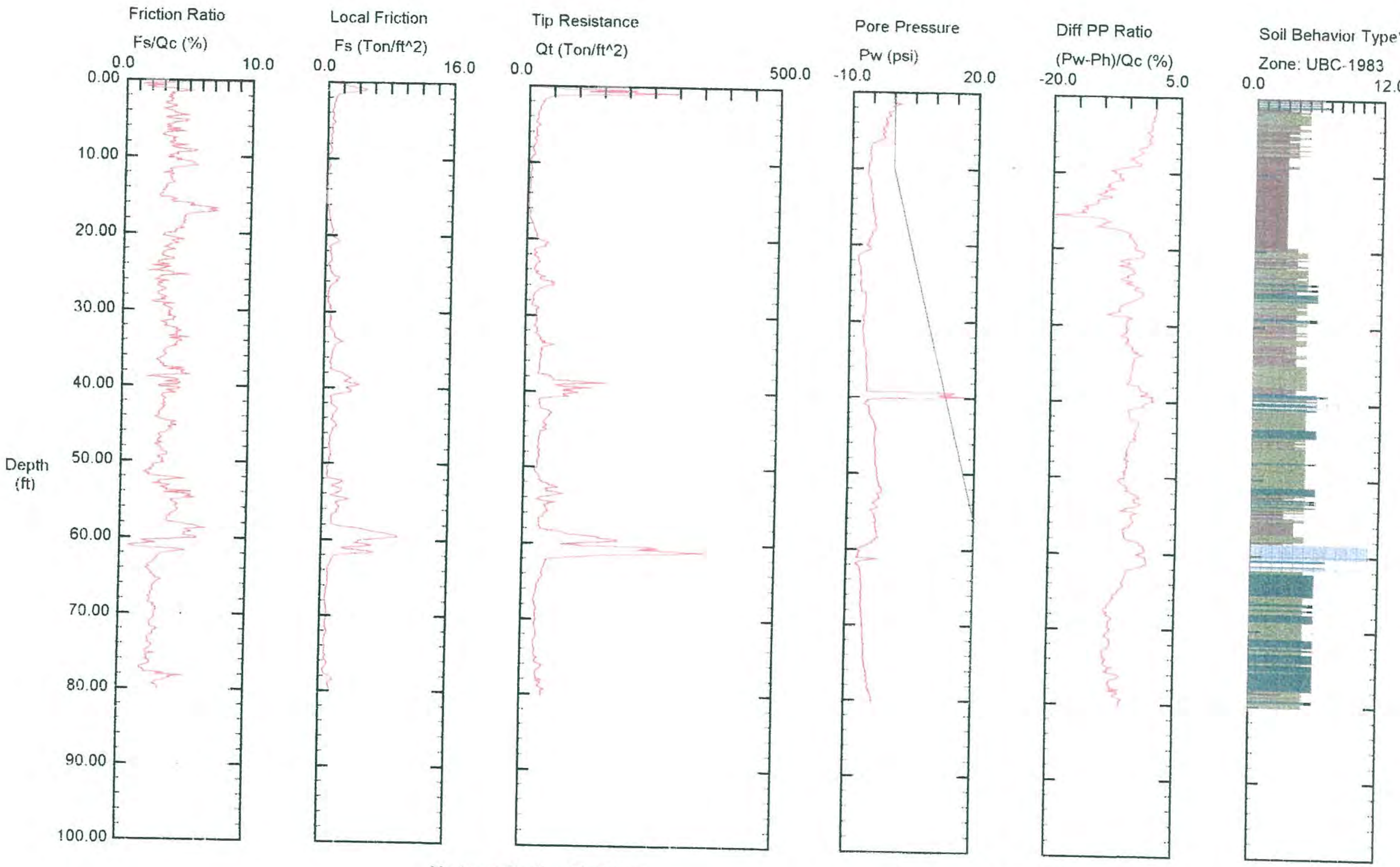
PROJECT NO.	DATE	BORING NO.
3045.006	May 2002	EB-26

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z079
Cone Used: HO752TC U2
Elevation: +19.0

CPT Date/Time: 04-17-01 16:41
Location: CPT-1
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

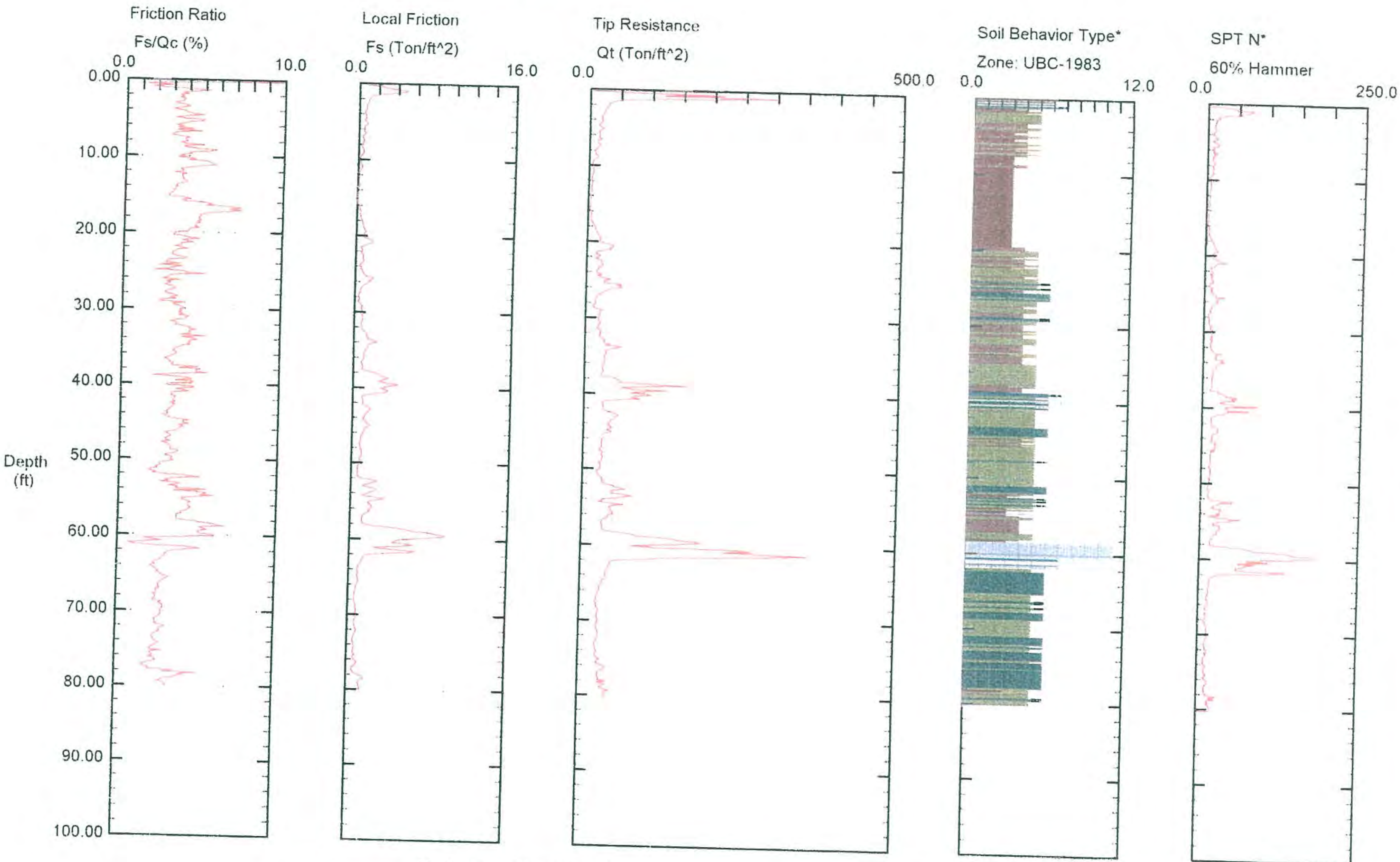
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z079
Cone Used: HO752TC U2
Elevation: +19.0

CPT Date/Time: 04-17-01 16:41
Location: CPT-1
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

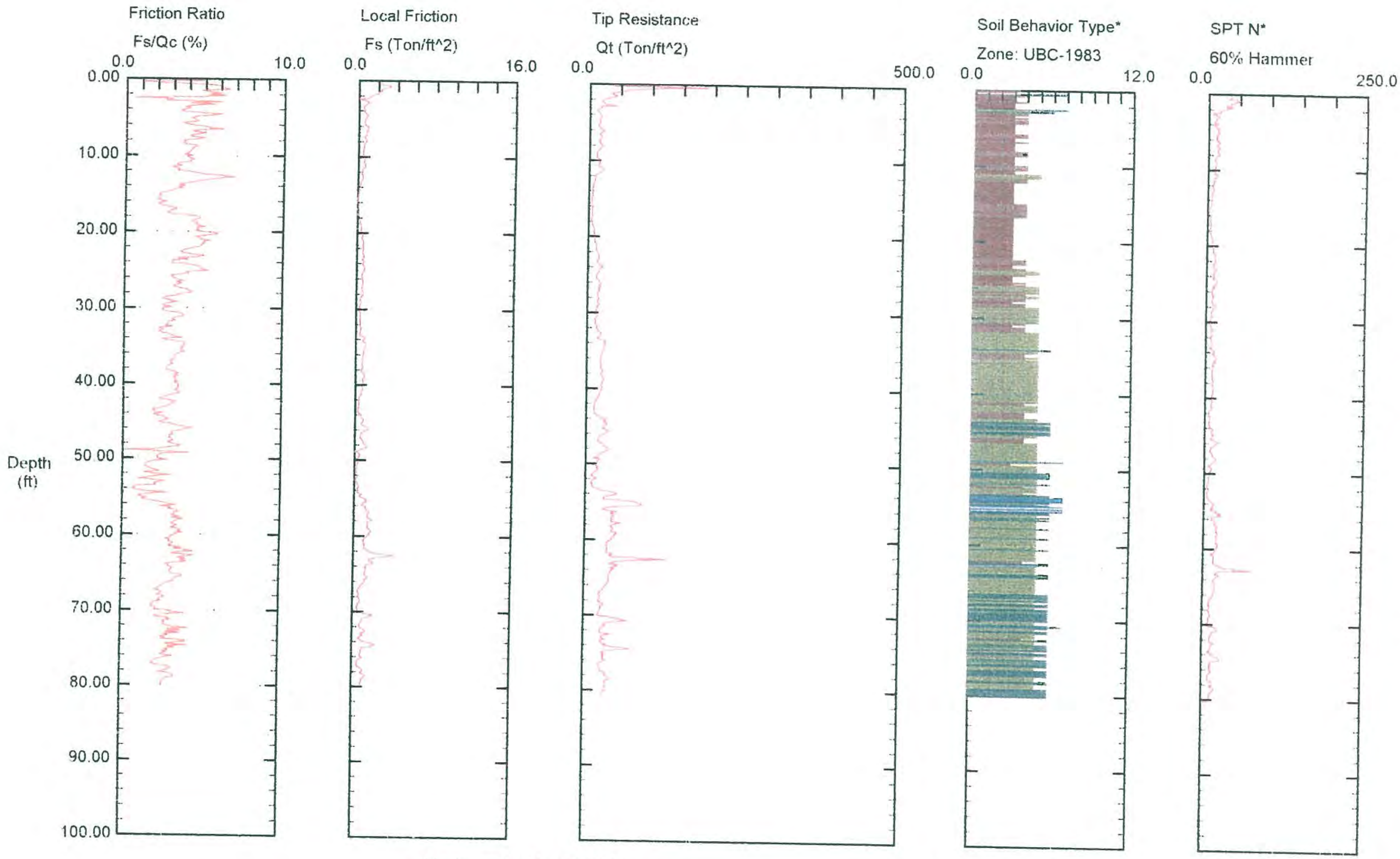
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z076
Cone Used: HO752TC U2
Elevation: +11.1

CPT Date/Time: 04-17-01 11:40
Location: CPT-2
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

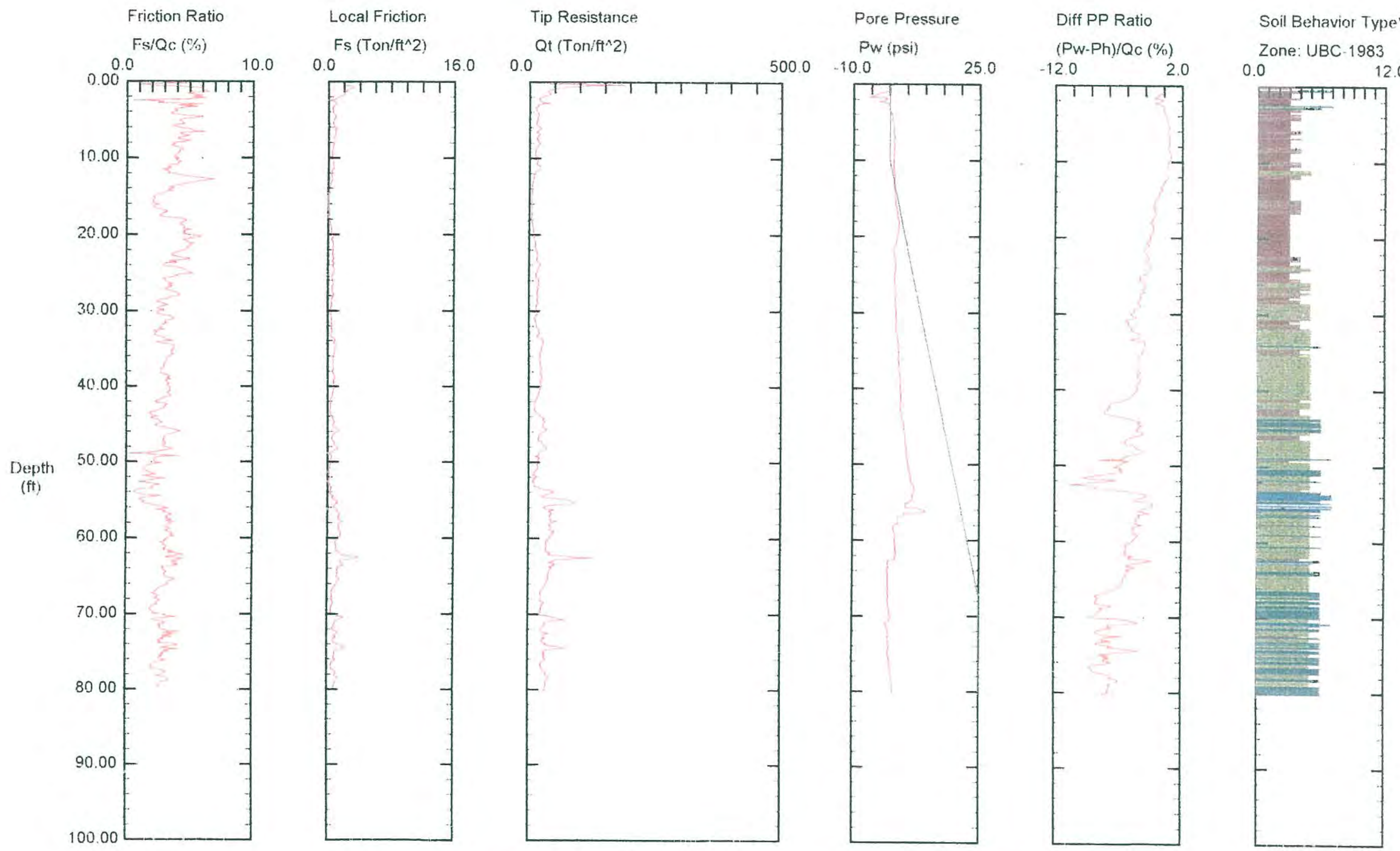
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z076
Cone Used: HO752TC U2
Elevation: +11.1

CPT Date/Time: 04-17-01 11:40
Location: CPT-2
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

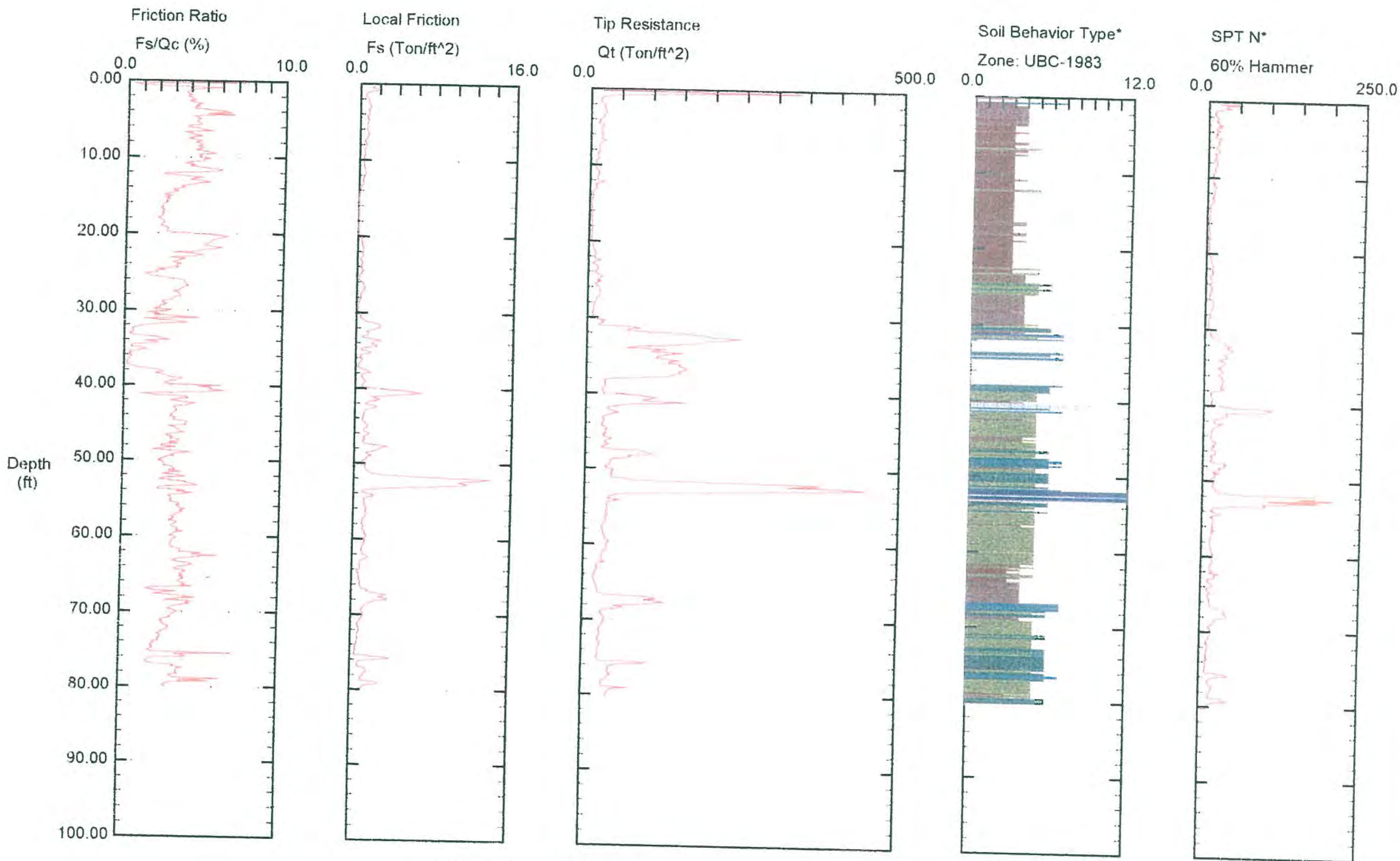
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z070
Cone Used: HO752TC U2
Elevation: +12.0

CPT Date/Time: 04-17-01 07:18
Location: CPT-3
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

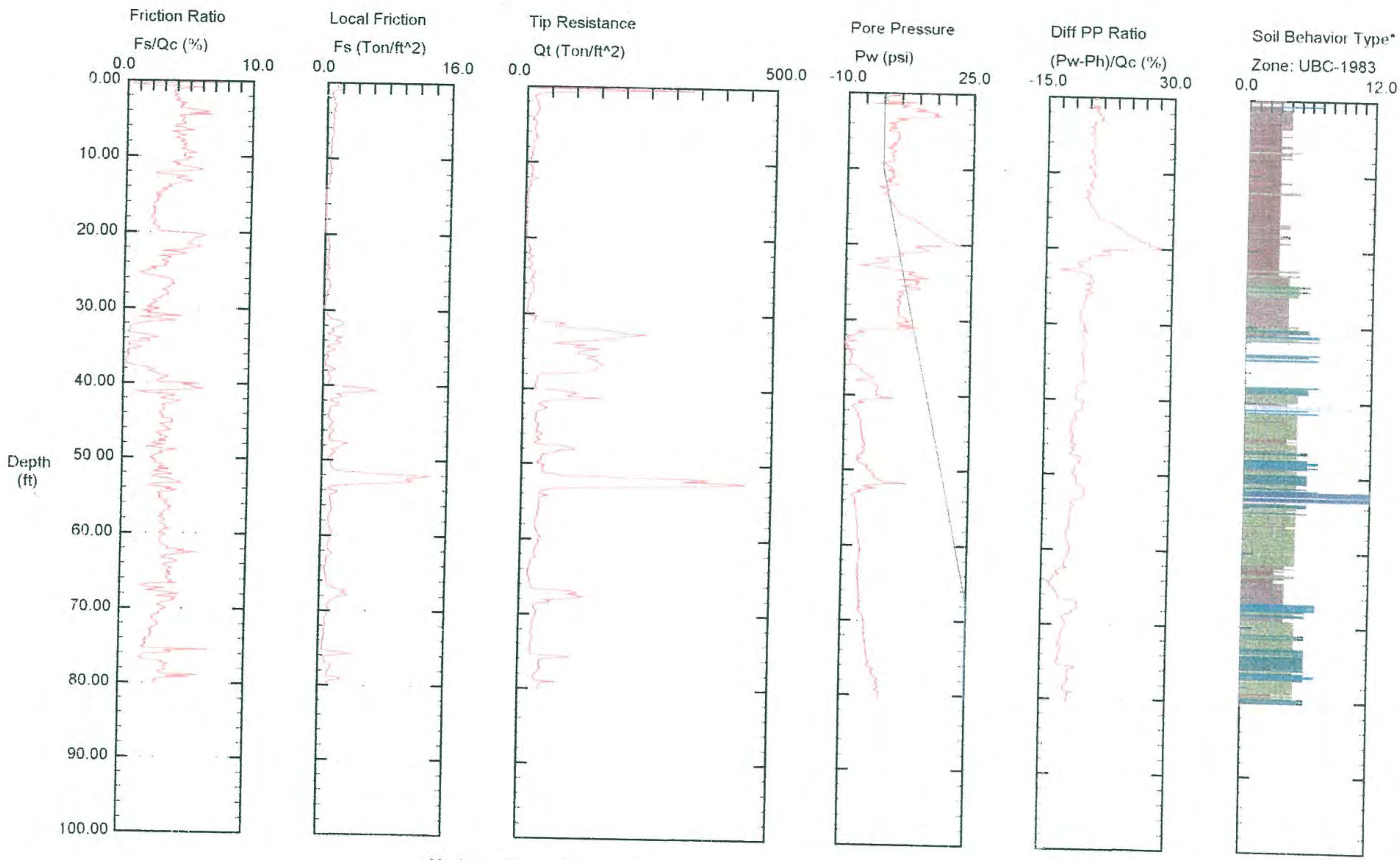
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z070
Cone Used: HO752TC U2
Elevation: +12.0

CPT Date/Time: 04-17-01 07:18
Location: CPT-3
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

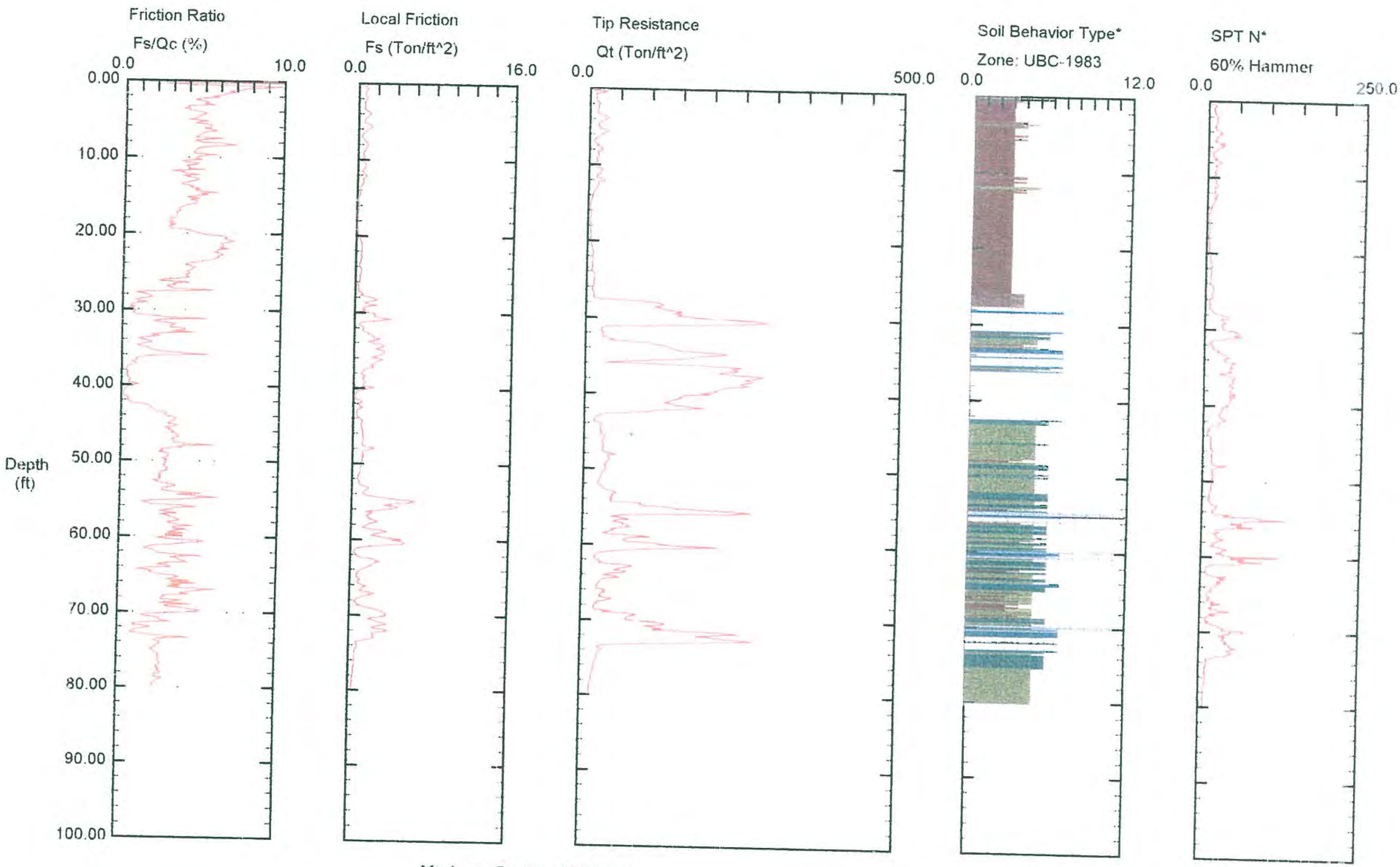
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z071
Cone Used: HO752TC U2
Elevation: +14.4

CPT Date/Time: 04-17-01 08:23
Location: CPT-4
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

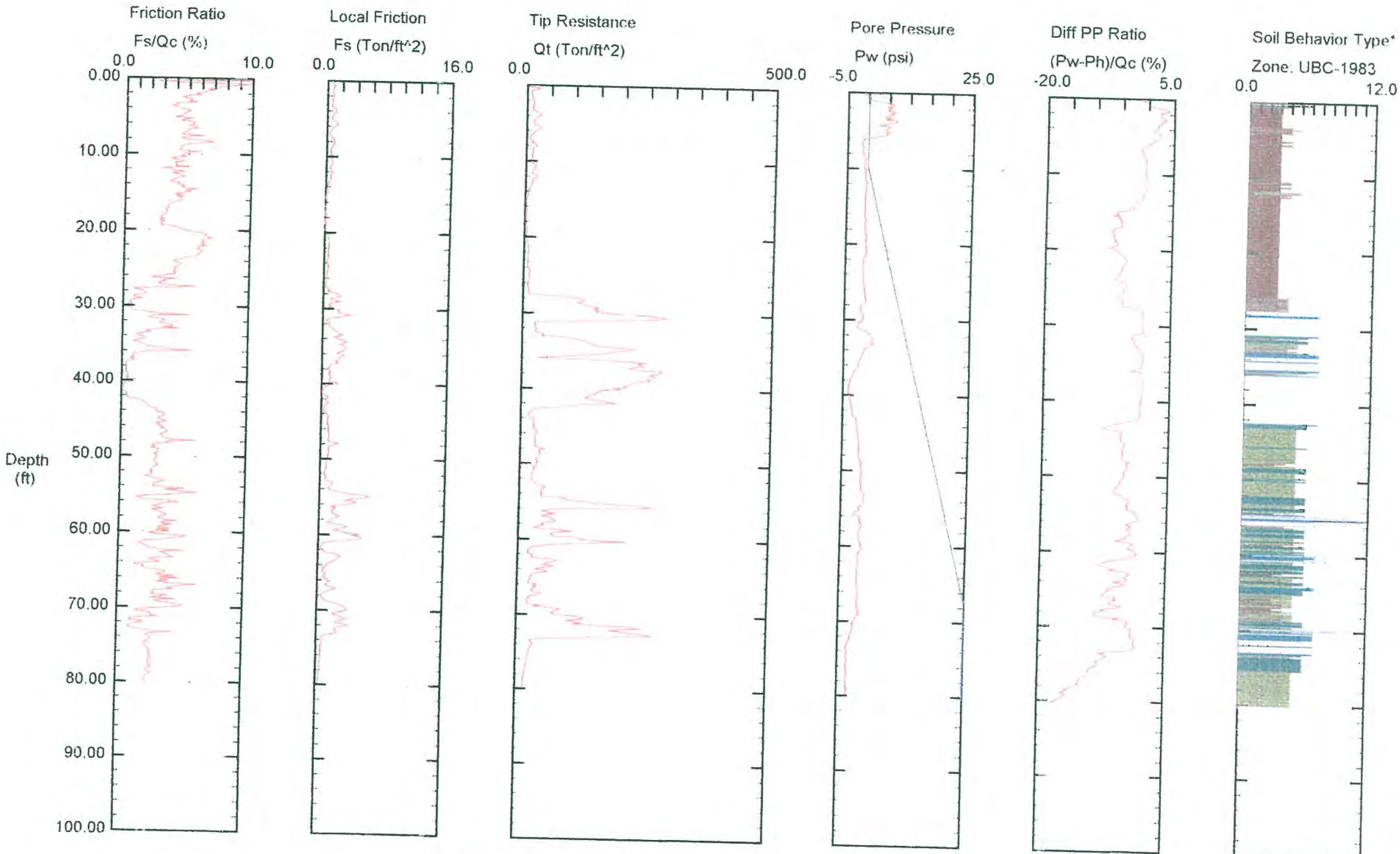
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z071
Cone Used: HO752TC U2
Elevation: +14.4

CPT Date/Time: 04-17-01 08:23
Location: CPT-4
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

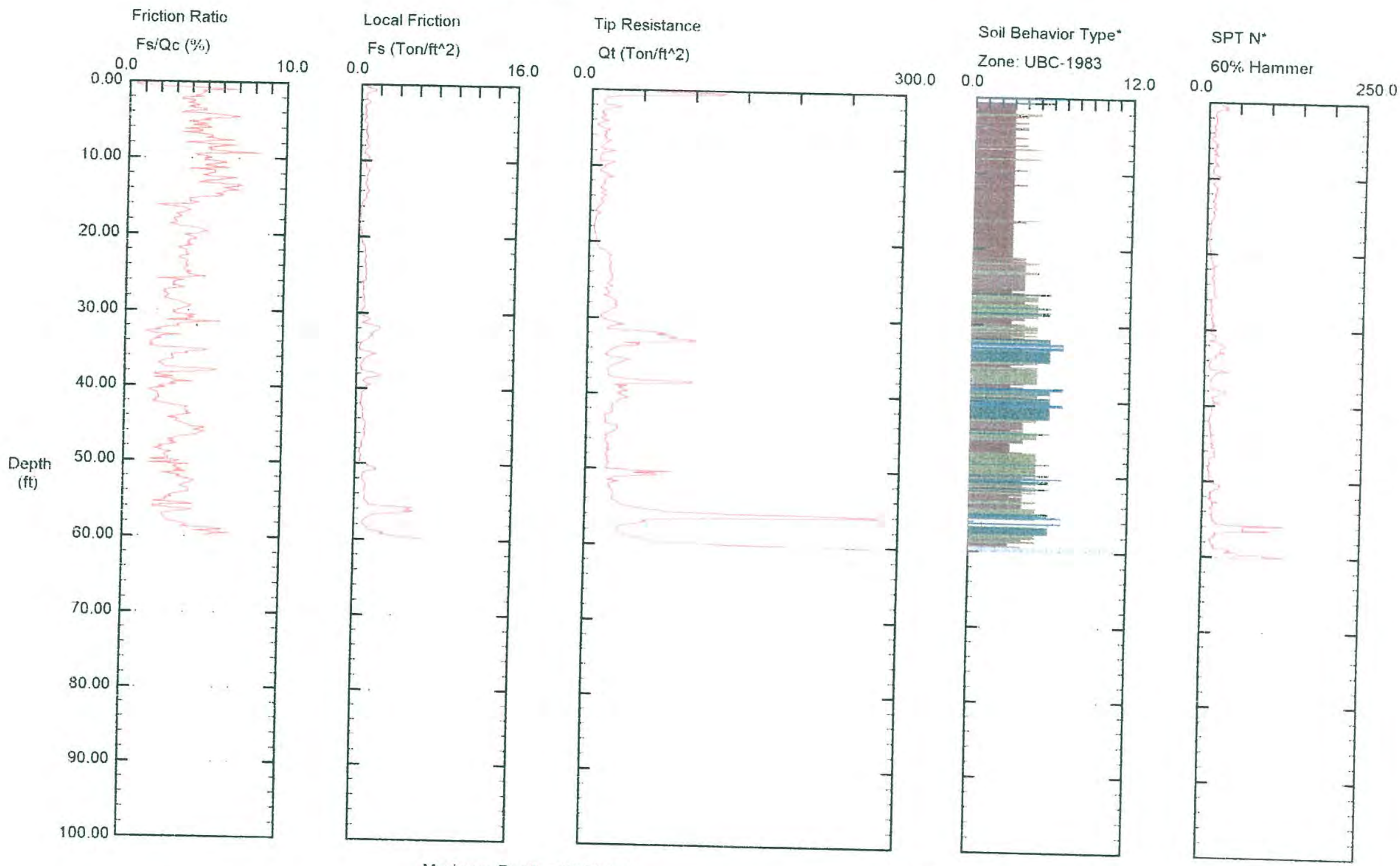
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z069
Cone Used: HO752TC U2
Elevation: +19.2

CPT Date/Time: 04-16-01 18:00
Location: CPT-5
Job Number: 18147-CA



Maximum Depth = 60.20 feet

Depth Increment = 0.16 feet

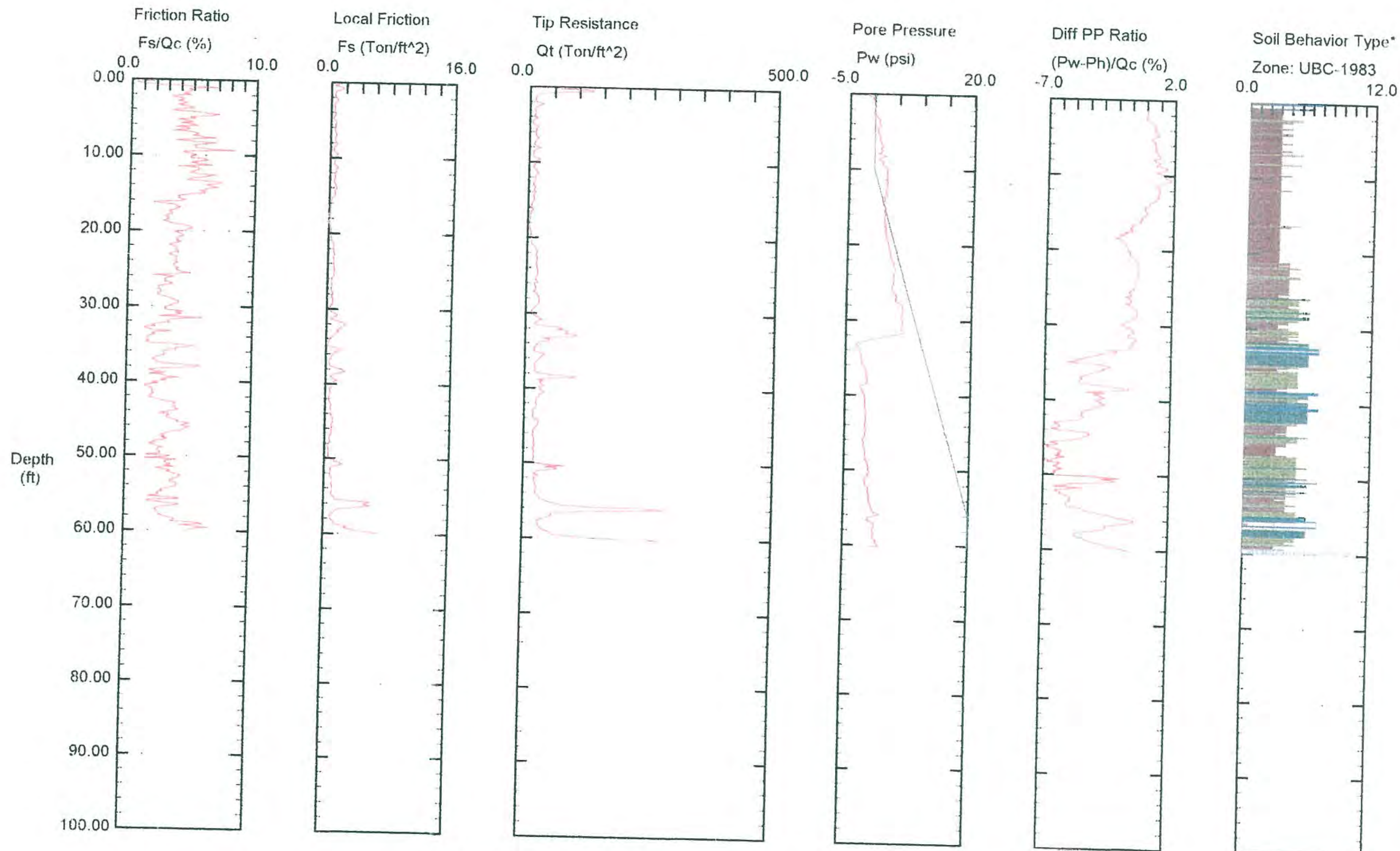
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z069
Cone Used: HO752TC U2
Elevation: +19.2

CPT Date/Time: 04-16-01 18:00
Location: CPT-5
Job Number: 18147-CA



Maximum Depth = 60.20 feet

Depth Increment = 0.16 feet

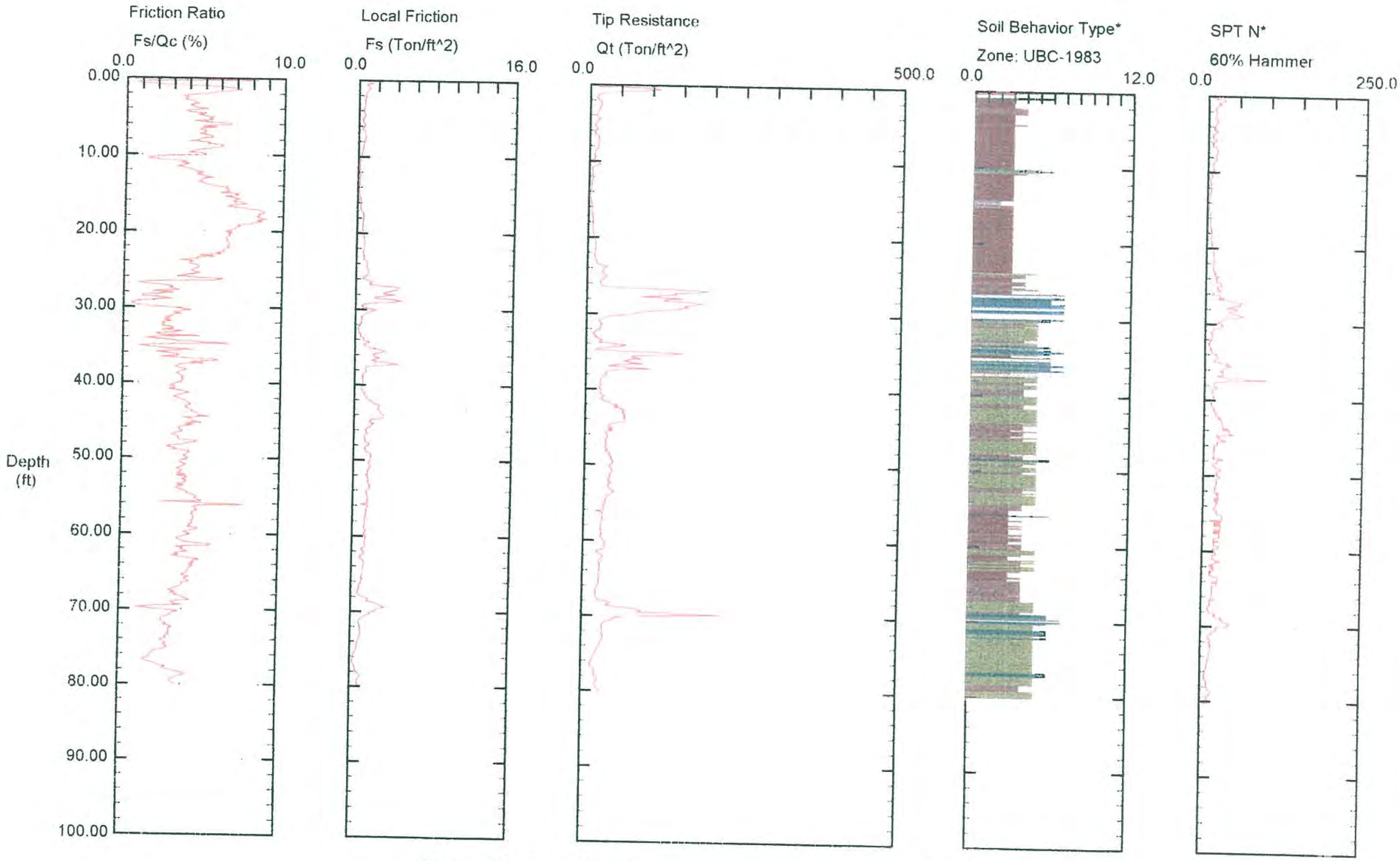
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z077
Cone Used: HO752TC U2
Elevation: +13.3

CPT Date/Time: 04-17-01 13:24
Location: CPT-6
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

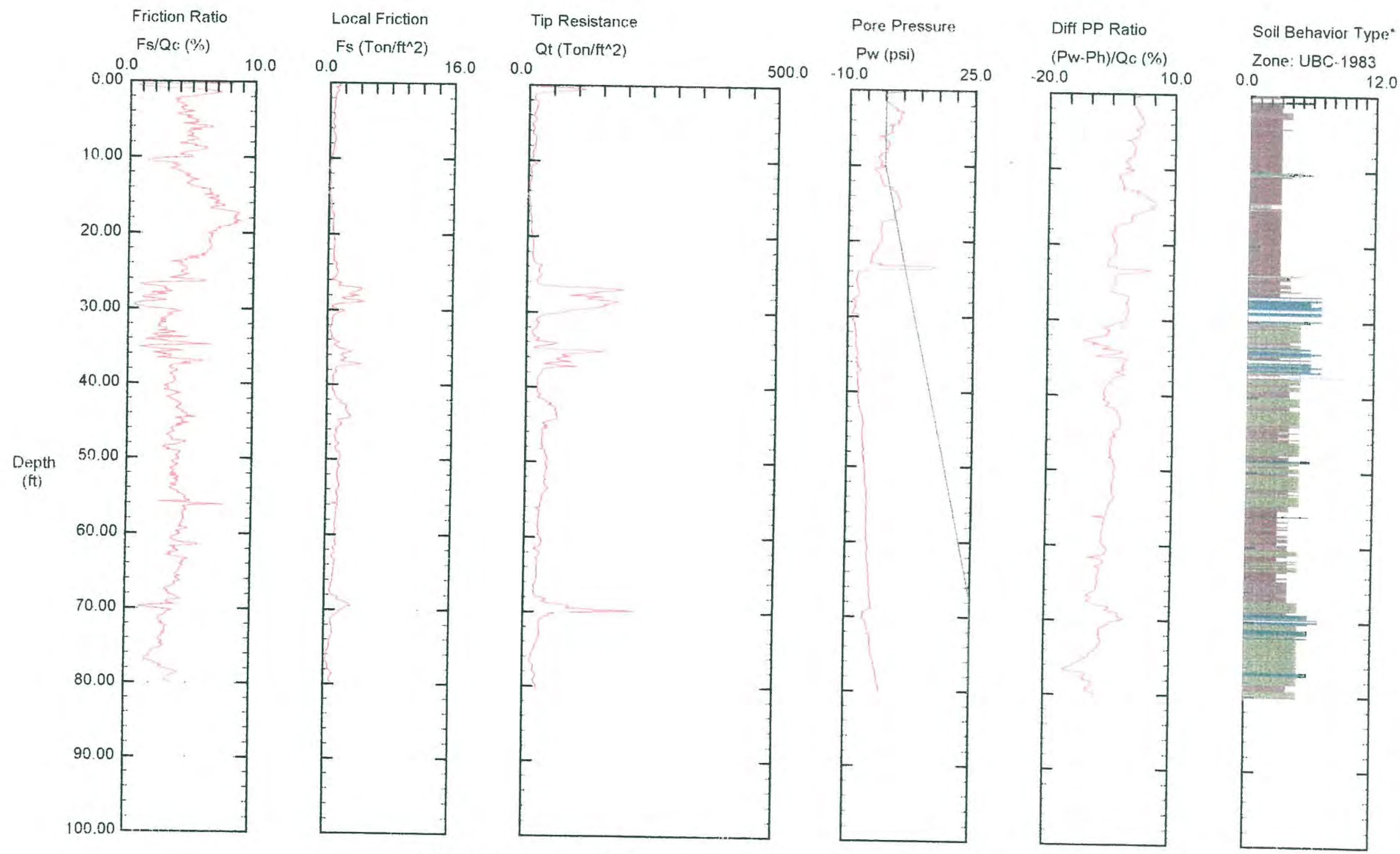
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z077
Cone Used: HO752TC U2
Elevation: +13.3

CPT Date/Time: 04-17-01 13:24
Location: CPT-6
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

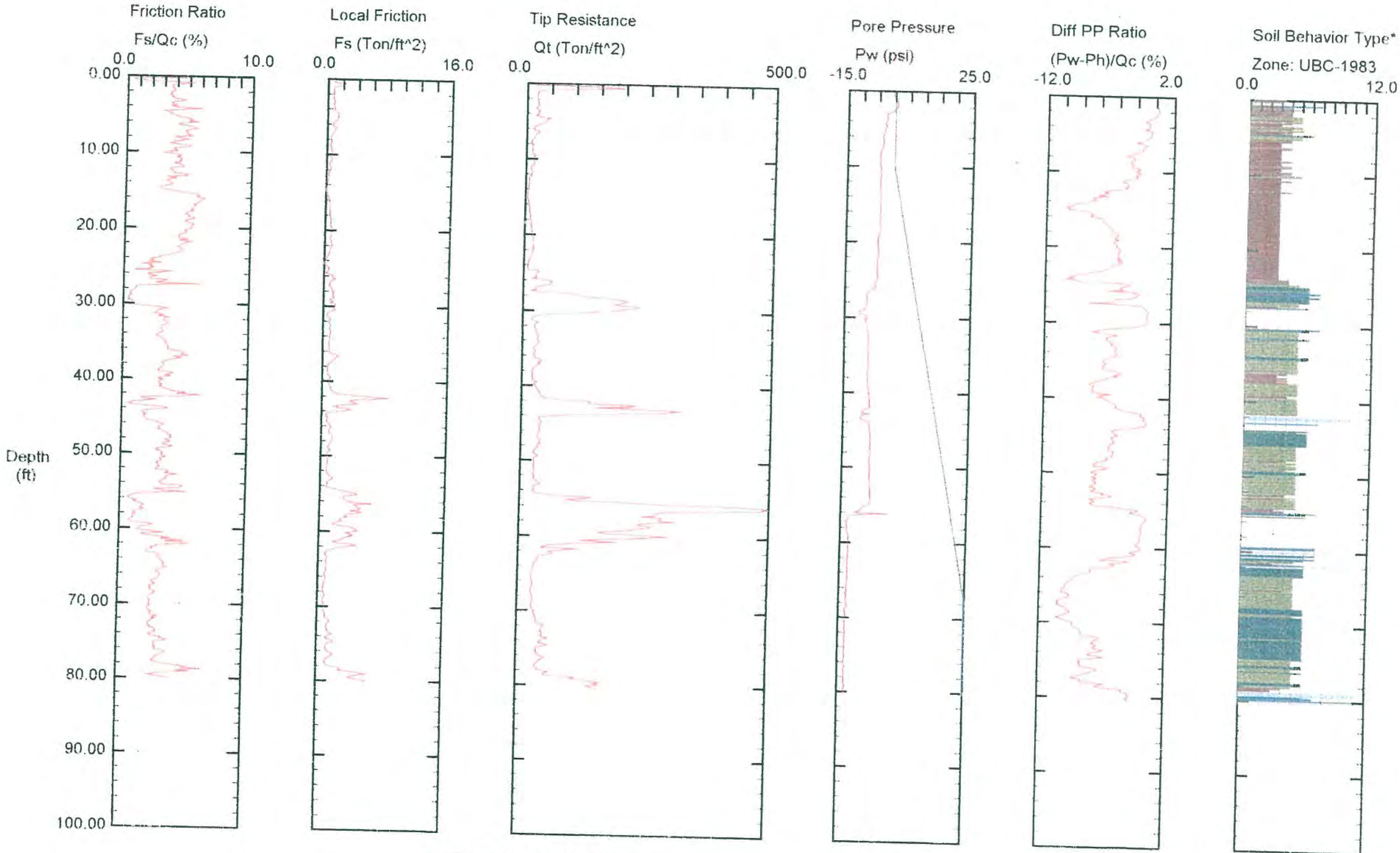
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z074
Cone Used: HO752TC U2
Elevation: +15.6

CPT Date/Time: 04-17-01 09:56
Location: CPT-7
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

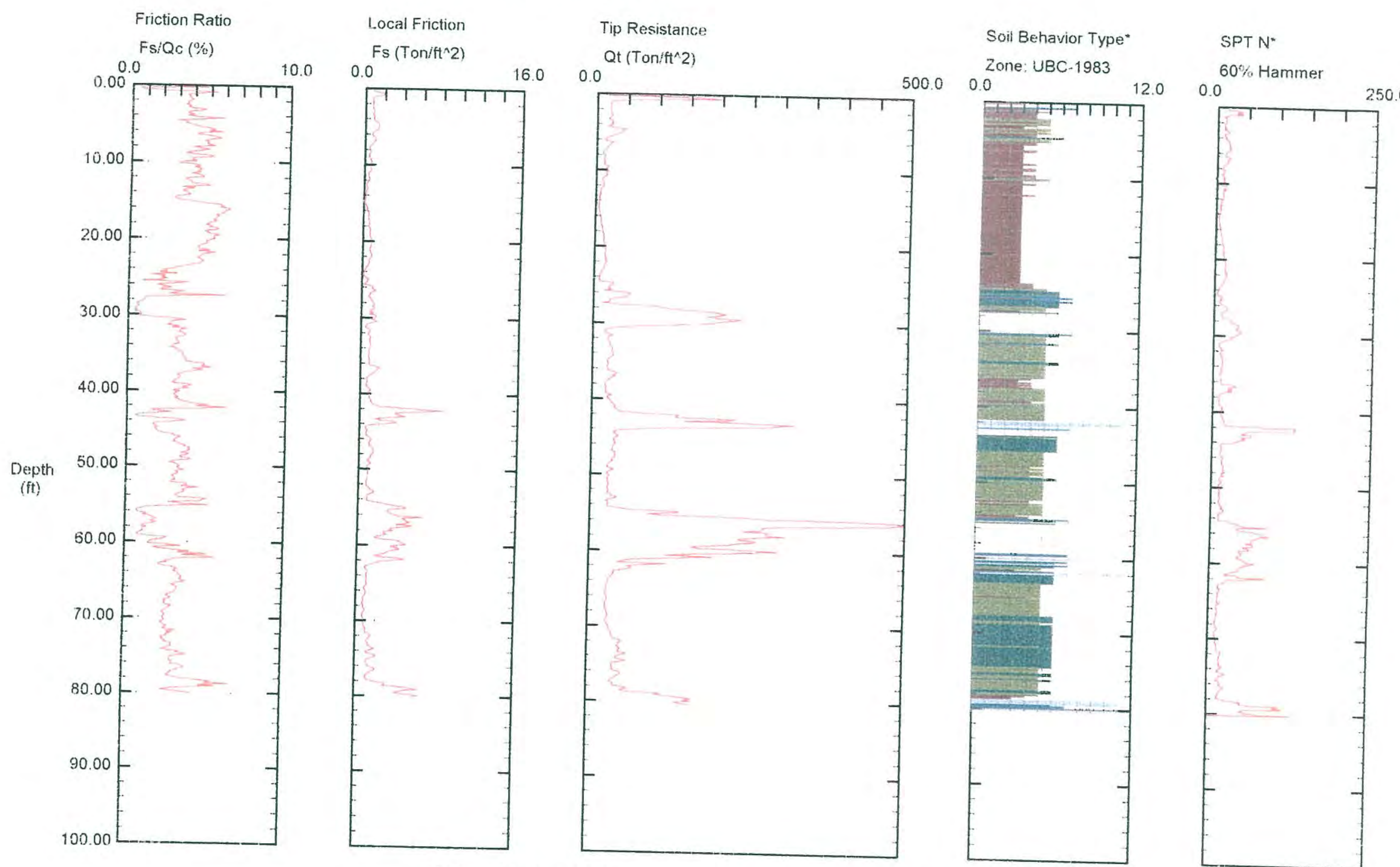
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z074
Cone Used: HO752TC U2
Elevation: +15.6

CPT Date/Time: 04-17-01 09:56
Location: CPT-7
Job Number: 18147-CA



Maximum Depth = 80.38 feet

Depth Increment = 0.16 feet

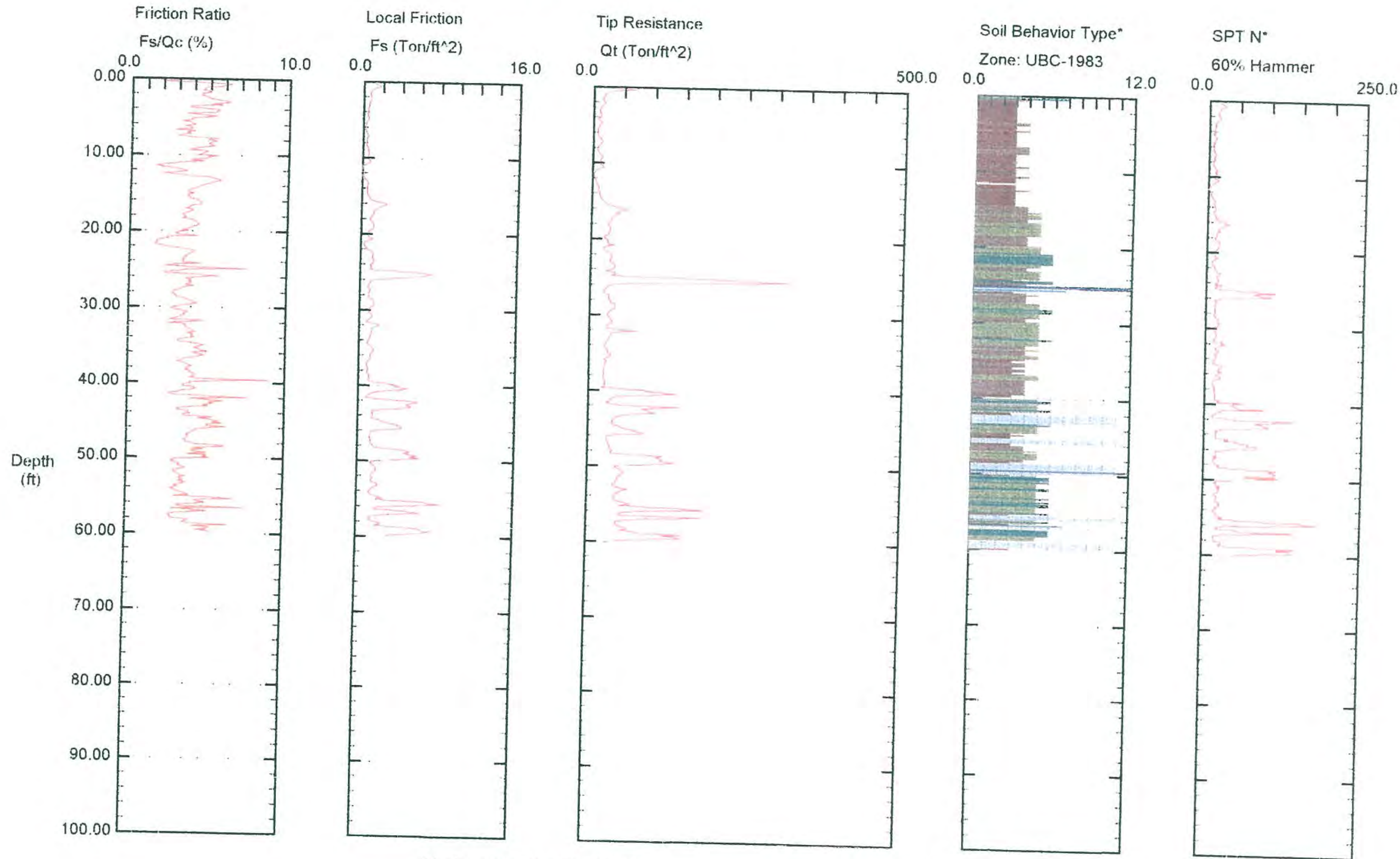
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z066
Cone Used: HO752TC U2
Elevation: +19.2

CPT Date/Time: 04-16-01 15:01
Location: CPT-8
Job Number: 18147-CA



Maximum Depth = 60.20 feet

Depth Increment = 0.16 feet

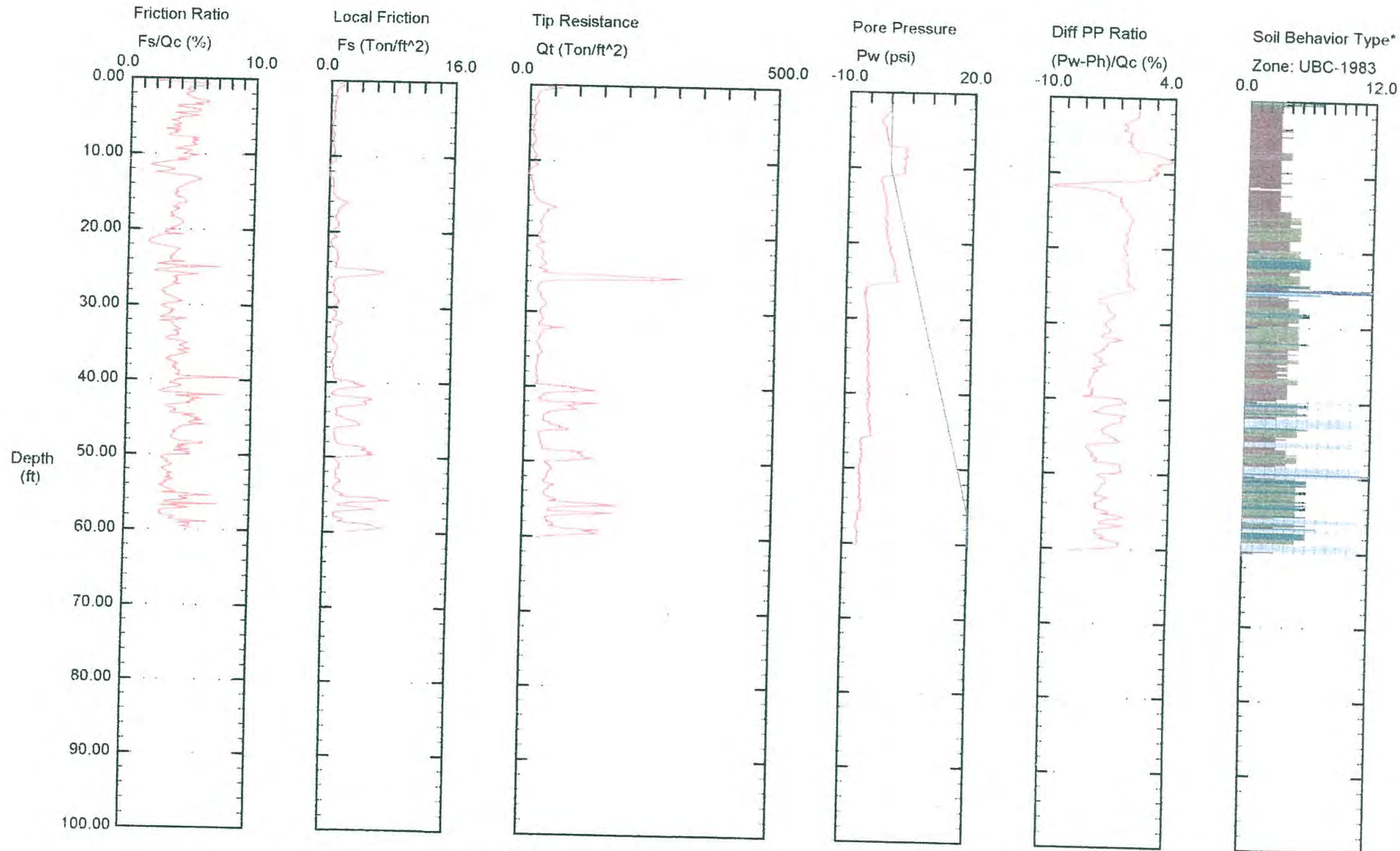
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z066
Cone Used: HO752TC U2
Elevation: +19.2

CPT Date/Time: 04-16-01 15:01
Location: CPT-8
Job Number: 18147-CA



Maximum Depth = 60.20 feet

Depth Increment = 0.16 feet

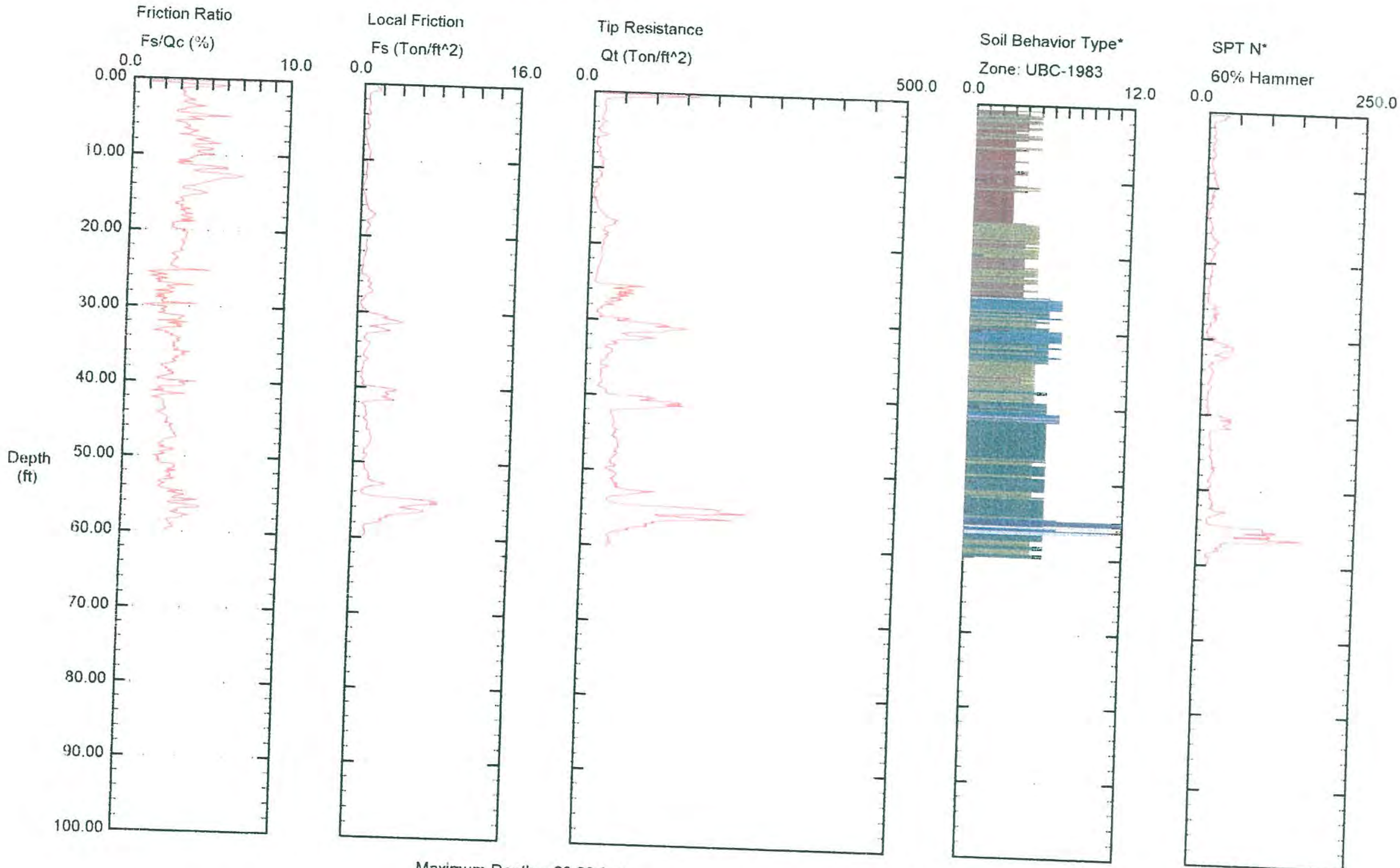
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z068
Cone Used: HO752TC U2
Elevation: +19.1

CPT Date/Time: 04-16-01 16:42
Location: CPT-9
Job Number: 18147-CA



Maximum Depth = 60.20 feet

Depth Increment = 0.16 feet

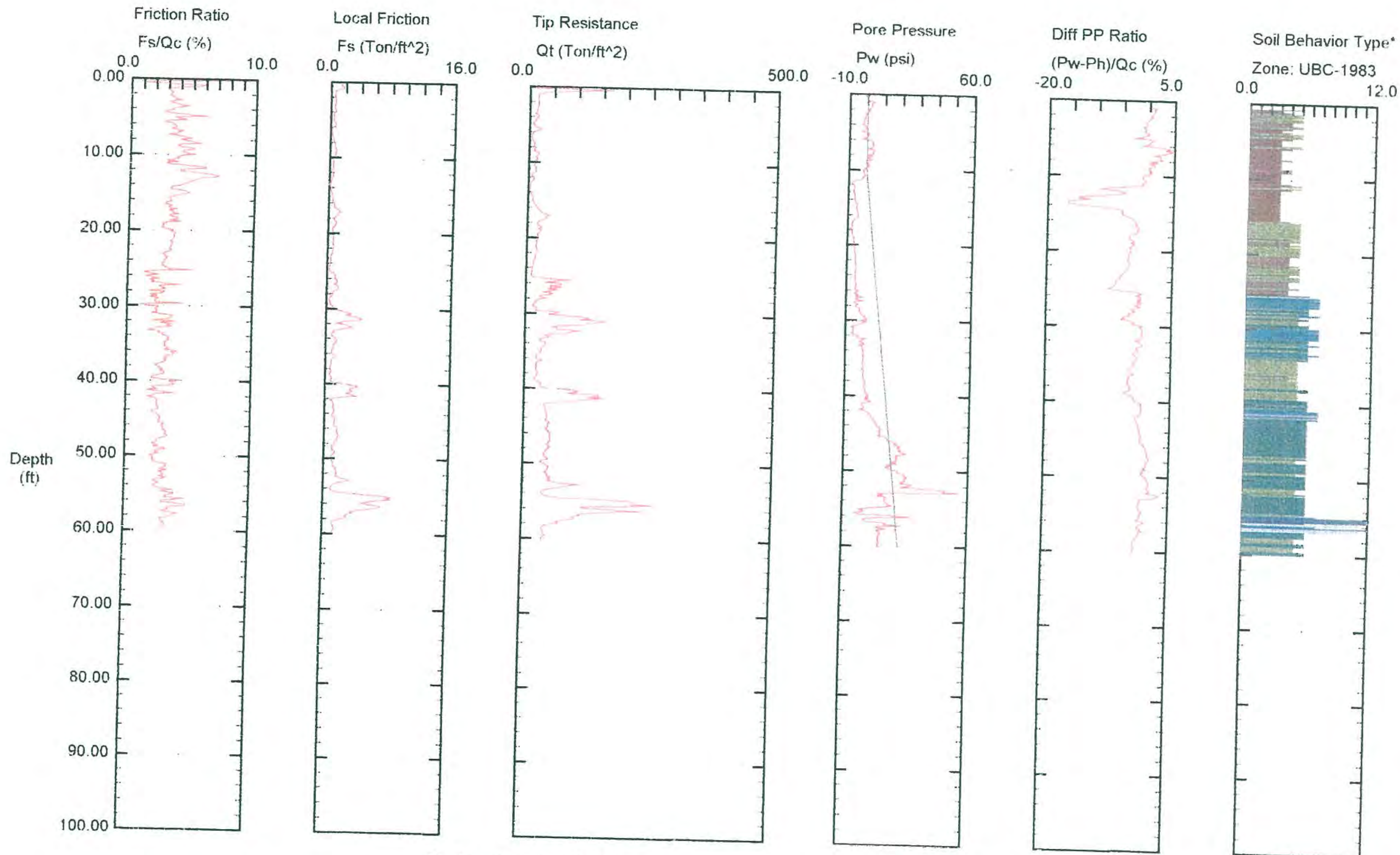
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z068
Cone Used: HO752TC U2
Elevation: +19.1

CPT Date/Time: 04-16-01 16:42
Location: CPT-9
Job Number: 18147-CA



Maximum Depth = 60.20 feet

Depth Increment = 0.16 feet

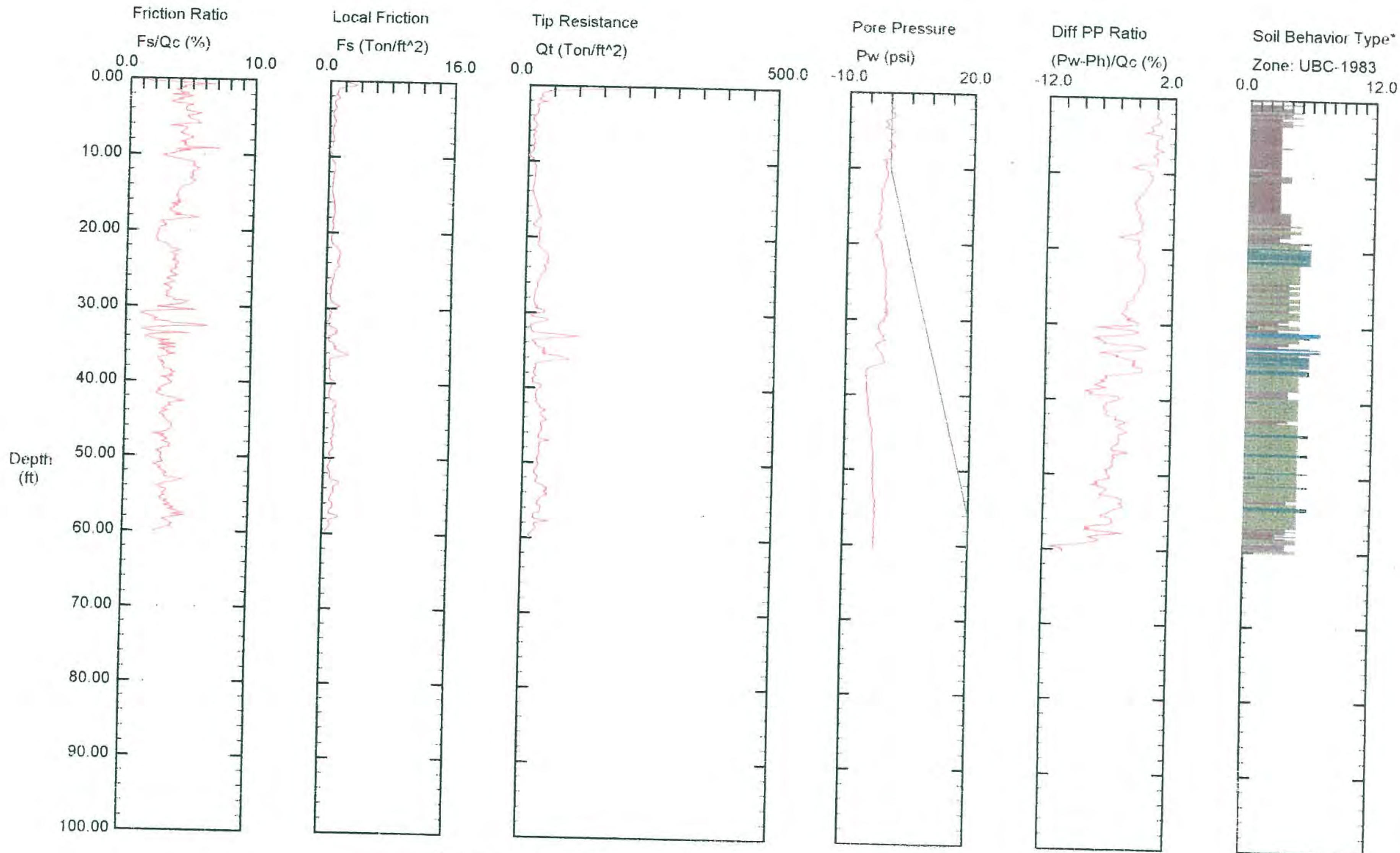
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z078
Cone Used: HO752TC U2
Elevation: +16.9

CPT Date/Time: 04-17-01 15:04
Location: CPT-10
Job Number: 18147-CA



Maximum Depth = 60.37 feet

Depth Increment = 0.16 feet

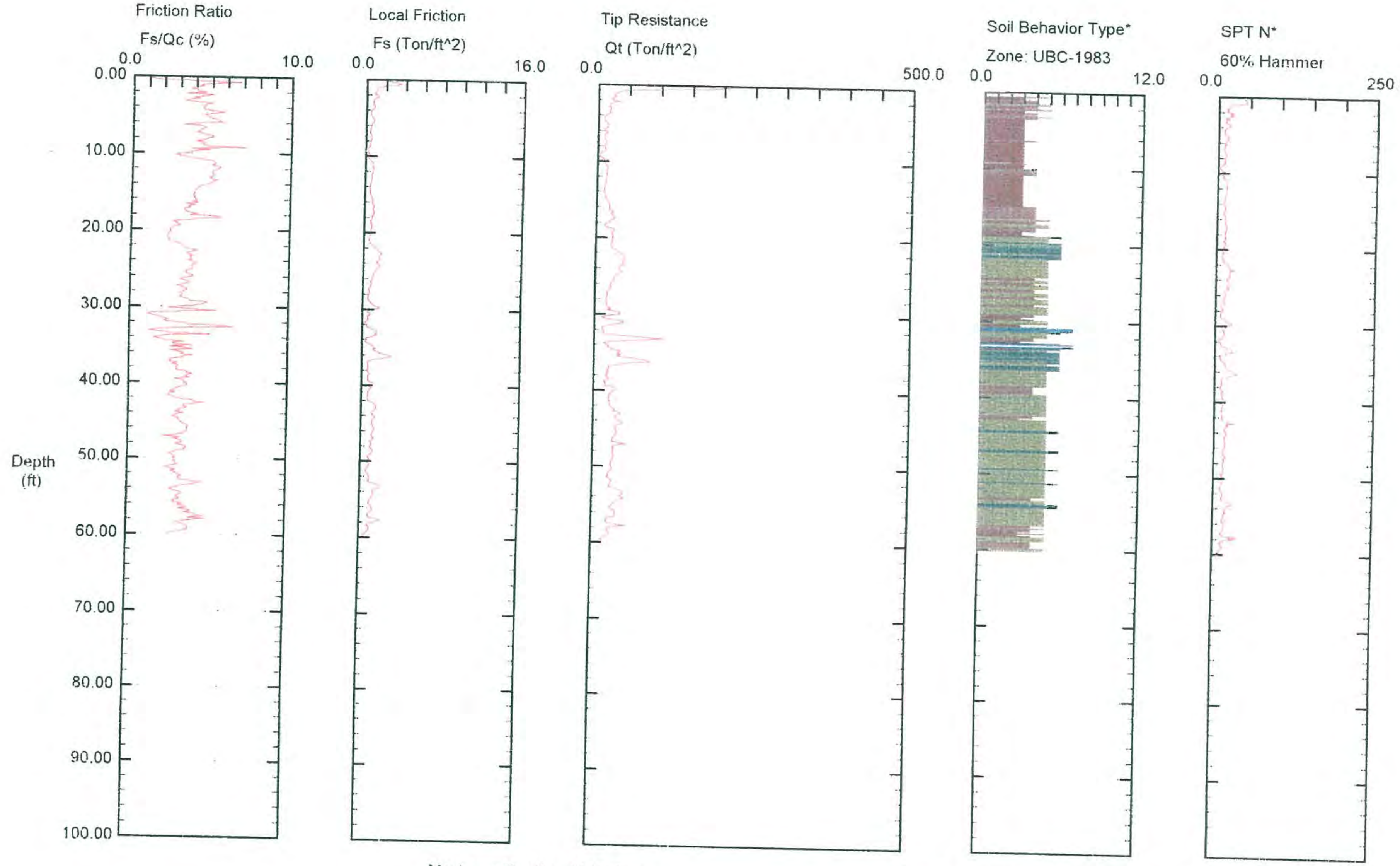
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01Z078
Cone Used: HO752TC U2
Elevation: +16.9

CPT Date/Time: 04-17-01 15:04
Location: CPT-10
Job Number: 18147-CA



Maximum Depth = 60.37 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

APPENDIX B
LABORATORY TEST RESULTS - OXIDATION PONDS

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils around the Oxidation Ponds.

The natural water content was determined on 77 samples of the materials recovered from the borings in accordance with ASTM Test Designation D-2216. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry density determinations were performed on 58 samples of the subsurface soils to evaluate their physical properties. The results of these tests are shown on the boring logs at the appropriate sample depths.

Atterberg Limit determinations were performed on nine samples of the subsurface soils to determine the range of water content over which these materials exhibit plasticity. The Atterberg Limits were determined in accordance with ASTM Test Designations D-428 and D-424. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of these tests are presented on Figures B-1 and B-2, and on the logs of the borings at the appropriate sample depths.

The percent passing the #200 sieve was determined on 28 samples of the subsurface soils to aid in the classification of these soils. These tests were performed in accordance with ASTM Designation D-1140. The results of these tests are shown on the boring logs at the appropriate sample depths.

Gradation tests were performed on six samples of the subsurface soils in accordance with California Test Method No. 202. These tests were performed to assist in the classification of the soils and to determine their grain size distribution. The results of these tests are presented on Figure B-3.

Hydrometer tests were performed on two samples of the subsurface soils in accordance with ASTM Test Method No. D422. These tests were performed to assist in the classification of the soils and to determine their grain size distribution. The results of these tests are presented on Figure B-3.

Unconfined compression tests were performed on 38 undisturbed samples of the clayey subsurface soils to evaluate the undrained shear strengths of these materials. The unconfined tests were performed in accordance with ASTM Test Designation D-2166 on samples having a diameter of 2.4 inches and a height-to-diameter ratio of at least two. Failure was taken as the peak normal stress. The results of these tests are presented in Figures B-4 through B-6, and on the boring logs at the appropriate sample depths.

Five unconsolidated, undrained, triaxial compression tests were performed on undisturbed samples of the subsurface soils to evaluate the strength of these materials. The tests were performed on samples having a diameter of 1½ inches, and height-to-diameter ratio of at least two. Failure was taken as the peak normal stress. The results of these tests are presented on the boring logs at the appropriate sample depths.

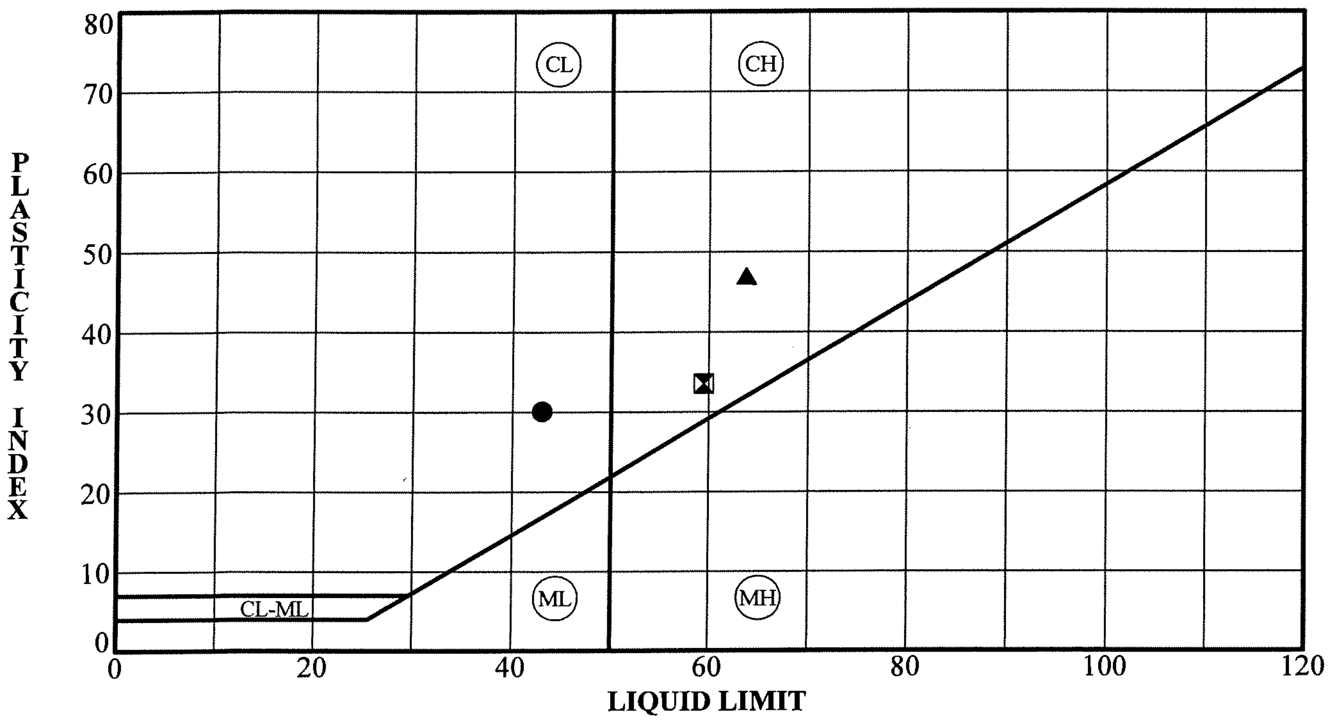
Two consolidation tests were performed on relatively undisturbed samples of the soft Bay Mud near Pond Nos. 1 and 10 to assist in evaluating the compressibility characteristics of these materials. The consolidation test was performed in accordance with ASTM Test Designation D-2438-70. The result of the consolidation test is presented graphically on Figures B-7 through B-11.

Laboratory compaction tests were performed on the bulk samples collected from depths up to 5 feet at potential borrow site Parcel B to determine the maximum dry density and optimum moisture content of these materials for use as backfill. The compaction test was performed in accordance with ASTM Test Designation D1557-91. The results of the tests are presented on Figure B-12.

Additionally, one constant head permeability test was performed on a representative sample of these soils to determine the permeability constant, k, of these materials. The test was performed in accordance with ASTM Test Designation D-4186 on a sample having a dry density of 94 psf and a water content of 28 percent. Using an applied pressure of 35 psi, the test indicated a permeability constant of 3 x 10^{-9} cm/second.

A resistance R- value test was performed on a representative sample of the surface soils on-site to provide data for pavement design. The test was performed in accordance with California Test Method 301-F and indicated an R-value of 13 at an exudation pressure of 300 pounds per square inch. The results of the test are presented below:

RESULTS OF R-VALUE TEST					
Description of Material	**Dry Density (pcf)**	**Water Content (%)**	**Exudation Pressure (psi)**	**Expansion Pressure (psf)**	**R-Value**
Gray Brown Silty Clay (CL-CH)	82.6	34.4	215	74	8
	85.2	33.2	255	135	11
	85.7	32.0	382	144	14
R-Value = 13 at Exudation pressure of 300 psi					



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	EB-10	4.0	43	30	0.2	20	74	CL
⊠	EB-13	3.5	59	34	0.5	41	85	CH
▲	EB-6	14.5	64	47	0.7	48	92	CH

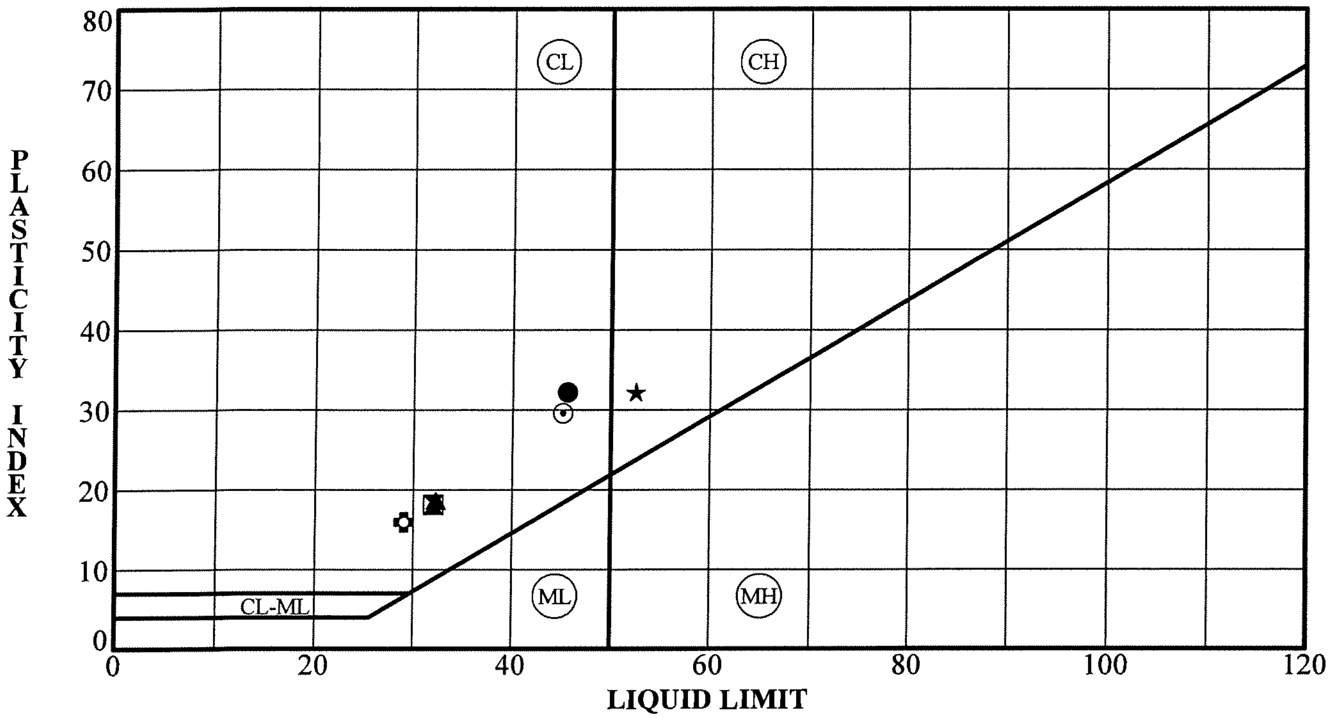
FUGRO

PREP'D BY:	JND
APP'D BY:	SR
DATE:	9/17/02
DWG FILE:	19639-GI.GPJ

PLASTICITY CHART AND DATA

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

FIGURE
B-1
PROJECT No.
3045.006



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	20	14.5	46	32	0.7	35	86	CL
⊠	21	24.5	32	18	0.5	22	34	SC
▲	23	29.5	32	19	0.8	28	54	ML
★	24	4.0	53	32	0.1	25	95	CH
⊙	24	14.0	45	30	0.5	30	91	CL
✪	24	19.5	29	16	0.4	20	41	CL

FUGRO

PREP'D BY:

APP'D BY:

DATE: 9/17/02

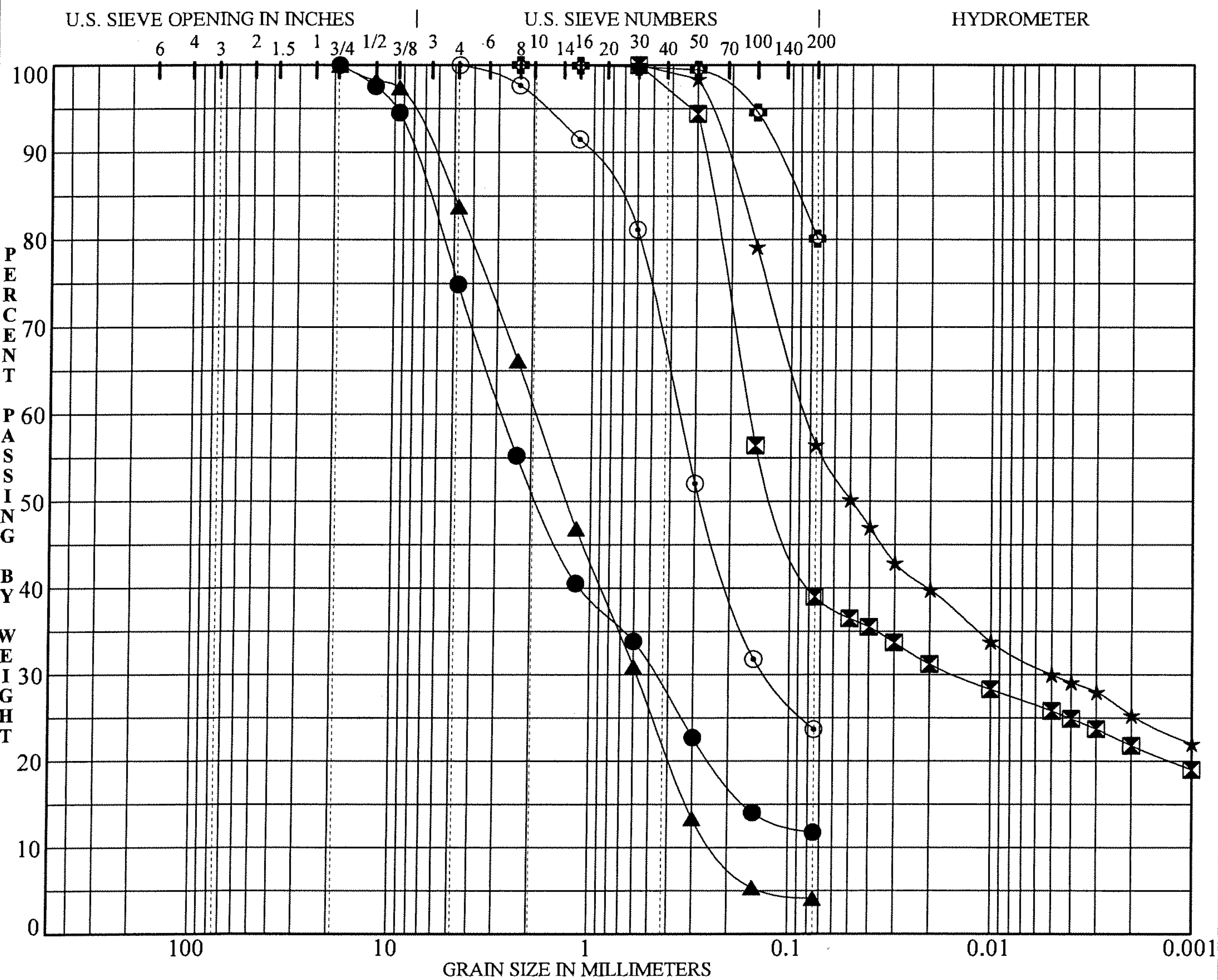
DWG FILE: 3045_006.GPJ

PLASTICITY CHART AND DATA

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

FIGURE
B-2

PROJECT No.
3045.006



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	20	54.5	12	75	Gray brown SAND, some gravel, trace clay	SW/SC
⊠	21	24.5	39		Light brown SAND, some clay and silt	SC
▲	21	49.5	4	84	Gray brown SAND, some gravel	SP
★	23	29.5	57		Light brown sandy SILT, some clay	ML
⊙	24	19.0	24	100	Brown SAND, some clay	SC
✪	24	49.0	80		Light Brown Silty CLAY, some sand	CL/ML

FUGRO

PREP'D BY:

APP'D BY:

DATE: 9/17/02

DWG FILE: 3045_006.GPJ

GRADATION TEST DATA

LAKEVILLE HIGHWAY WRF PROJECT

Petaluma, California

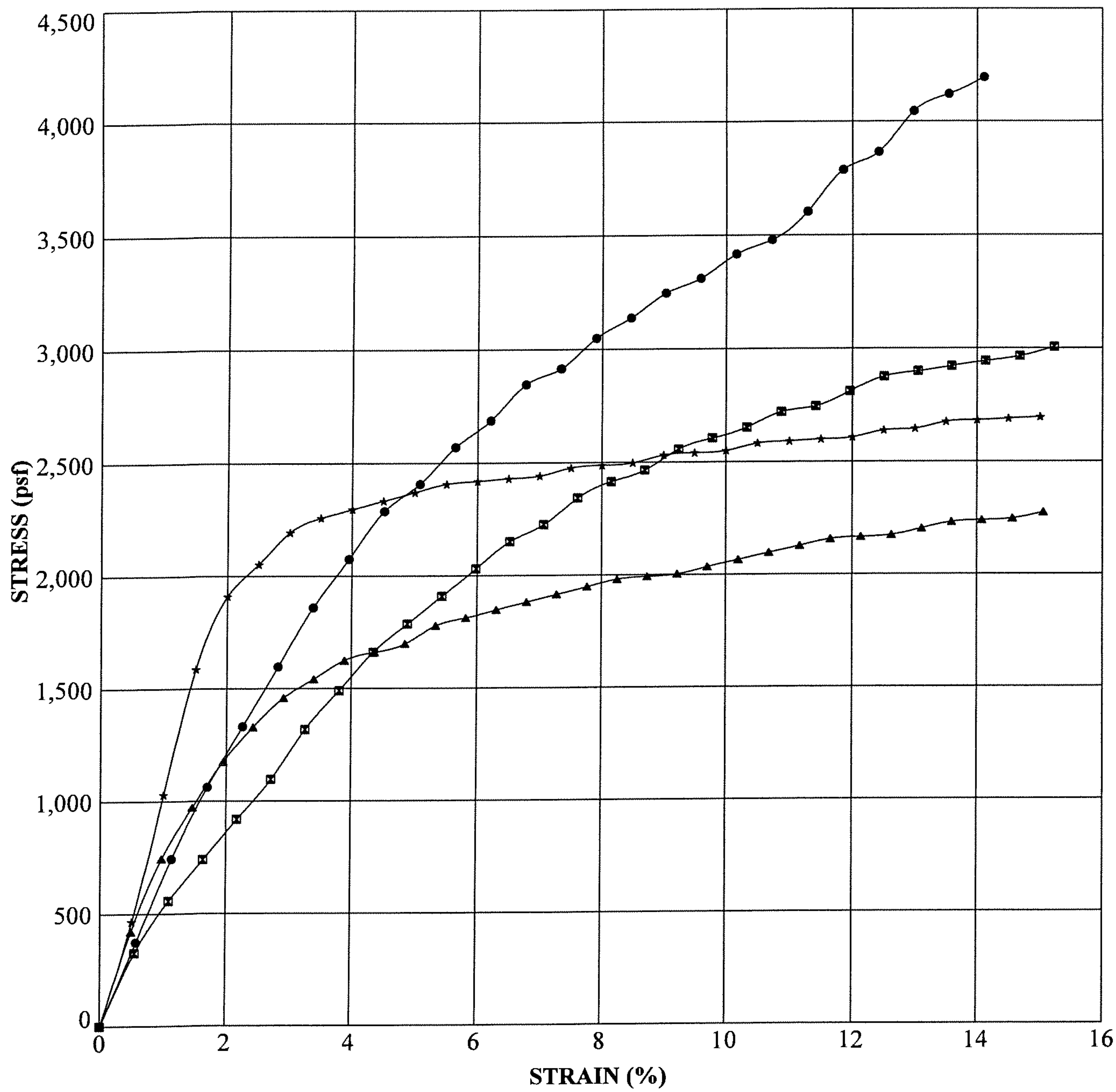
FIGURE

B-3

PROJECT No.

3045.006

File Name: G:\ENGINEER\GINTW\PROJECTS\3045_006.GPJ Report Template: FUGRO GRADATION B Output Date: 9/17/02



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Unconfined Strength (psf)	Strain (%)
●	20	44.0	Very Light Brown Silty CLAY Some Sand (CL)	112	19	4,195	14.1
⊠	21	59.5	Light brown silty CLAY, trace sand (CL)	114	17	3,006	15.2
▲	22	19.0	Gray Brown Silty CLAY, trace sand (CL)	99	23	2,274	15.0
★	23	19.0	Dark Brown CLAY (CH/CL)	107	21	2,696	15.0

MWH
MONTGOMERY WATSON HARZA

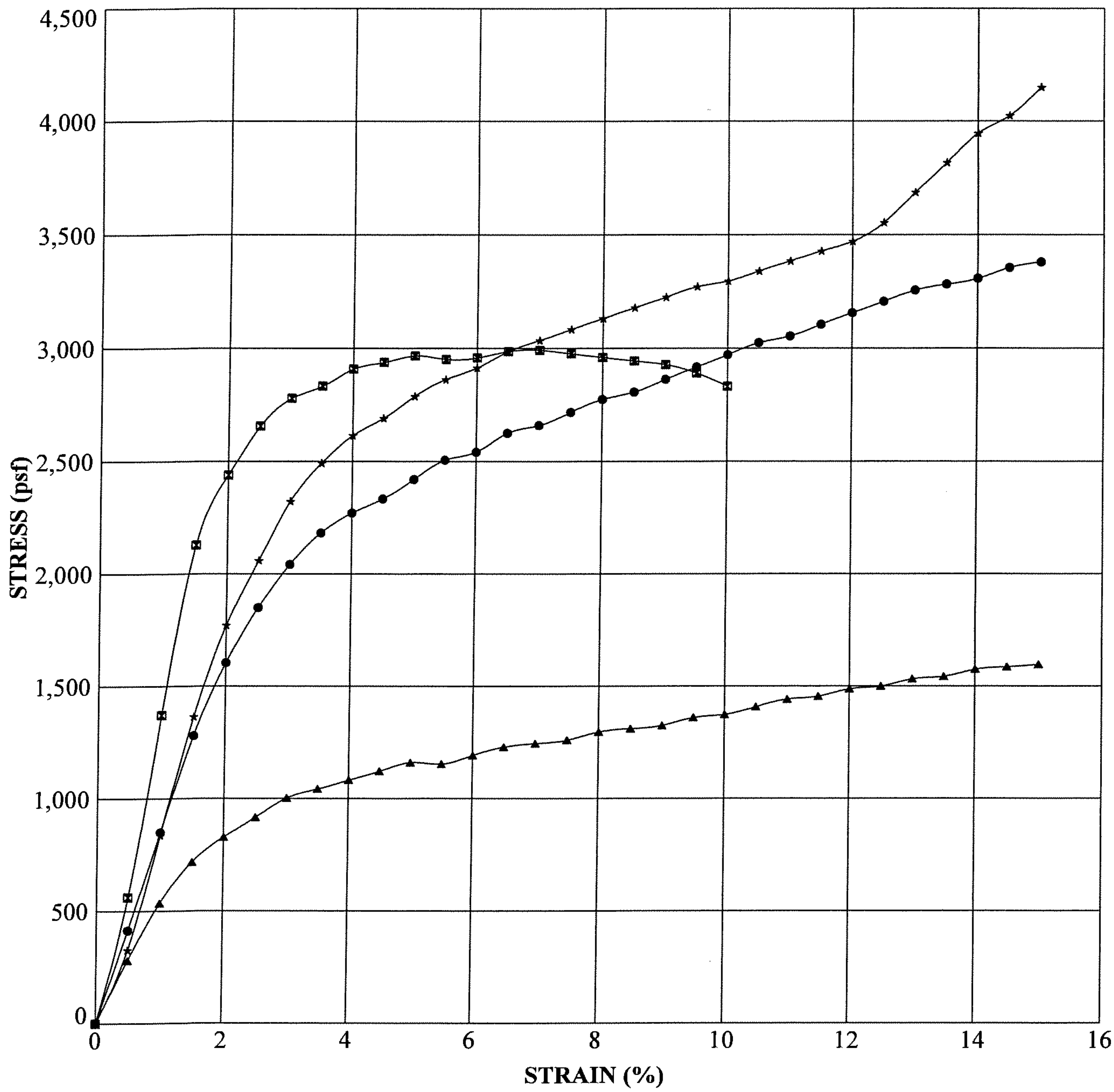
PREP'D BY:
APP'D BY:
DATE: 9/17/02
DWG FILE: 3045_006.GPJ

UNCONFINED COMPRESSION TEST DATA

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

FIGURE
B-4

PROJECT No.
3045.006



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Unconfined Strength (psf)	Strain (%)
●	24	29.0	Light Brown Silty CLAY (CL)	109	21	3,381	15.0
⊠	24	59.0	Light Brown Silty CLAY (CL)	96	27	2,992	7.0
▲	25	14.5	Gray Silty CLAY Trace Sand (CL)	100	21	1,597	15.0
★	25	34.0	Br & Gry Silty CLAY with Trace Sand (CL-ML)	104	21	4,151	15.0

MWH MONTGOMERY WATSON HARZA

PREP'D BY:
APP'D BY:
DATE: 9/17/02
DWG FILE: 3045_006.GPJ

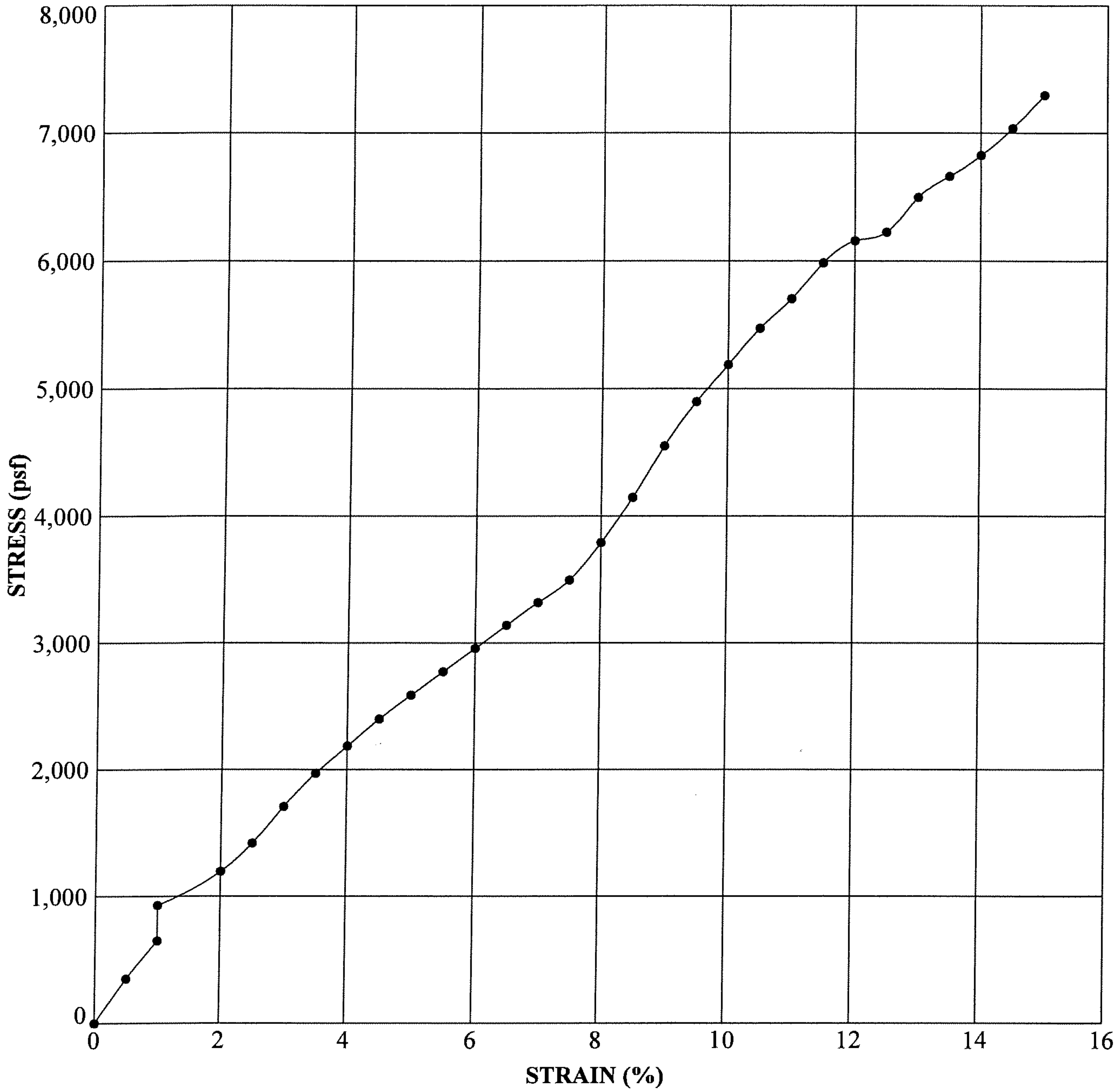
UNCONFINED COMPRESSION TEST DATA

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

FIGURE B-5

PROJECT No. 3045.006

File Name: G:\ENGINEER\GINTW\PROJECTS\3045_006.GPJ Report Template: UNCONFINED Output Date: 9/17/02



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Unconfined Strength (psf)	Strain (%)
●	26	9.5	Light Brown Sandy CLAY (CL)	122	15	7,295	15.0

File Name: G:\ENGINEER\GINT\WPROJECTS\3045_006.GPJ Report Template: UNCONFINED Output Date: 9/17/02

MWH
MONTGOMERY WATSON HARZA

PREP'D BY:

APP'D BY:

DATE: 9/17/02

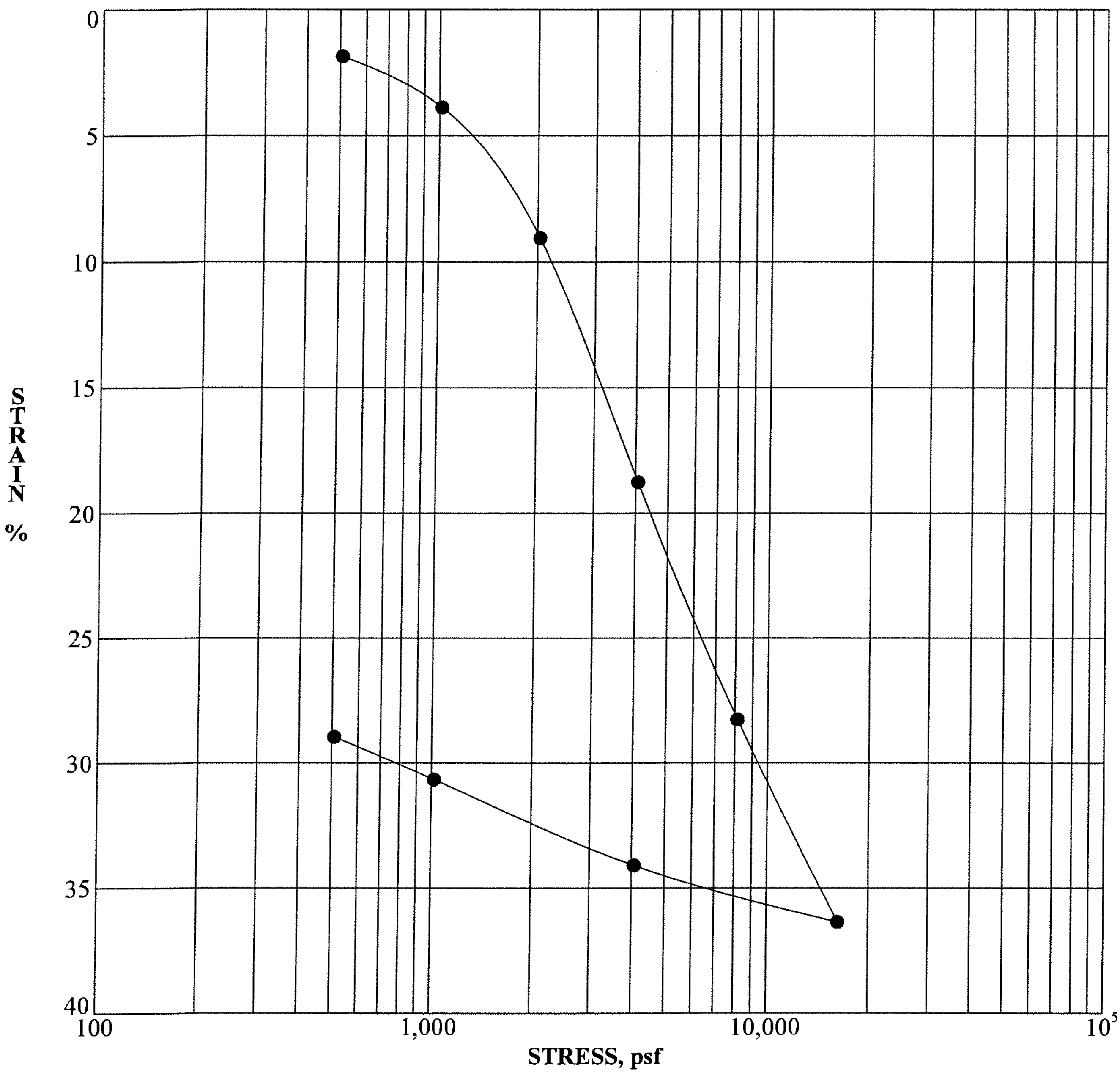
DWG FILE: 3045_006.GPJ

UNCONFINED COMPRESSION TEST DATA

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

FIGURE B-6

PROJECT No. 3045.006



Key Symbol	Boring No.	Depth (Feet)	Water Content (%)		Dry Density (pcf)		Void Ratio		Saturation (%)		Max. Past Pressure (psf)	Compr. Index,Cec	Recompr. Index,Cer
			Initial	Final	Initial	Final	Initial	Final	Initial	Final			
●	EB-2	18.0	92.4	56.2	47.6	67.0	2.541	1.516	98.2	100.1			

FUGRO

PREP'D BY: JND

APP'D BY: SR

DATE: 9/17/02

DWG FILE: 19639-GI.GPJ

CONSOLIDATION TEST RESULTS

LAKEVILLE HIGHWAY WRF PROJECT

Petaluma, California

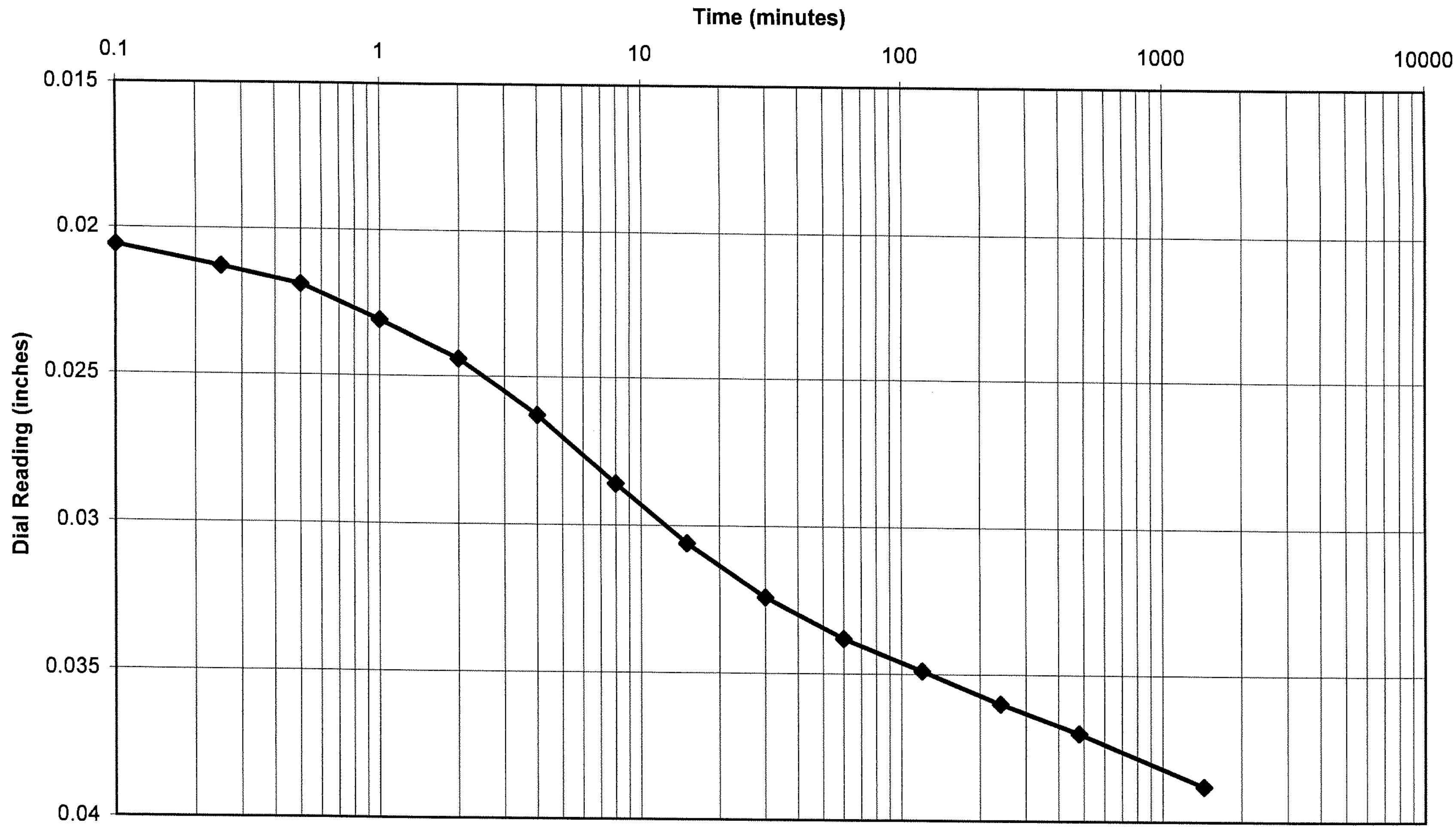
FIGURE

B-7

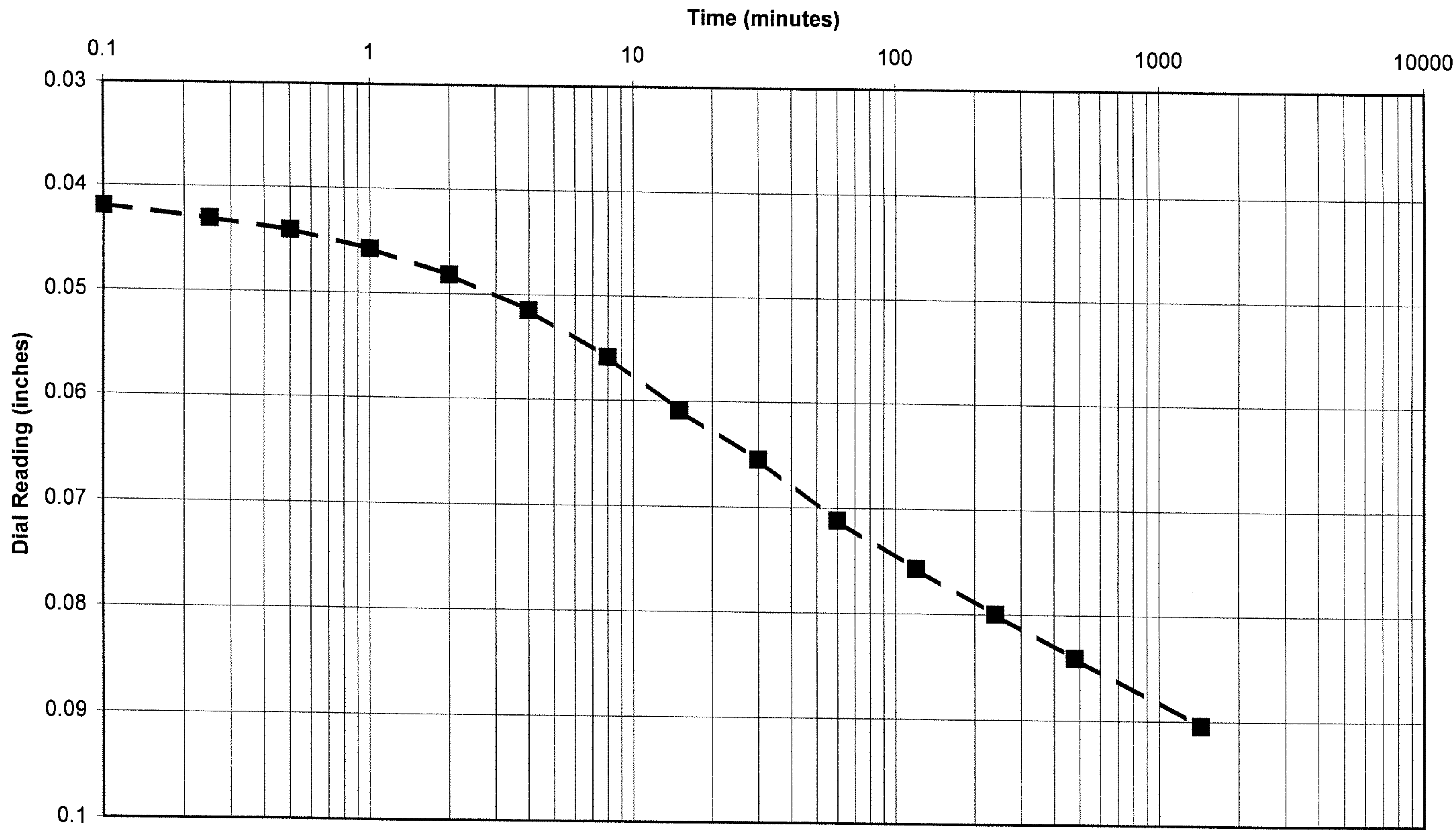
PROJECT No.

3045.006

File Name: G:\ENGINEER\GINTW\PROJECTS\19639-GI.GPJ Report Template: FUGRO CONSOL STRAIN Output Date: 9/17/02



Drawn By: SR Prepared By: SR Approved By: SR Scale: AS SHOWN Date: May 23 2001 Drawing File: 3045.006	Consolidation Test Data (1017 psf), EB-2 at 18 feet **Lakeville Highway WRF Project** **Petaluma, California**	FIGURE **B-8** PROJECT NO. **3045.006**



FUGRO

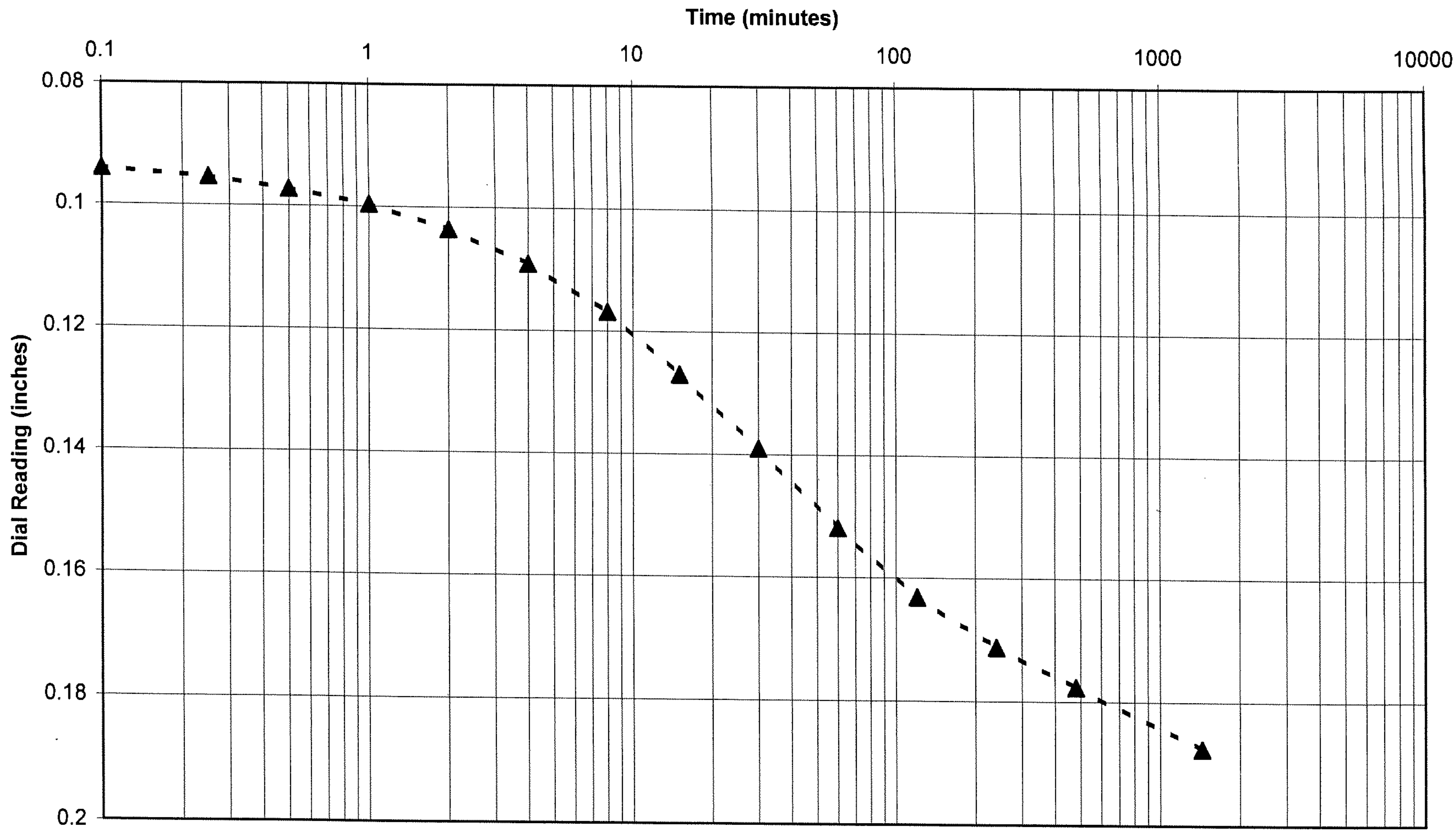
Drawn By:	SR
Prepared By:	SR
Approved By:	SR
Scale:	AS SHOWN
Date:	May 23 2001
Drawing File:	3045.006

Consolidation Test Data (2033 psf), EB-2 at 18 feet

Lakeville Highway WRF Project
Petaluma, California

FIGURE
B-9

PROJECT NO.
3045.006



FUGRO

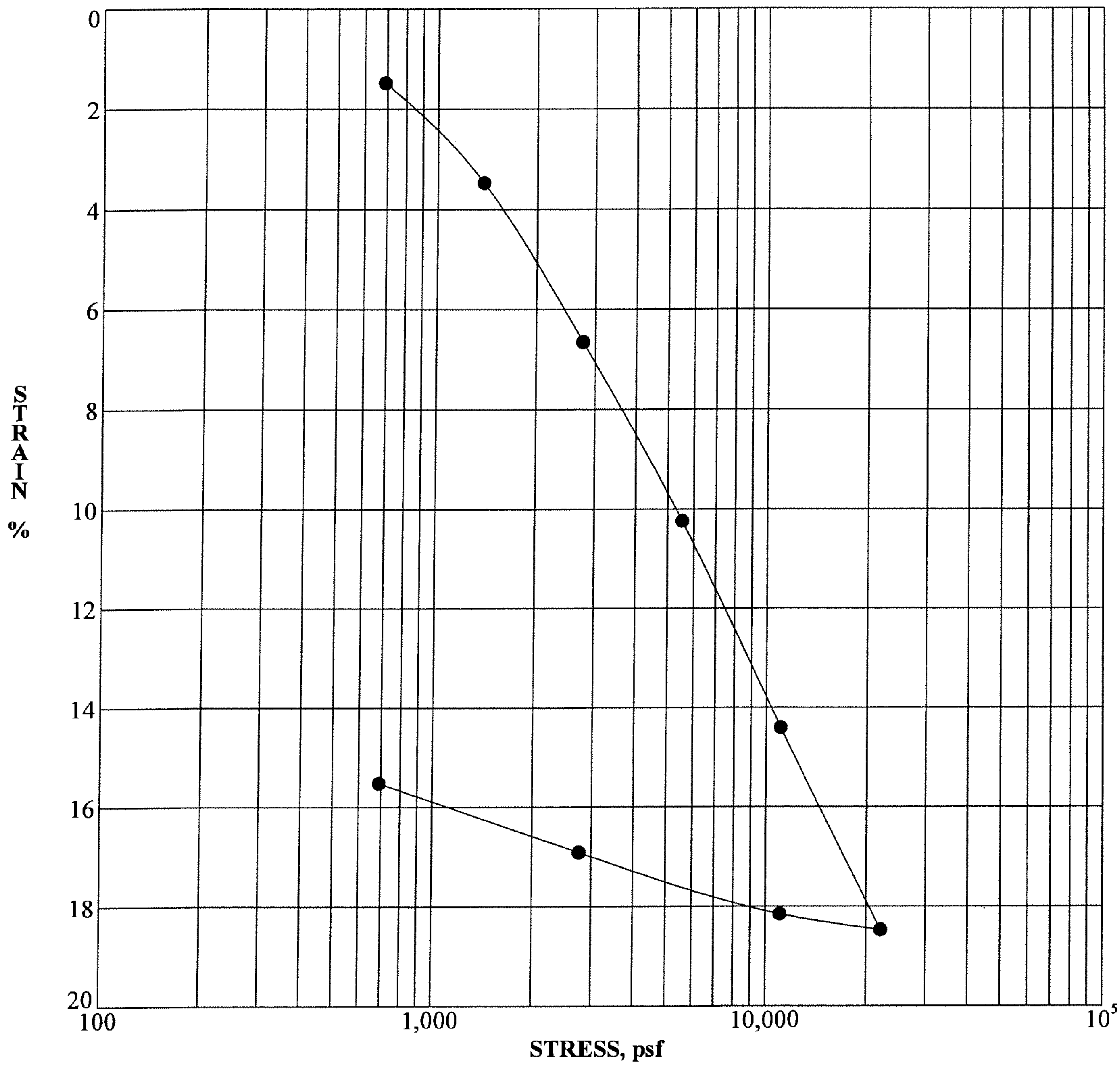
Drawn By:	SR
Prepared By:	SR
Approved By:	SR
Scale:	AS SHOWN
Date:	May 23 2001
Drawing File:	3045.006

Consolidation Test Data (4066 psf), EB-2 at 18 feet

Lakeville Highway WRF Project
Petaluma, California

FIGURE
B-10

PROJECT NO.
3045.006



Key Symbol	Boring No.	Depth (Feet)	Water Content (%)		Dry Density (pcf)		Void Ratio		Saturation (%)		Max. Past Pressure (psf)	Compr. Index,Cec	Recompr. Index,Cer
			Initial	Final	Initial	Final	Initial	Final	Initial	Final			
●	23	14.0	29.2	24.0	86.5	102.3	0.949	0.648	83.1	100.1			

FUGRO

PREP'D BY:

APP'D BY:

DATE: 9/17/02

DWG FILE: 3045_006.GPJ

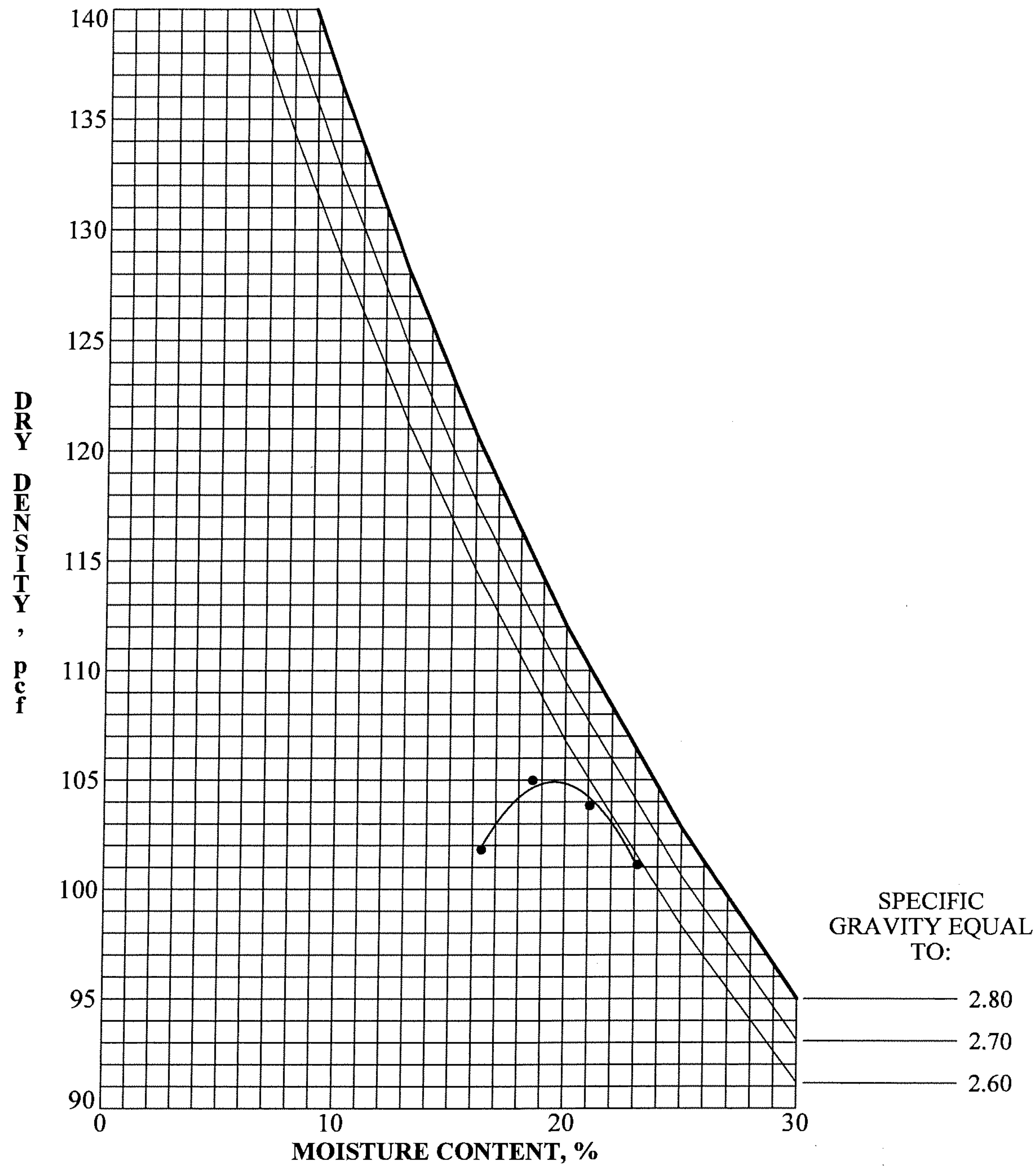
CONSOLIDATION TEST RESULTS

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

FIGURE **B-11**

PROJECT No. 3045.006

File Name: G:\ENGINEER\GINTW\PROJECTS\3045_006.GPJ Report Template: FUGRO CONSOL_STRAIN Output Date: 9/17/02



Key Symbol	Location	Depth (Feet)	Sample Description (USCS)	Maximum Dry Density (pcf)	Optimum Water Content (%)	Test Designation
●	BS-1&2	0.5	Gray brown silty CLAY, trace sand (CL-CH)	105	19	ASTM D1557 (B)

FUGRO

PREP'D BY:

APP'D BY:

DATE: 9/17/02

DWG FILE: 19639-GI.GPJ

COMPACTION TEST RESULTS

LAKEVILLE HIGHWAY WRF PROJECT
Petaluma, California

FIGURE **B-12**

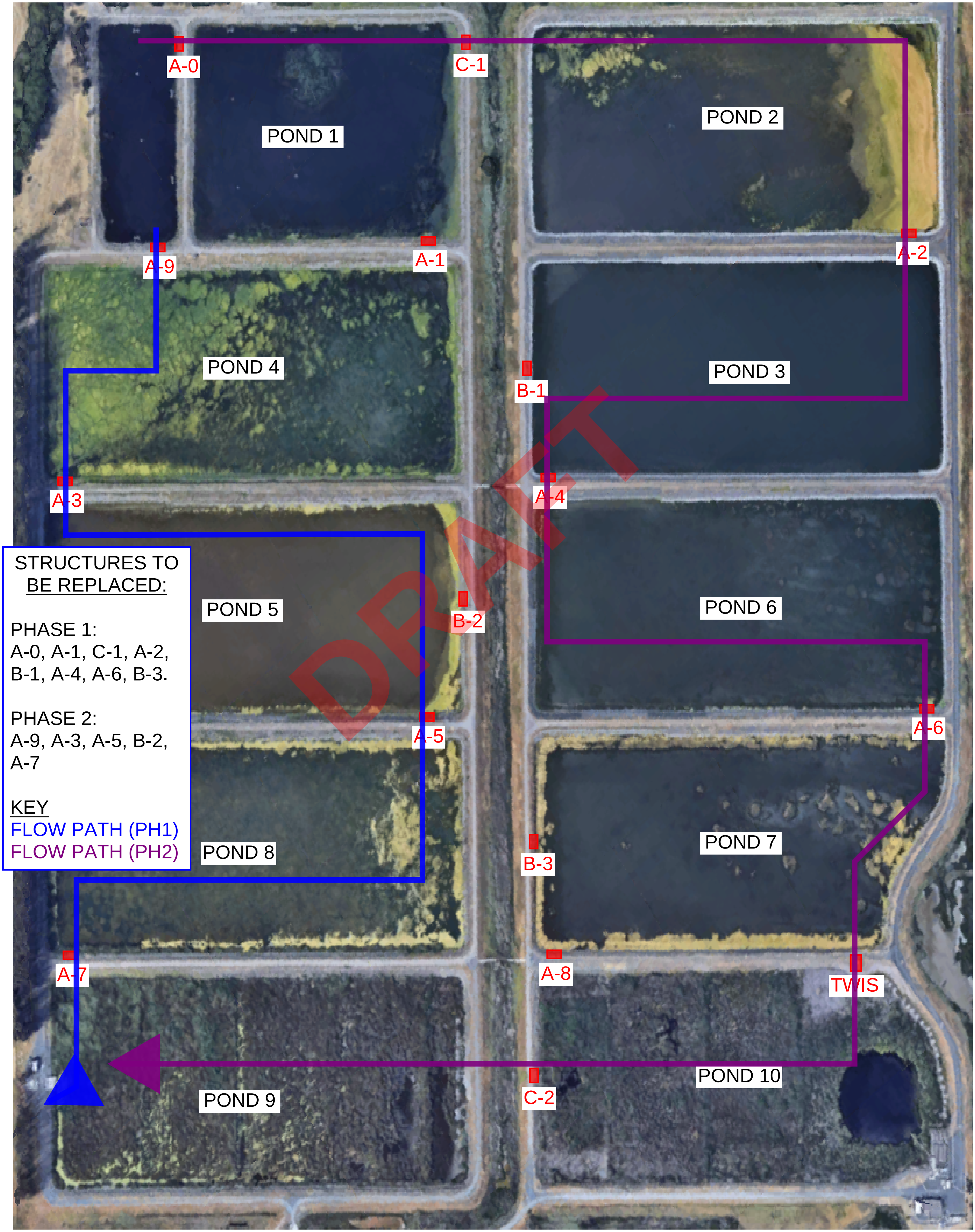
PROJECT No. 3045.006

File Name: G:\ENGINEER\GINTW\PROJECTS\19639-GI.GPJ Report Template: FUGRO COMP_B Output Date: 9/17/02

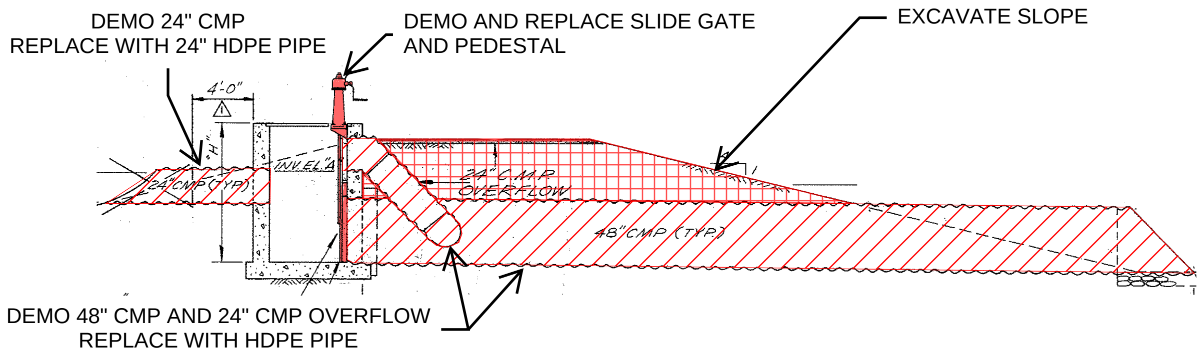
APPENDIX E

Draft Transfer Structure Drawings

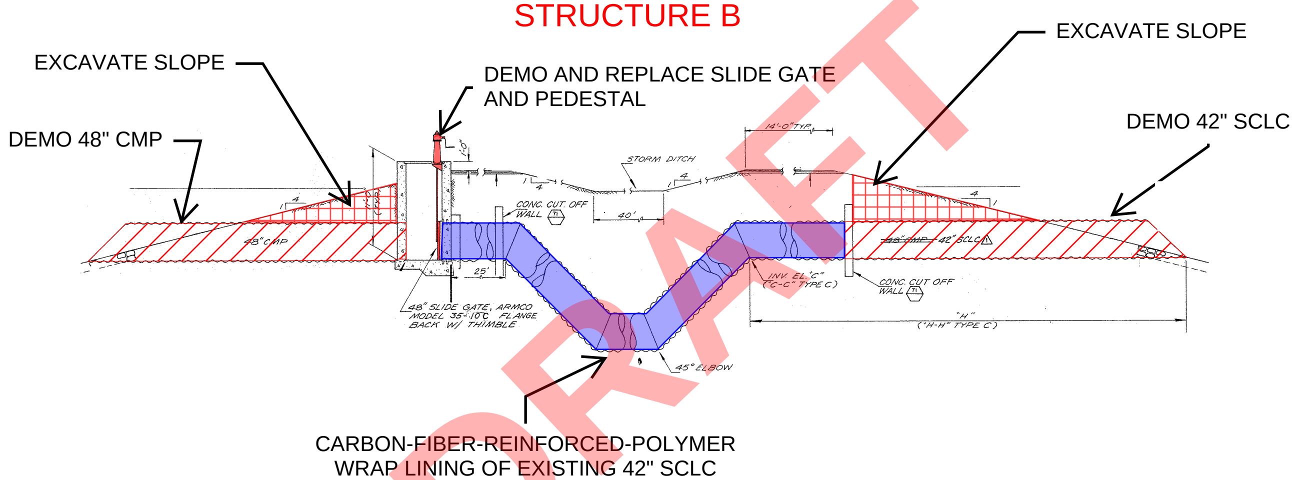
TRANSFER STRUCTURE REPLACEMENT PLAN & SCHEDULE

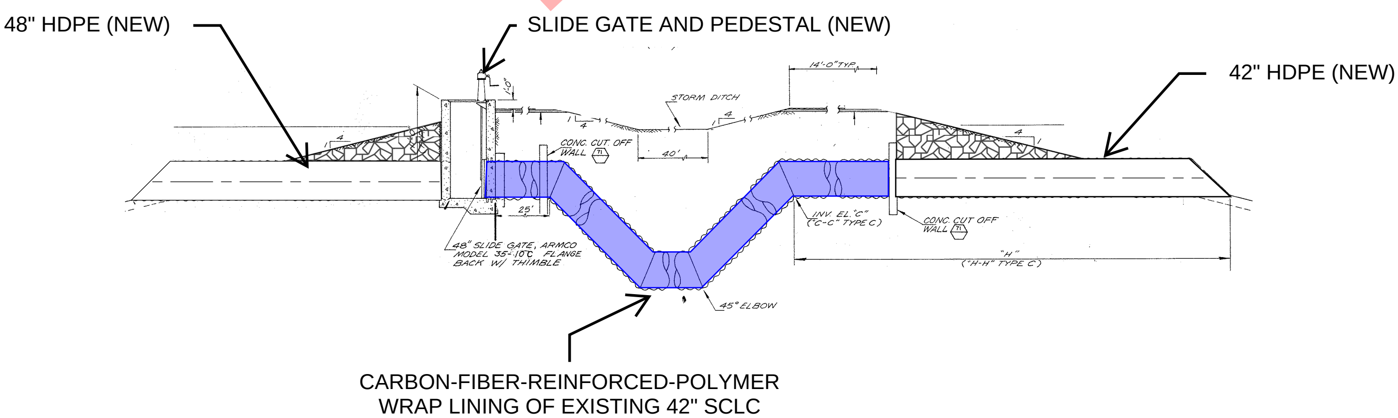


STRUCTURES A-1 AND A-9

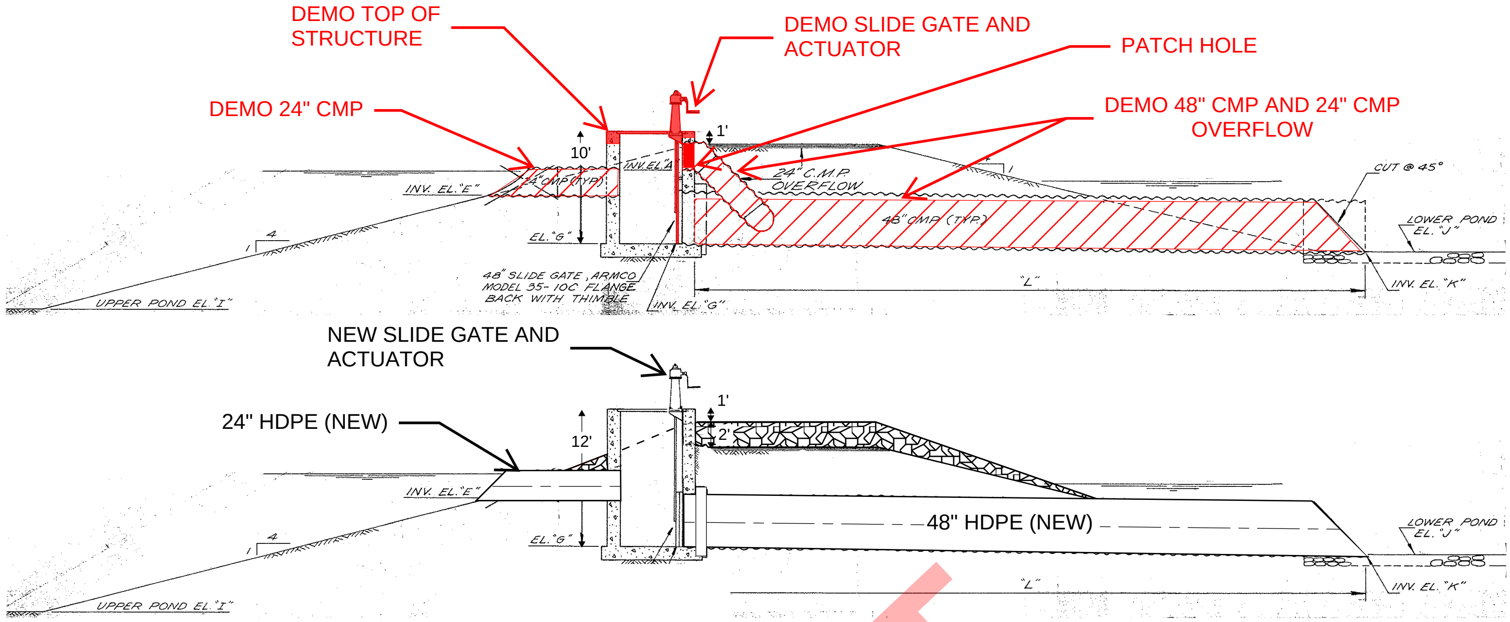


STRUCTURE B

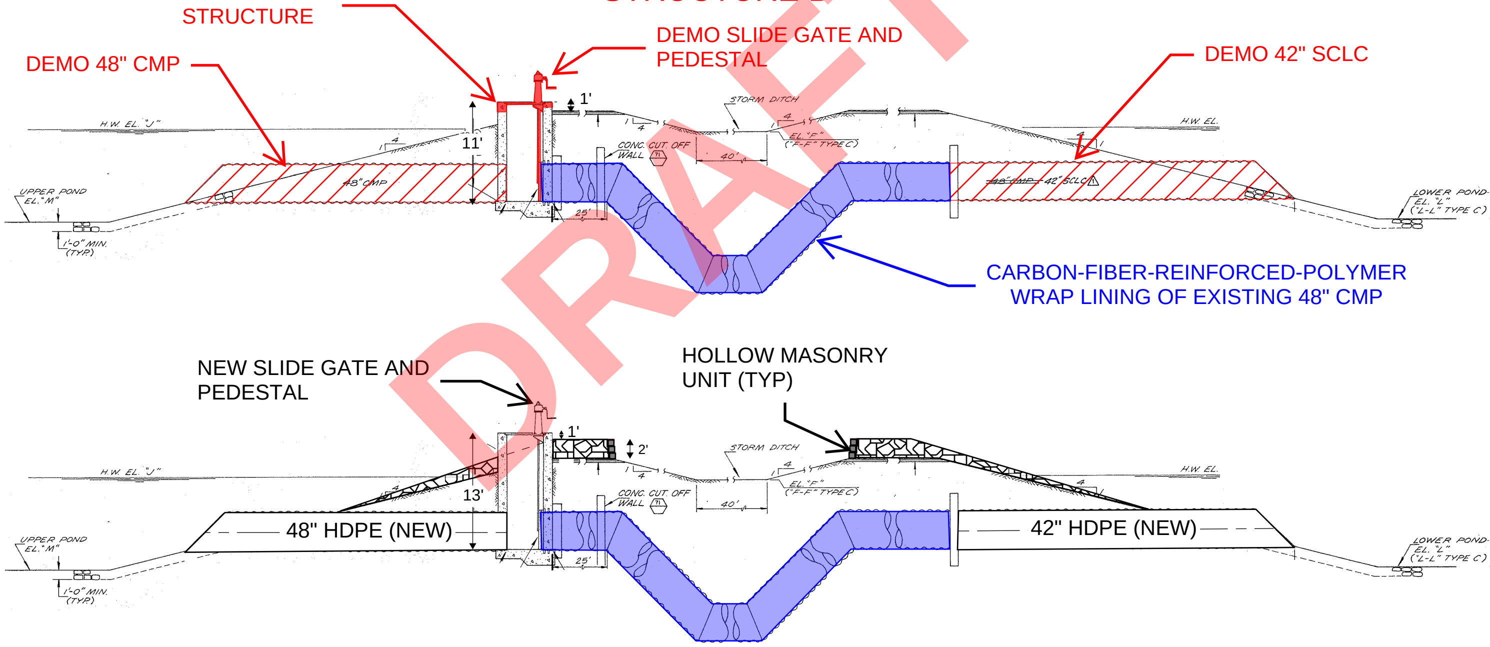




STRUCTURES A-1 AND A-9



STRUCTURE B



APPENDIX F

Site-Specific Ground Motion Hazard Analysis

June 7, 2023

Cristiano Melo, PE, GE
BSK Associates
399 Lindbergh Avenue,
Livermore, CA 94551

Re: ECWRF Oxidation Ponds
Petaluma, CA
Site-Specific Ground Motions

Dear Cristiano,

At your request, I have developed site-specific ground motions for this site in accordance with the provisions of Chapter 21 of ASCE 7-16. Based on the shear wave velocity measurements made at the site using seismic cone penetration tests, the site is classified as Site Class D.

The site is located in Sonoma County, CA with representative co-ordinates being latitude 38.2221 and longitude -122.5681. The site lies in an area of active seismicity and is close to the Rogers Creek – Healdsburg fault.

In order to obtain site-specific ground motions for this site, I compared the appropriate probabilistic and deterministic response spectra for Site Class D. I obtained the 2475-year return period probabilistic spectrum using the USGS web site https://earthquake.usgs.gov/hazards/interactive/. Details of the results for this location are reproduced in Appendix A. The deterministic spectra were obtained using a magnitude 7.22 earthquake on the Rogers Creek – Healdsburg fault at a distance of 5.16 km. This magnitude and distance were obtained from the de-aggregation of the seismic hazard on the USGS site. I applied equal weighting of four of the five ground motion prediction equations (GMPEs) (excluding that of Idriss) using the NGAWest2 spreadsheet which is downloadable from https://peer.berkeley.edu/peer-nga-west2-research-program-releases-excel-file-five-horizontal-ground-motion-prediction. Risk adjustment factors were obtained from the SEA/OSHPD web site https://seismicmaps.org/ and the adjustment to “maximum direction” spectra was made using the factors suggested by Shahi and Baker (2014). The results are shown in Figure 1.

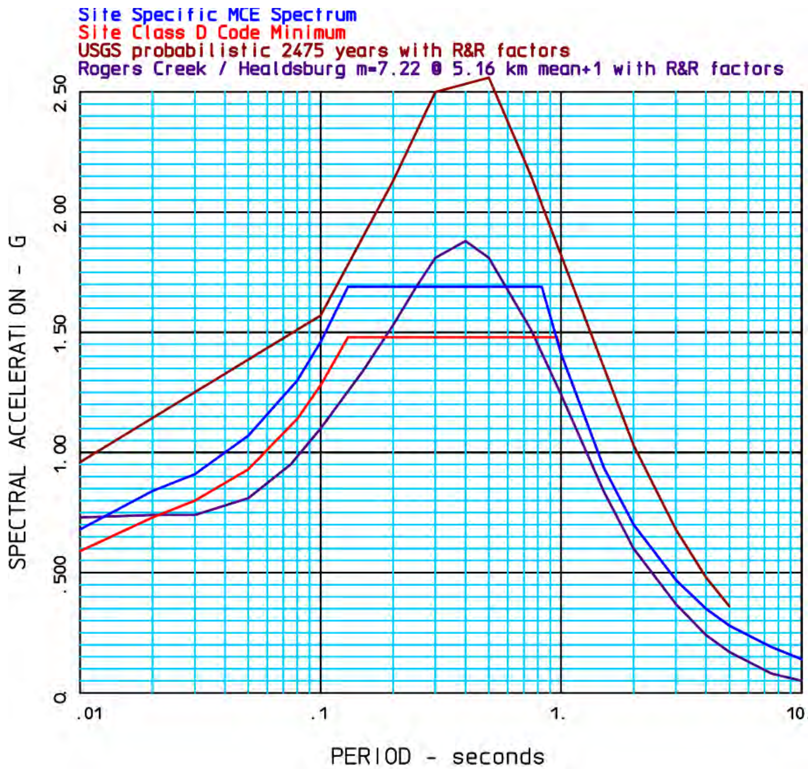


Figure 1 – MCE_R Ground Surface Response Spectra

As expected, the Rogers Creek - Healdsburg fault deterministic spectrum falls below the probabilistic spectrum, and therefore governs.

Parameters for the standard code spectra for this site, obtained from the SEA/OSHPD web site, are shown in the Appendix. The code minimum spectrum, which is 80% of the standard code spectrum, is also shown in Figure 1. In accordance with Section 21.3 and 21.4 of ASCE 7-16, the site-specific design spectrum has a flat top with a spectral acceleration that is 90% of the peak spectral acceleration of the governing site-specific spectrum. The longer period arm of the design spectrum would normally be based the spectral acceleration at a period of 1 second or greater, but in this case these values fall below the code minimum values, so that the recommended design spectrum follows the code minimum spectrum at longer periods. Figure 1 and Table 1 show only the MCE_R values, but the design values are simply two-thirds of these values.

The values of the parameters S_{Ms} and S_{M1} are 1.69 g and 1.41 g and the values for S_{Ds} and S_{D1} are 1.13 g and 0.94 g.

PERIOD	Sa
seconds	[g]
0.01	0.68
0.02	0.84
0.03	0.91
0.05	1.07
0.08	1.30
0.1	1.46
0.13	1.69
0.2	1.69
0.35	1.69
0.5	1.69
0.6	1.69
0.75	1.69
0.83	1.69
1	1.41
1.5	0.94
2	0.70
3	0.47
4	0.35
5	0.28
7.5	0.19
10	0.14

Table 1 – Recommended MCE_R Spectrum

I would be happy to address any questions that you or the structural engineer might have.

Sincerely,

Robert Pyke Ph.D., G.E.

Attachments:

Appendix – Outputs from SEA/OSHPD and USGS

Reference:

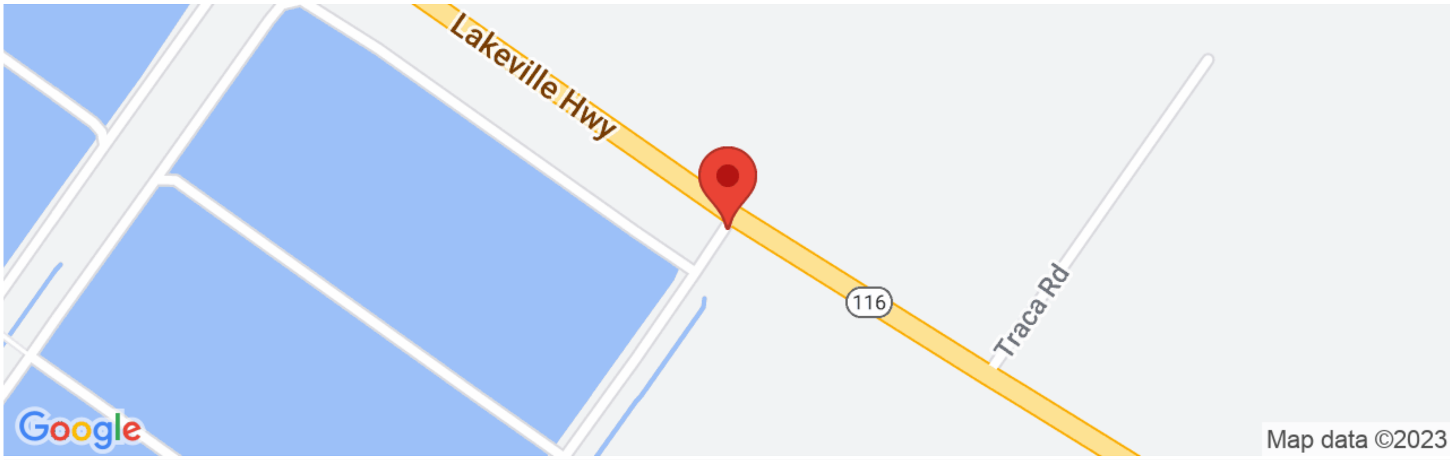
Shahi, S.K. and Baker, J.W., "NGA-West 2 Models for Ground Motion Directionality", Earthquake Spectra, Volume 30, No. 3, August 2014

Appendix

Outputs from SEA/OSHPD web site for Site Class D
and from USGS Seismic Hazard Tool

Petaluma Oxidation Ponds

Latitude, Longitude: 38.222148, -122.568094



Date	6/6/2023, 3:14:17 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	**Value**	**Description**
S_S	1.847	MCE_R ground motion. (for 0.2 second period)
S_1	0.704	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.847	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.231	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	**Value**	**Description**
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.777	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.854	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.12	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.361	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.847	Factored deterministic acceleration value. (0.2 second)
S1RT	0.816	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.913	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.704	Factored deterministic acceleration value. (1.0 second)
PGAd	0.777	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.918	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.898	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.893	Mapped value of the risk coefficient at a period of 1 s
C_V	1.469	Vertical coefficient

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u...

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

38.2221

Time Horizon

Return period in years

2475

Longitude

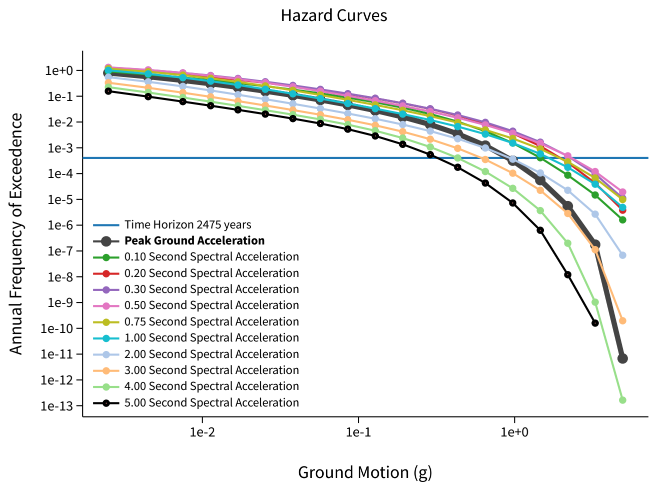
Decimal degrees, negative values for western longitudes

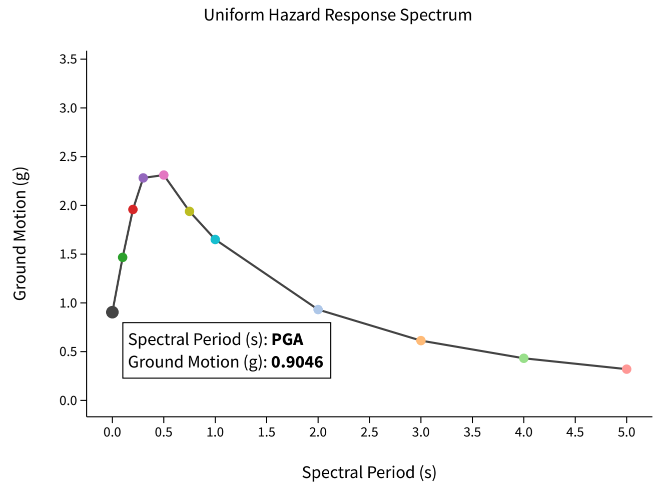
-122.5681

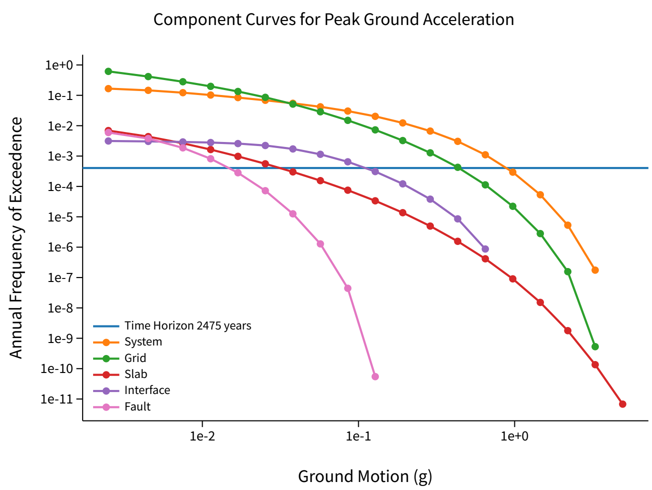
Site Class

259 m/s (Site class D)

Hazard Curve





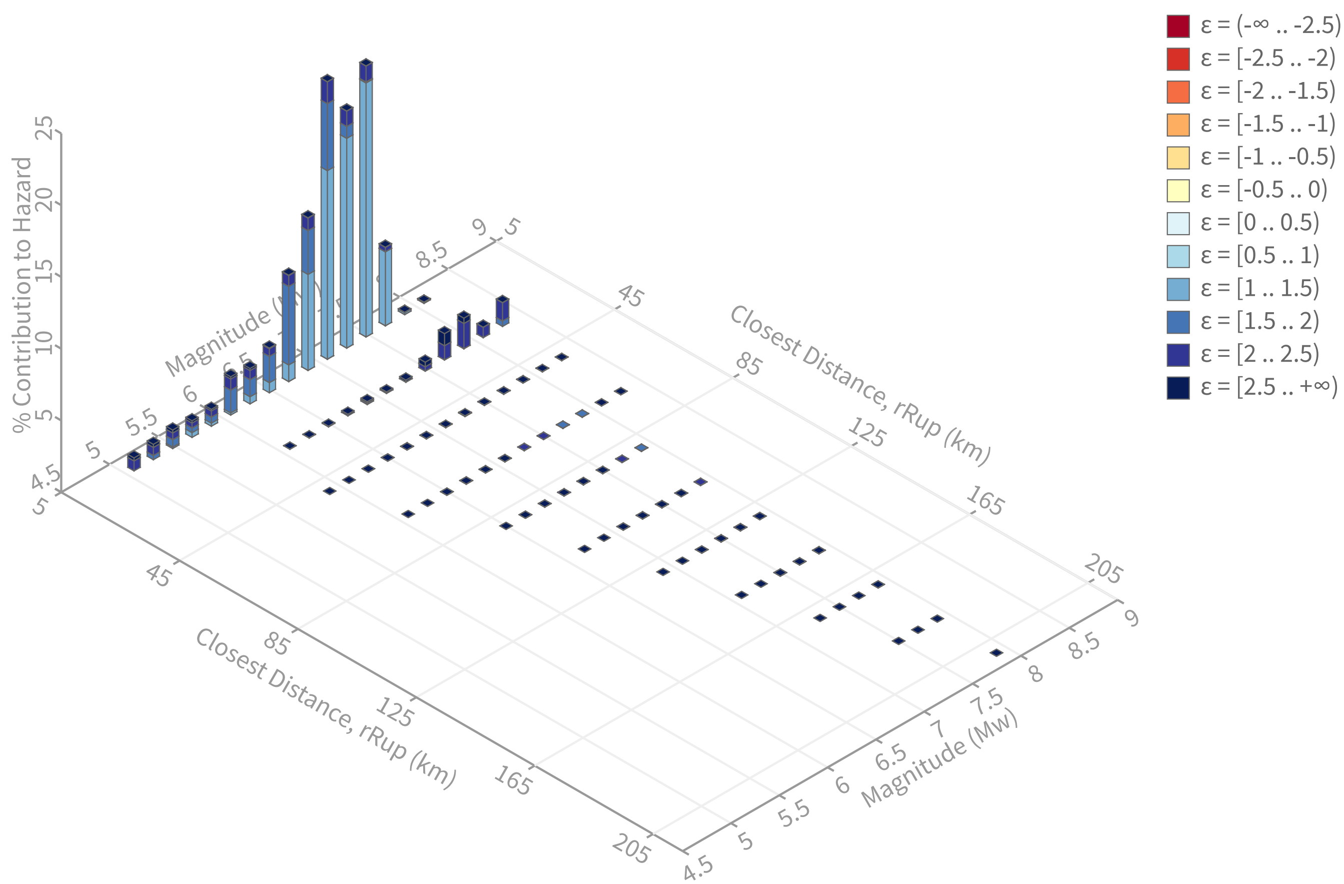


View Raw Data

Deaggregation

Component

Total


% Contribution to Hazard
Magnitude (Mw)
Closest Distance, rRup (km)
Closest Distance, rRup (km)
Magnitude (Mw)
ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.90464882 g

Recovered targets

Return period: 3134.8346 yrs
Exceedance rate: 0.00031899609 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.16 %

Mean (over all sources)

m: 7.1
r: 7.46 km
ε_0: 1.55 σ

Mode (largest m-r bin)

m: 7.11
r: 5.39 km
ε_0: 1.43 σ
Contribution: 19.33 %

Mode (largest m-r-ε_0 bin)

m: 7.51
r: 5.17 km
ε_0: 1.23 σ
Contribution: 17.69 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							46.98
Rodgers Creek - Healdsburg [2]		5.16	7.22	1.37	122.522°W	38.250°N	52.08	35.64
Bennett Valley [0]		7.07	6.49	1.85	122.503°W	38.259°N	54.15	3.41
San Andreas (North Coast) [4]		27.84	7.94	2.33	122.817°W	38.066°N	231.55	3.08
Rodgers Creek - Healdsburg [1]		5.30	7.04	1.45	122.522°W	38.251°N	52.11	1.89
UC33brAvg_FM31	System							46.11
Rodgers Creek - Healdsburg [2]		5.16	7.23	1.37	122.522°W	38.250°N	52.08	35.54
San Andreas (North Coast) [4]		27.84	7.94	2.33	122.817°W	38.066°N	231.55	3.07
Bennett Valley [0]		7.07	6.48	1.85	122.503°W	38.259°N	54.15	3.06
Rodgers Creek - Healdsburg [1]		5.30	7.05	1.45	122.522°W	38.251°N	52.11	1.79
UC33brAvg_FM31 (opt)	Grid							3.44
UC33brAvg_FM32 (opt)	Grid							3.44

APPENDIX G

Important Information About This Geotechnical-Engineering Report

Important Information about This

Geotechnical-Engineering Report

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

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- for a different client;
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- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
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Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

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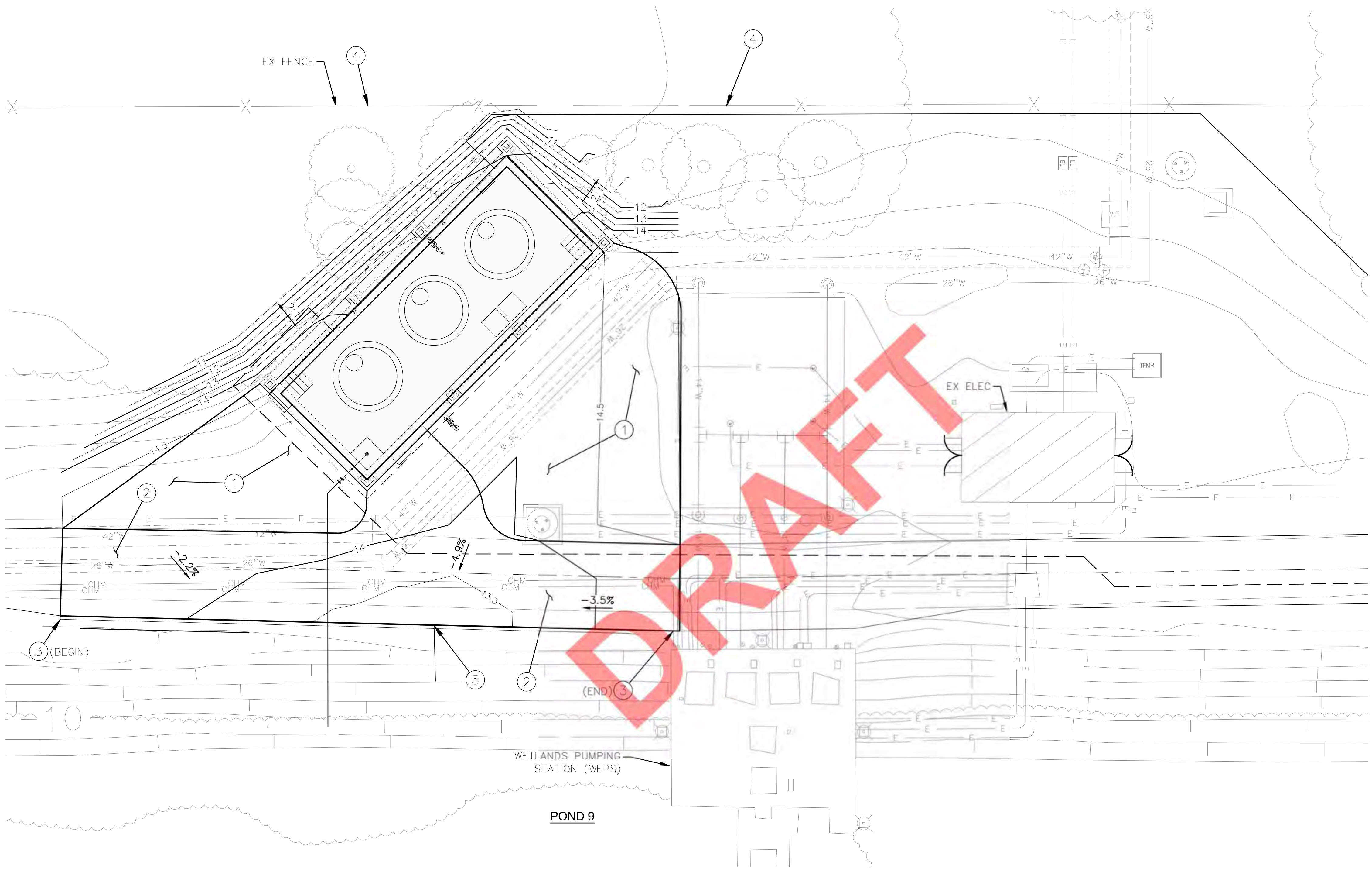
While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the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

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B

Conceptual Grading Plans for New Sodium Hypochlorite Storage Tanks



PLAN
SCALE: 1"=10'

CONSTRUCTION LEGEND

1. X" AC PAVEMENT OVER X" AGGREGATE BASE
2. X" PCC PAVEMENT OVER X" AGGREGATE BASE
3. SEE DETAIL X ON SHEET C-X FOR PCC CURB
4. REMOVE AND REPLACE APPROXIMATELY XX FL OF EXISTING FENCE
5. SEE DETAIL X ON SHEET C-X FOR CURB CORE PIPE OUTLET

NOTES:

1. FOR CONTINUATION OF PIPING SEE CIVIL SHEETS.
2. 5,000 GAL TANKER TRUCK VOLUME CONTAINED WITHIN SURFACE OF NEW CURB AND PAVEMENT AREA

GRAPHIC SCALE
SCALE: 1" = 10'

30% DESIGN SUBMITTAL
DATE: May 10, 2023
NOT FOR CONSTRUCTION

DUDEK
605 Third Street
Encinitas, CA
92024 760.942.5147

DATE: 05/XX/2023
DESIGNED BY: A. HESS
DRAWN BY: N. HUNTER
CHECKED BY: P. GIORI

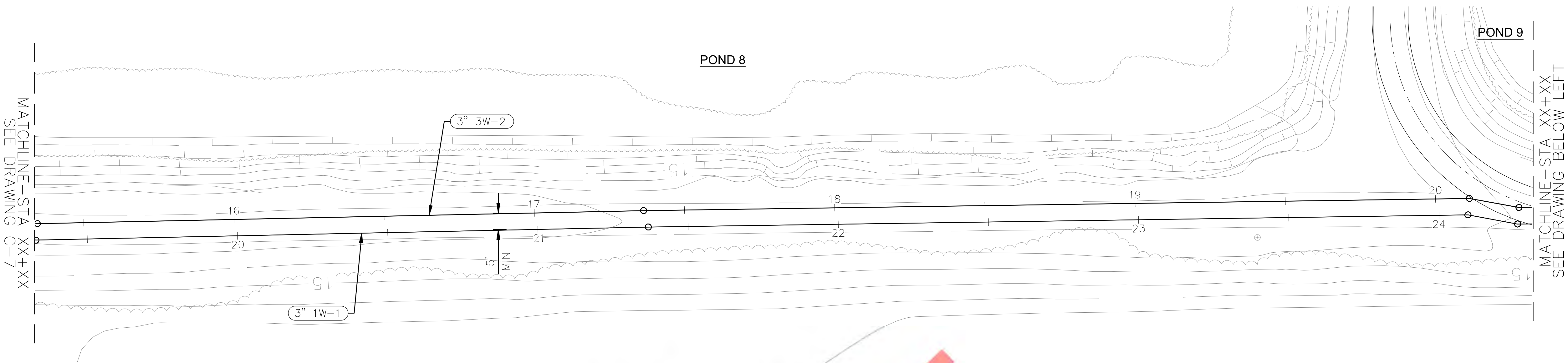
PROJECT NO. CXXXXXXXXXX

CITY OF PETALUMA
PUBLIC WORKS & UTILITIES
202 N. McDowell Blvd., PETALUMA, CALIFORNIA, 94954
PH. 707-778-4546 FAX. 707-778-4508

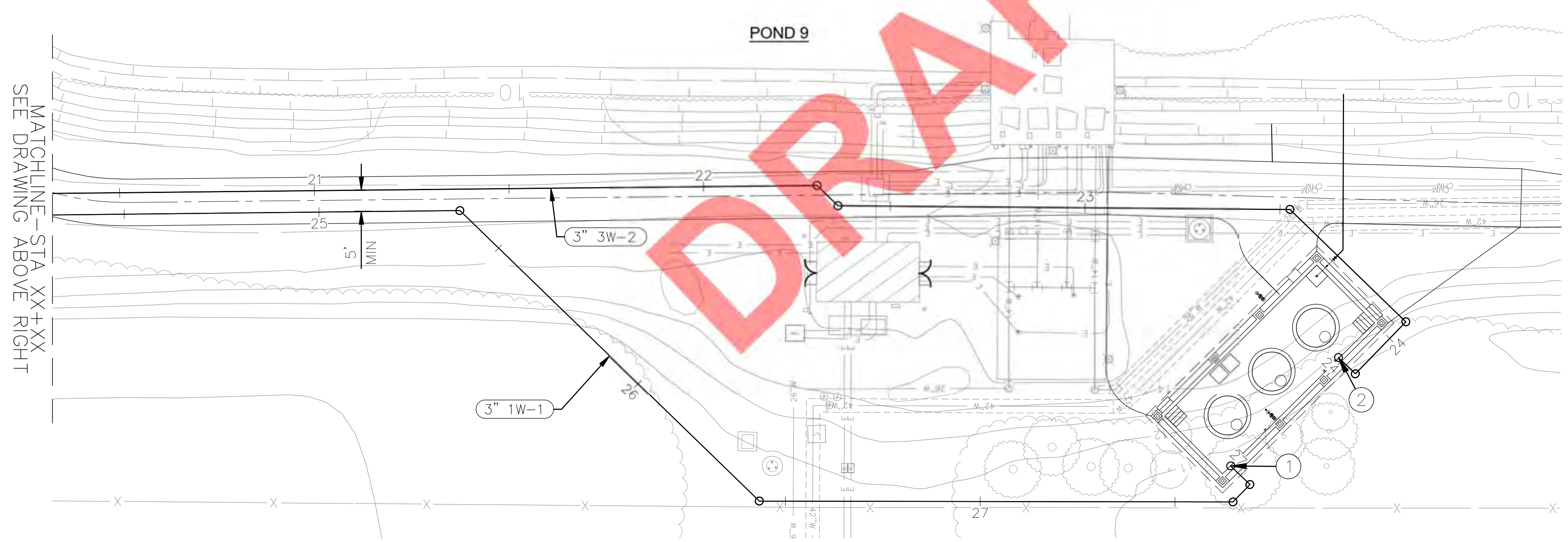
SODIUM HYPOCHLORITE SYSTEM REPLACEMENT AND RELOCATION
CIVIL
CHEMICAL STORAGE GRADING PLAN

SHEET C-5
X OF XX

34"x22" ORIGINAL SCALE IN INCHES



PLAN
SCALE: 1"=20'



PLAN
SCALE: 1"=20'

CONSTRUCTION LEGEND

1. 1W POINT OF CONNECTION. SEE SHEET M–2
2. 3W POINT OF CONNECTION. SEE SHEET M–2
3.
4.

NOTES:

1. FOR SITE IMPROVEMENTS SEE SHEET C–X.
2. PIPE SCHEDULE ON SHEET G–3

GRAPHIC SCALE
0 20' 40'
SCALE: 1" = 20'

30% DESIGN SUBMITTAL
DATE: May 10, 2023
NOT FOR CONSTRUCTION

DUDEK
605 Third Street
Encinitas, CA
92024 760.942.5147

DATE: 05/XX/2023
DESIGNED BY: A. HESS
DRAWN BY: N. HUNTER
CHECKED BY: P. GIORI

PROJECT NO.
CXXXXXXXXXX

REGISTERED PROFESSIONAL ENGINEER PHILLIP J. GIORI No. 87516 CIVIL STATE OF CALIFORNIA

CITY OF PETALUMA
PUBLIC WORKS & UTILITIES
202 N. McDowell Blvd., PETALUMA, CALIFORNIA, 94954
PH. 707-778-4546 FAX. 707-778-4508

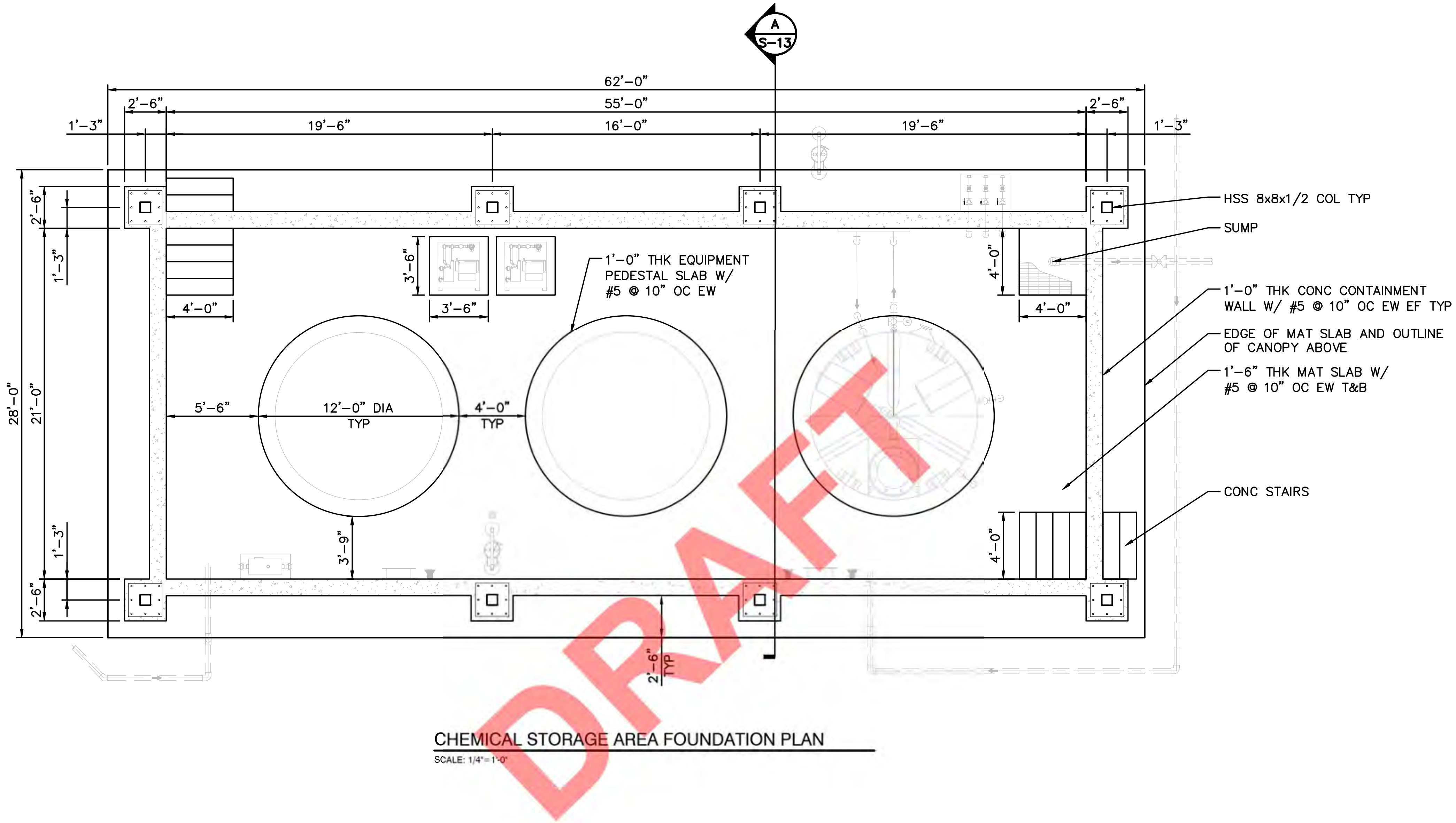
PETALUMA 1858

SODIUM HYPOCHLORITE SYSTEM REPLACEMENT AND RELOCATION
CIVIL
YARD PIPING PLANS - 3

SHEET
C-8
X OF XX

3/4"x22" ORIGINAL SCALE IN INCHES

P:\101-Engineering\Petaluma, City of\15219_ECWRF Chemical System Upgrade\6-CAD\15219 SHxx C-06 to C-08 pipe.dwg 05/10/2023 21:59



CHEMICAL STORAGE AREA FOUNDATION PLAN

SCALE: 1/4"=1'-0"

30% DESIGN SUBMITTAL

DATE: MAY 11, 2023

NOT FOR CONSTRUCTION

KELSEY STRUCTURAL

DATE: 05/09/2023
DESIGNED BY: MS
DRAWN BY: AM
CHECKED BY: DI

PROJECT NO. CXXXXXXXXXX

CITY OF PETALUMA

PUBLIC WORKS & UTILITIES

202 N. McDowell Blvd., PETALUMA, CALIFORNIA, 94954

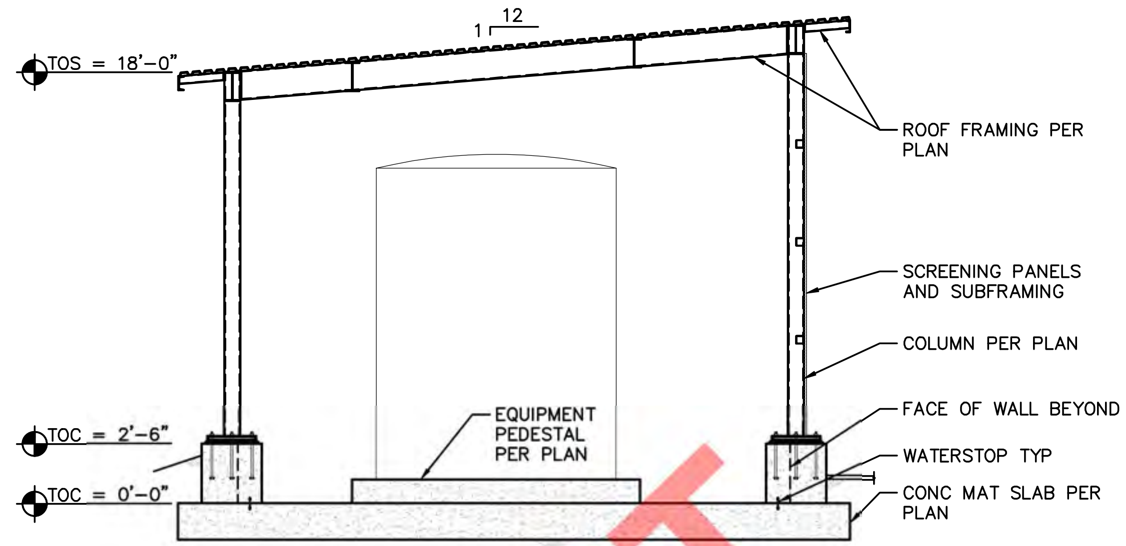
PH. 707-778-4546 FAX. 707-778-4508

SODIUM HYPOCHLORITE SYSTEM REPLACEMENT AND RELOCATION

CHEMICAL STORAGE AREA FOUNDATION PLAN

SHEET S - 11 OF XX

34"x22" ORIGINAL SCALE IN INCHES



CHEMICAL STORAGE AREA SECTION

SCALE: 1/4"=1'-0"

A
S-11

DRAFT

30% DESIGN SUBMITTAL
DATE: MAY 11, 2023
NOT FOR CONSTRUCTION

KELSEY STRUCTURAL

DATE: 05/09/2023
DESIGNED BY: MS
DRAWN BY: AM
CHECKED BY: DI

PROJECT NO.
CXXXXXXXXXX

CITY OF PETALUMA
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SODIUM HYPOCHLORITE SYSTEM REPLACEMENT AND RELOCATION

CHEMICAL STORAGE AREA SECTIONS

SHEET
S - 13
OF XX

34"x22" ORIGINAL SCALE IN INCHES

APPENDIX C

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Important Information about This

Geotechnical-Engineering Report

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GEOPROFESSIONAL BUSINESS ASSOCIATION

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e-mail: info@geoprofessional.org www.geoprofessional.org

ATTACHMENT D

City of Petaluma
1318 Redwood Way Suite 120, Petaluma, CA 94954
707-778-4546

ELLIS CREEK CHEMICAL SYSTEM UPGRADE PROJECT – PHASE 1
CITY PROJECT NO: C66501840

PRE-BID MEETING SIGN IN SHEET

Date: June 20, 2024 **Time: 10:00am**

Name	Organization	Role	Contact No.	Email	Signature
Steve Worrell	City of Petaluma	Project Manager	707-776-3608	sworrell@cityofpetaluma.org	
VANESSA SILLMAN	GBI	Estimator	415-720-0486	vanessas@gbi1914.com	[signature]
BILL BAREND	VALENTINE CORP	Sr PM	415 716 9572	bbarend@valentinecorp.com cwalsh@valentinecorp.com	[signature]
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Ian Prindle	Aqua Poppy	estimator	530-848-2260	cprindle@aquapoppy.com	[signature]
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