

**ALASKA BASIN BULKHEAD  
ALAMEDA, CALIFORNIA**

**SUPPLEMENTAL GEOTECHNICAL EXPLORATION**

**Submitted to:**

Mike O'Hara  
STL Company, LLC  
3300 Douglas Boulevard, Suite 450  
Roseville, California 95661

**Prepared by:**

ENGEO Incorporated

December 6, 2016  
Revised November 15, 2018

**Project No:**

9769.000.001

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December 6, 2016  
Revised November 15, 2018

Mr. Mike O'Hara  
STL Company, LLC  
c/o Tim Lewis Communities  
3300 Douglas Boulevard, Suite 450  
Roseville, CA 95661

Subject: Alaska Basin Bulkhead  
1501 Buena Vista Ave  
Alameda, California

## GEOTECHNICAL EXPLORATION

Dear Mr. O'Hara:

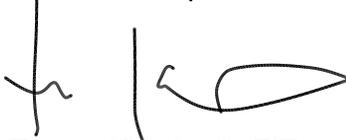
We prepared this supplemental geotechnical exploration for the Alaska Basin bulkhead as outlined in our agreement dated October 31, 2016. We previously prepared a geotechnical exploration report dated October 23, 2014 and revised November 24, 2014 for this project. We reviewed the previous geotechnical data presented in the referenced report and performed further investigation of the subsurface closer to the bulkhead to provide the enclosed updated geotechnical recommendations.

Based on our review, the main geotechnical concerns regarding the existing bulkhead are (1) liquefaction and lateral spreading potential of native sand deposits and (2) potential slope instability along the shoreline. Our report addresses these concerns and provides recommendations regarding mitigation, as necessary.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated



Teresa Klotzback, EIT  
tk/jf/bvv



Jeff Fippin, GE



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## 1.0 INTRODUCTION

### 1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for evaluation of the existing Alaska Basin bulkhead in Alameda, California. The purpose of this report is to analyze the stability of the shoreline behind the bulkhead and to provide geotechnical recommendations. Our authorized scope included the following:

- Site reconnaissance and review of available literature
- Subsurface exploration
- Data analysis and conclusions
- Report preparation

This report was prepared for the exclusive use of STL Company, LLC and their consultants for design of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to determine whether modifications are necessary.

### 1.2 PROJECT LOCATION

The project site is located at 1501 Buena Vista Avenue in Alameda, California; Figure 1 displays a Site Vicinity Map. The subject project site is shown as Parcel Number 72-383-4 on the Alameda County Assessor's Parcel Map. The existing bulkhead is located along the shoreline of an inlet known as Alaska Basin. The bulkhead is approximately 65 feet from the northern property boundary and approximately 150 feet from the existing warehouse.

The adjacent Del Monte Warehouse property consists of an active warehouse building, office space with paved areas located west and north of the building; the majority of the site is occupied by the existing buildings. The site is bordered on the northeast by the Encinal Terminals site. The site is also bordered on the north by the Alaska Basin and a commercial business park with associated parking lots. The site is bordered on the northwest and south by residential buildings and Littlejohn Park. A Site Plan is provided as Figure 2.

### 1.3 PROJECT DESCRIPTION

We understand that the proposed development plan includes Clement Avenue running parallel to the existing Bulkhead along Alaska Basin. This road is proposed to be located on Encinal Terminal and Del Monte Warehouse Properties. Currently the plan for the Del Monte Property is to redevelop the existing warehouse building by adding additional floors within the existing masonry facade. The building will be redeveloped with a mix of residential and commercial uses.

The bulkhead is approximately 230 feet long and comprises steel sheet piles with an approximate thickness of 3/8-inch and it is supported with tiebacks. There are reportedly two eras of construction. The western portion of the wall was constructed in 1973 while the eastern portion is older and assumed to be constructed approximately 60 years ago. The older portion of the wall comprises U-sheets and has a tieback with a timber wale approximately 3 to 4 feet above high water while the western side of the wall comprises "Z" shaped sheets with a steel

wale slightly above the high water line. The wall has a noticeable bulge out to the water east of the mid-point of the wall. The tiebacks are HP 8x36 sections, approximately 76-feet long, driven at a 15-degree inclination from horizontal. The anchors are attached to the whaler with a 1¼-inch-diameter stressing rod attached to the end of the tieback. According to the as-built plans, the anchors are spaced either 6.25-feet or 9.5-feet on center, depending on location.

A seawall conditions report by Moffatt & Nichol dated September 21, 2015, indicates the steel is in fair condition. Some repairs are recommended by Moffatt & Nichol to extend the life of the structure in areas of steel section loss.

## 2.0 FINDINGS

### 2.1 FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration included advancing two Cone Penetration Tests (CPTs) to depths of roughly 50 feet below the ground surface. Figure 2 shows the approximate CPT locations. The CPT results are presented in Appendix A.

To measure soil gradation, plasticity and moisture content, we tested push and drive samples recovered during CPT testing. A summary of laboratory testing and test methods as well as test results are presented in Appendix B.

We previously drilled two borings along the alignment of the future Clement Avenue in 2014. The locations of these borings are shown on Figure 2.

### 2.2 GEOLOGY AND SEISMICITY

We evaluated the regional and local geology and seismicity as part of this investigation. Our evaluation was based on our review of published reports, our experience in the project area, and the results of the subsurface investigation.

#### 2.2.1 Regional Geology

The San Francisco Bay Valley and the peripheral hill system that encloses it make up the main geological features of the bay region. The bay region includes two main fault structures, the San Andreas and Hayward rift zones. Diverse crustal movements within this system control the morphology and structural stability of the area.

Because of its proximity to the Pacific Ocean, the Bay Area's hydrologic, and thus, sedimentary, conditions are dominated by relative sea level fluctuations and changes in the rate of precipitation. The Bay Area has experienced four episodes of intense erosion followed by four periods of massive deposition in recent geologic history. This process has resulted in the removal of large amounts of bedrock and subsequent covering by Pleistocene sediments to considerable depths. We are currently in an interglacial period in which the earth is warming. During this warming period, relative sea level has risen and heavy sedimentation has occurred in the bay valley (the well-documented Young Bay Mud).

The Bay Area can thus be described as a region formed by depositional and erosional cycles with stratigraphic beds that increase in age with depth. The youngest deposits should be expected to be soft and unconsolidated, while the older horizons will be more indurated due to

overburden pressure and severe in-situ weathering. The site is situated on Quaternary Beach and Dune Sand (Qs), as mapped by Graymer (2006).

### 2.2.2 Seismicity

Numerous small earthquakes occur every year in the San Francisco Bay Region and larger earthquakes have been recorded and can be expected to occur in the future. Figure 3 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region. The most common nearby active faults within 25 miles of the site and their estimated maximum earthquake magnitudes, based on the United States Geologic Survey (USGS) 2008 National Seismic Hazard Maps, are provided in the following table. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart and Bryant, 1997).

**TABLE 2.2.2-1: Regional Faults**

Fault Name	Approximate Distance (miles)	Estimate of Maximum Magnitude (Ellsworth)
Hayward (South)-Rodgers Creek	3.8	7.3
Hayward (North)-Rodgers Creek	4.0	7.1
Mount Diablo Thrust	13.6	6.6
Calaveras	13.8	7.0
North San Andreas	14.2	7.9
Green Valley Connected	16.8	6.8
San Gregorio Connected	18.7	7.5
Greenville Connected	24.2	6.9

The Working Group on California Earthquake Probabilities (WGCEP, 2014) estimates there is a 33 percent probability that a moment magnitude ( $M_w$ ) of 6.7 or greater earthquake will occur on the Hayward fault within 30 years of the publish date (2014 to 2044). Likewise, WGCEP estimates a 72 percent probability of a similarly sized earthquake in the San Francisco Bay Area, as a whole, in this same timeframe.

## 2.3 SUBSURFACE CONDITIONS

Below the asphalt concrete, our CPTs encountered behavior consistent with clay in the upper 4 to 8 feet. A thick layer of medium dense silty sand was encountered from approximately 8 feet to 24 feet below existing grade. Laboratory tests on samples that we collected from this layer indicate low plasticity, medium to high fines content material. Underlying this silty sand layer was dense to very dense silty sand from approximately 24 feet to 48 feet below ground surface. From 48 feet to the final depth of 52 feet below ground surface, the CPTs encountered stiff clay.

Our previously performed borings to the south encountered similar conditions in the upper approximately 10 feet to the CPTs. The soil conditions at the boring locations indicate stiff clay from approximately 10 to 24 feet underlain by dense to very dense sand over very stiff clay.

Our exploration locations are presented on the Site Plan (Figure 2), and the specific stratigraphy for location is depicted on the CPT results in Appendix A.

## 3.0 CONCLUSIONS

We analyzed the liquefaction susceptibility of the bulkhead shoreline using methods developed by Idriss and Boulanger (2008). Based on this analysis, some of the soil in the upper 24 feet behind the bulkhead is potentially liquefiable. We recommend further analysis of the structural integrity of the actual bulkhead using pressures associated with liquefied soil and seismic loading. If the bulkhead cannot support these loads, remedial alternatives will be needed to stabilize the shoreline.

### 3.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a design earthquake include ground rupture (surface faulting), soil liquefaction and its associated effects, lateral spreading and landslides. Liquefaction-induced lateral spreading and stability of the shoreline under seismic loading are the primary seismic hazards that could affect the bulkhead shoreline. The following sections present a discussion of these hazards as they apply to the site.

#### 3.1.1 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore water pressure build-up under the reversing cyclic shear stresses associated with earthquakes. Our borings encountered a layer of silty sand at approximately 8 feet to 24 feet below ground surface; this sand layer has a potential of liquefaction under seismic loading.

To assess liquefaction potential, we performed liquefaction analyses utilizing data obtained from the two CPT probes advanced as part of the current field exploration. We assigned a design groundwater level of 5 feet below the existing ground surface, a peak ground acceleration (PGA) of 0.62g, and a Moment Magnitude ( $M_w$ ) of 7.3 contributed by the Hayward fault; these values are based on the 2016 California Building Code and the commonly accepted potential earthquake magnitude of the closest faults. While there is liquefaction at the site, since the analysis herein is governed by accelerations at short periods, we are conservatively using the mapped PGA for a Site Class D our experience indicates that a site-specific site-response analysis would have a lower PGA considering the lower shear wave velocity of a liquefied soil; if necessary for final design, a site-response analysis can be performed at during design of mitigation. We performed our analyses using the computer software CLiq Version 1.7 developed by GeoLogismiki, using methods developed by Idriss and Boulanger (2008).

Our analysis indicates that the silty sand layer between a depth of 8 and 24 feet is potentially liquefiable. We include our liquefaction calculations in Appendix C of this report.

#### 3.1.2 Lateral Spreading

Lateral spreading is a failure within weak soil, typically due to liquefaction, which causes a soil mass to move toward a free face or down a gentle slope. As discussed above, there is a potential for liquefaction of the silty sand layer underlying the shallow clay. If the bulkhead were to fail leaving a free face at the side of the Alaska Basin, then this layer has the potential to

cause lateral spreading leading to large deformations of the land behind the bulkhead and under the proposed improvements along the planned future Clement Avenue extension. **Therefore, we recommend that either the bulkhead's structural integrity be analyzed and the bulkhead maintained to remain intact during seismic loading, or alternative remediation, as discussed in Section 4.0, be constructed to contain the lateral spreading.**

### 3.1.3 Lateral Earth Pressures on the Existing Bulkhead

We evaluated active pressures using the procedures shown in the National Cooperative Highway Research Program (NCHRP) Report 611, "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments." We determined combined active and seismic loading using the GLE method (Section 7.4 of the NCHRP). In performing this analysis, we modified our previously performed slope stability analysis to include the subsurface conditions encountered in our recent CPTs. We also included an additional 1 foot of fill in our revised analysis to model the planed grade change based on discussions with you on December 9, 2016. To model the earthquake and earth pressure on the bulkhead, we removed all soil in our slope stability model on the passive side of the wall down to the bottom of the liquefied sand layer. We applied a horizontal boundary force at the location of the resultant force of active and dynamic earth pressures to determine the force that results in a factor of safety of 1 with a pseudostatic coefficient applied. We selected a pseudostatic coefficient of 0.28 based on an approximate displacement of 1 to 2 inches and the methodology in NCHRP 611 for estimating slope displacement. The pseudostatic coefficients are associated with the building code Maximum Considered Earthquake (MCE), as the critical failure surfaces extend below the adjacent warehouse and are related to liquefaction. We modeled the liquefied sand using a residual strength coefficient of critical strength divided by effective stress of 0.25 consistent with correlations by Stark and Mesri (1992). We assumed a horizontal load inclination neglecting friction between the wall and backfill. This force represents the total kinematic, static, and inertial earth load on the wall to achieve stability at the deformation amount assumed. The results of this analysis are shown in Appendix D.

We developed passive pressures using a variant of the GLE method. We determined passive pressures by removing soil on the active side of the wall. We applied a triangularly distributed pressure that pushes into the soil at the wall location to determine the force needed to achieve a factor of safety of 1.0. We began this triangular force at a value of 0 at the top of the dense sand layer underlying the liquefiable sand layer and terminated the force at the bottom of the wall. This triangular distributed pressure represents the ultimate passive pressure to be used for evaluation of the structural integrity of the seawall. The results of this analysis are also shown in Appendix D.

Based on the liquefied soil, we estimate that the maximum capacity of the tiebacks are approximately 70 kips each.

Figure 4 graphically shows our recommended active, earthquake and passive pressures on the bulkhead for analysis. Based on coordination with the Marine Structural Engineer, Moffatt & Nichol, the pressures on the wall overstress the sheet piles and the anchors. Therefore, we recommend buttressing the liquefiable soil behind the bulkhead to reduce seismic loading on the structure as described in Section 3.3.

### 3.1.4 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the 2016 CBC requirements, as a minimum. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

### 3.1.5 Ground Rupture

Since no known active faults cross the property, and the site is not located within an Alquist-Priolo Earthquake Fault Zone, it is our opinion that ground rupture is unlikely at the subject property.

## 3.2 GROUNDWATER CONDITIONS

Our explorations indicate that the depth to groundwater is about 6 to 7 feet below existing grade. We used a depth to groundwater of 5 feet for our liquefaction analysis and slope stability modeling. Due to proximity to the Oakland-Alameda Estuary and granular nature of the fill, the groundwater level is likely influenced by tide level.

## 3.3 LIQUEFIABLE SOIL MITIGATION OPTIONS

### 3.3.1 Densification

If the soil can be made non-liquefiable through ground improvement techniques, the loads on the bulkhead wall from an earthquake would be dramatically reduced. Based on our experience, however, many of the typical densification techniques may not be feasible due to potential damage to the bulkhead or tiebacks.

Surface compaction methods such as Rapid Impact Compaction, Deep Dynamic Compaction or Vibro Tamping would likely either not supply enough energy to densify the liquefiable soil to the required depth of 25 feet if energy was controlled to reduce risk of damage to the bulkhead, or potentially damage the bulkhead and tiebacks if larger energy amounts are used.

Vibratory methods such as Vibro Compaction or Direct Power Compaction, which use a vibrating probe may be more successfully in improving the soil. However, due to the spacing of the tiebacks, it may be difficult to perform either of these methods without impacting, and potentially damaging the tiebacks. If one of these methods is used, the ground should be improved for a distance at least 50 feet behind the bulkhead. The tiebacks should be carefully located and the vibratory treatment layout should be developed to minimize potential conflicts with the existing tiebacks. Additionally, we recommend appropriate vibration thresholds be developed by the structural engineer and monitoring be performed to reduce the risk of structural damage to the sheet piles due to ground vibrations.

### 3.3.2 Deep Soil Mixing

Since Deep Soil Mixing (DSM) is planned for the shoreline stabilization at the adjacent Encinal Terminal Project, it would likely be economical to perform ground improvement behind the bulkhead wall. DSM is a method of ground improvement where cement is mixed with the in-situ soil in overlapping vertical columns via one or more mixing augers. The mixing augers are advanced to the target depth, and then wet or dry cement is injected through the auger stems as they continue to rotate and mix the injected matter with the native soil; for the site soil, wet mixing is the most likely optimal method. DSM increases the strength of loose sand.

We recommend creating continuous panels by creating overlapping columns of DSM that extend below the base of the liquefiable soil. Commonly, in liquefiable soil, DSM is performed to create perpendicular walls that form cells. These cells assist in reducing the mobility of potentially liquefiable soil. Due to the existing tiebacks, the walls can only be created perpendicularly to the bulkhead. Therefore, we recommend that they be spaced no more than approximately 2 to 2.5 diameters on center so that the liquefiable soil bridges between columns and they serve to reduce almost all loading on the bulkhead from soil between the columns. Due to the spacing of the columns in the area of closer tieback spacing (approximately 6.25 feet on center), we recommend using 3 to 4-foot-diameter augers so the columns can fit between the H-pile tiebacks. Alternatively, the soil mixing can be performed with jet grouting where a large-diameter column of soil mix can be created from an approximately 4- to 6-inch-diameter hole.

To evaluate the approximate minimum depth and lateral extent of this ground improvement method, we performed a slope stability analysis using the pseudo-static coefficient for 6 inches of deflection. In our analyses, we ignored the presence of the bulkhead and the tiebacks in stabilizing the soil (a conservative assumption). We modeled the DSM as a block assuming a compressive strength of 200 pounds per square inch (psi) which would result in a shear strength of about 100 psi, and a replacement ratio of 30 percent; we ignored the contribution to shear strength by the soil between the soil DSM columns to account for strain incompatibility and liquefied strengths. We then changed the dimensions of the DSM block until achieving at least a factor of safety of 1, which according to the NCHRP method referenced above, would result in a deformation of approximately 6 inches. Based on this analysis, the DSM should extend at least 30 feet back from the back face of the wall and extend to a minimum depth of 35 feet below existing grade.

The DSM should be performed as close as practical to the back of the bulkhead. Prior to beginning DSM, we recommend that geophysical methods, such as magnetometer, be performed by a qualified company to locate and map all tiebacks. The final DSM layout should be developed after each tieback has been appropriately located in the field.

### 3.3.3 Drilled Displacement Columns (DDC)

Another alternative to stabilize soil behind the bulkhead is Drilled Displacement Columns (DDCs). DDCs are constructed by first drilling to a desired depth of improvement with a heavy crowd; the crowd displaces the soil and results in only minimal drill spoils. Once the desired depth is reached, the auger is slowly raised while simultaneously injecting grout under high pressure to form a well-defined cement column. Steel rebar is optionally installed within the column if analysis indicates additional stiffness is needed. DDCs decrease the proportion of loose or soft soil, thereby decreasing the total susceptibility to excessive deformation resulting

from a seismic event or additional loads. DDC has negligible construction vibration and a relatively quiet construction method. The DDC is a displacement corrective treatment method and typically generates spoils volumes that are less than 3 percent of the volume of soil improved. As a preliminary recommendation, we suggest the DDCs be spaced a distance of 2 to 2.5 column diameters apart to be cost effective, and to extend 10 feet below the liquefiable sand layer (approximately 35 feet deep). The DDCs should be constructed in a triangular pattern in approximately four rows (depending on the diameter selected).

As with the DSM solution, The DDCs should be performed as close as practical to the back of the bulkhead. Prior to beginning DDCs, we recommend that geophysical methods, such as magnetometer, be performed by a qualified company to locate and map all tiebacks. The final DDC layout should be developed after each tieback has been appropriately located in the field.

#### **4.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS**

This report presents geotechnical recommendations for design of the bulkhead near the Del Monte Warehouse Property discussed in Section 1.3. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs and if unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include evaluations for excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, we did not include work to determine the existence of possible hazardous materials.

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Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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## **FIGURES**

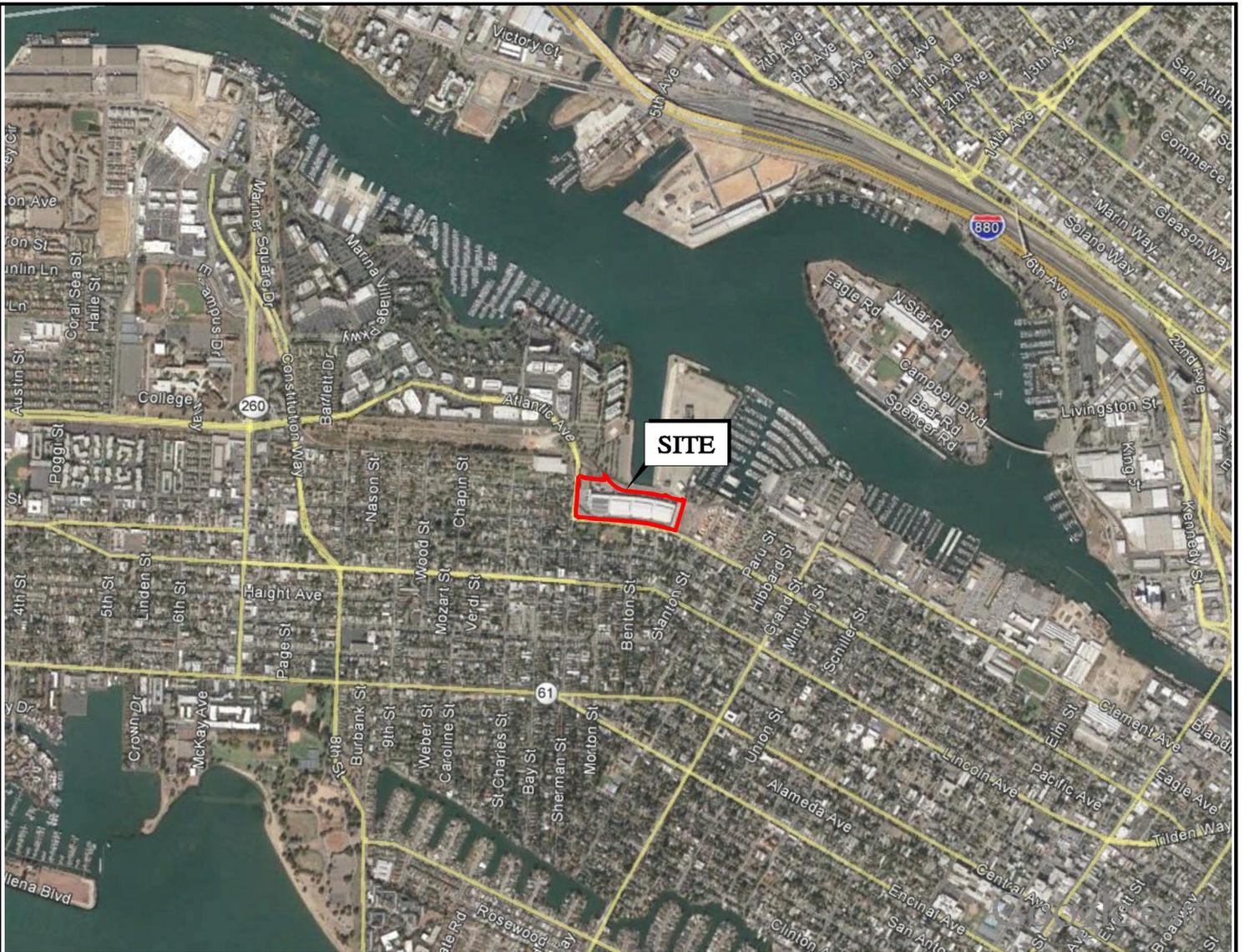
**FIGURE 1 – Vicinity Map**

**FIGURE 2 – Site Plan**

**FIGURE 3 – Regional Faulting and Seismicity**

**FIGURE 4 – Earth Pressure Diagram**

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BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE



VICINITY MAP  
 ALASKA BASIN BULKHEAD  
 ALAMEDA, CALIFORNIA

PROJECT NO.: 9769.000.001

SCALE: AS SHOWN

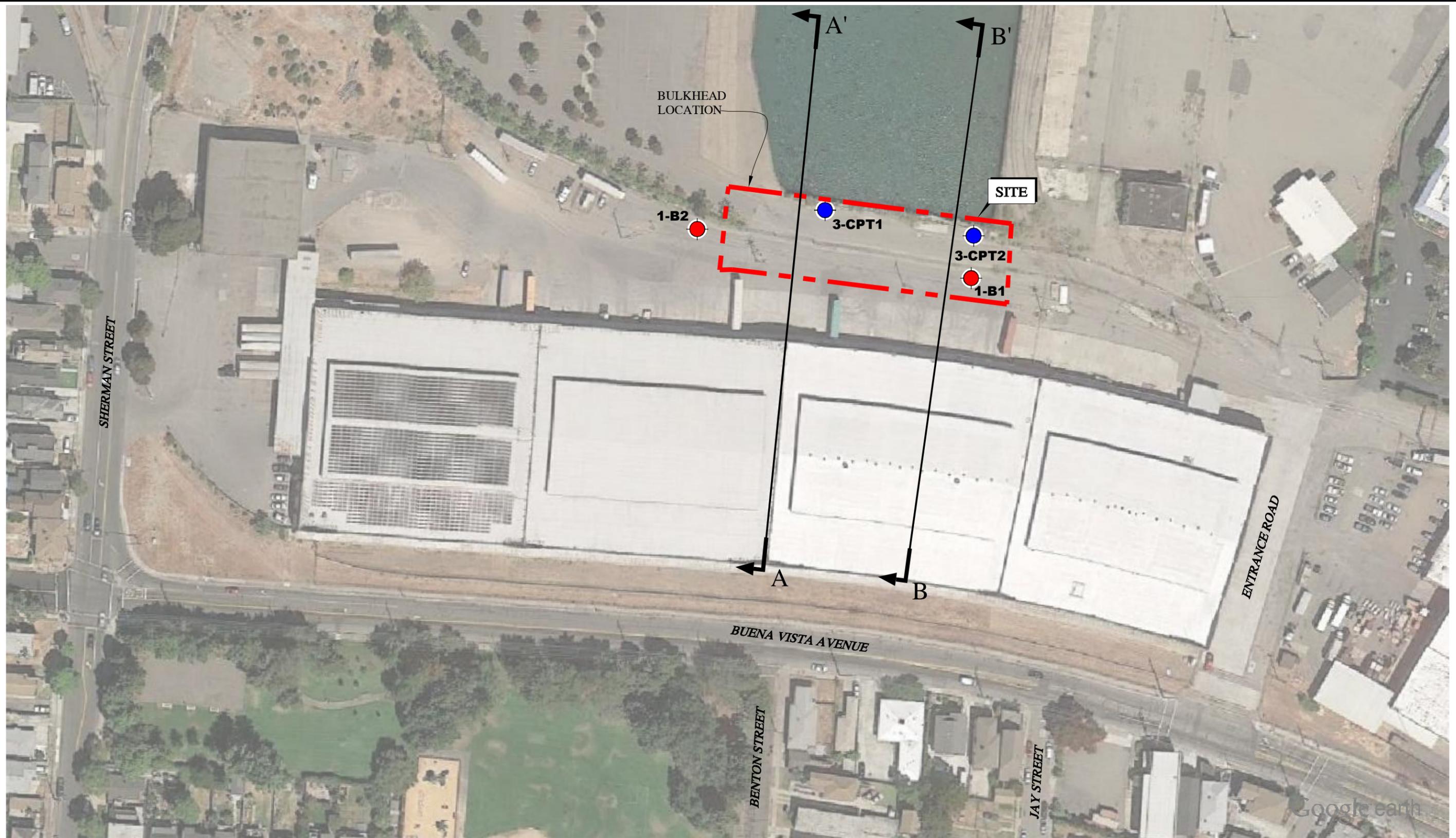
DRAWN BY: LL

CHECKED BY: JAF

FIGURE NO.

1

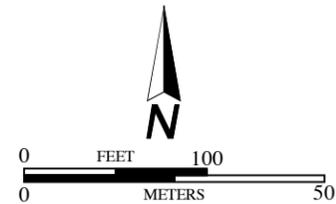
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**EXPLANATION**

ALL LOCATIONS ARE APPROXIMATE

- 3-CPT2**  CONE PENETRATION TEST (ENGeo, 2016)
- 1-B2**  BORING (ENGeo, 2014)



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

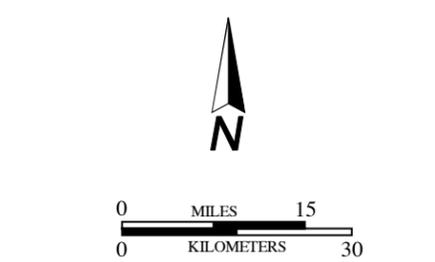
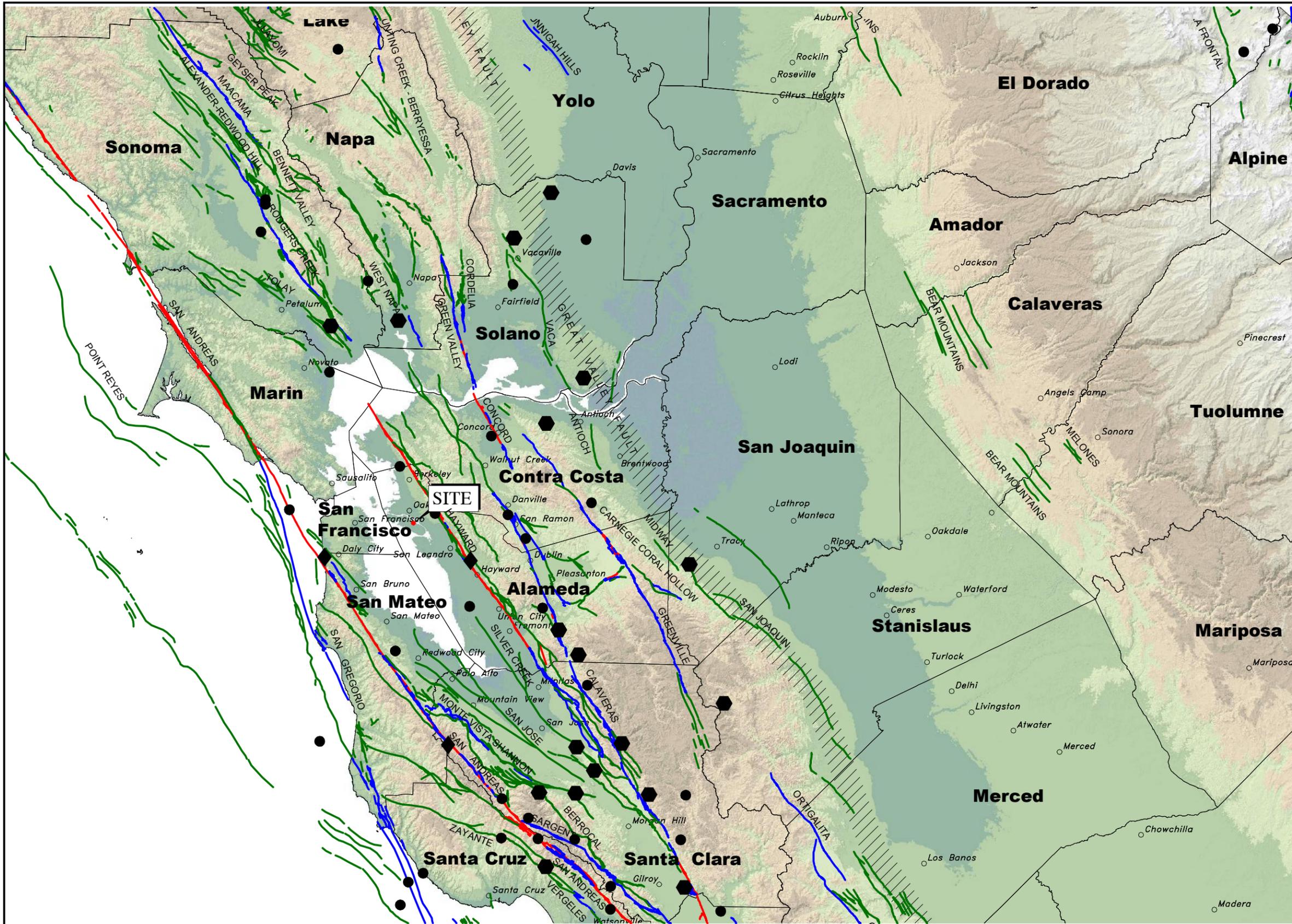


**SITE PLAN**  
 ALASKA BASIN BULKHEAD  
 ALAMEDA, CALIFORNIA

PROJECT NO.: 9769.000.001  
 SCALE: AS SHOWN  
 DRAWN BY: LL    CHECKED BY: JAF

FIGURE NO.  
**2**

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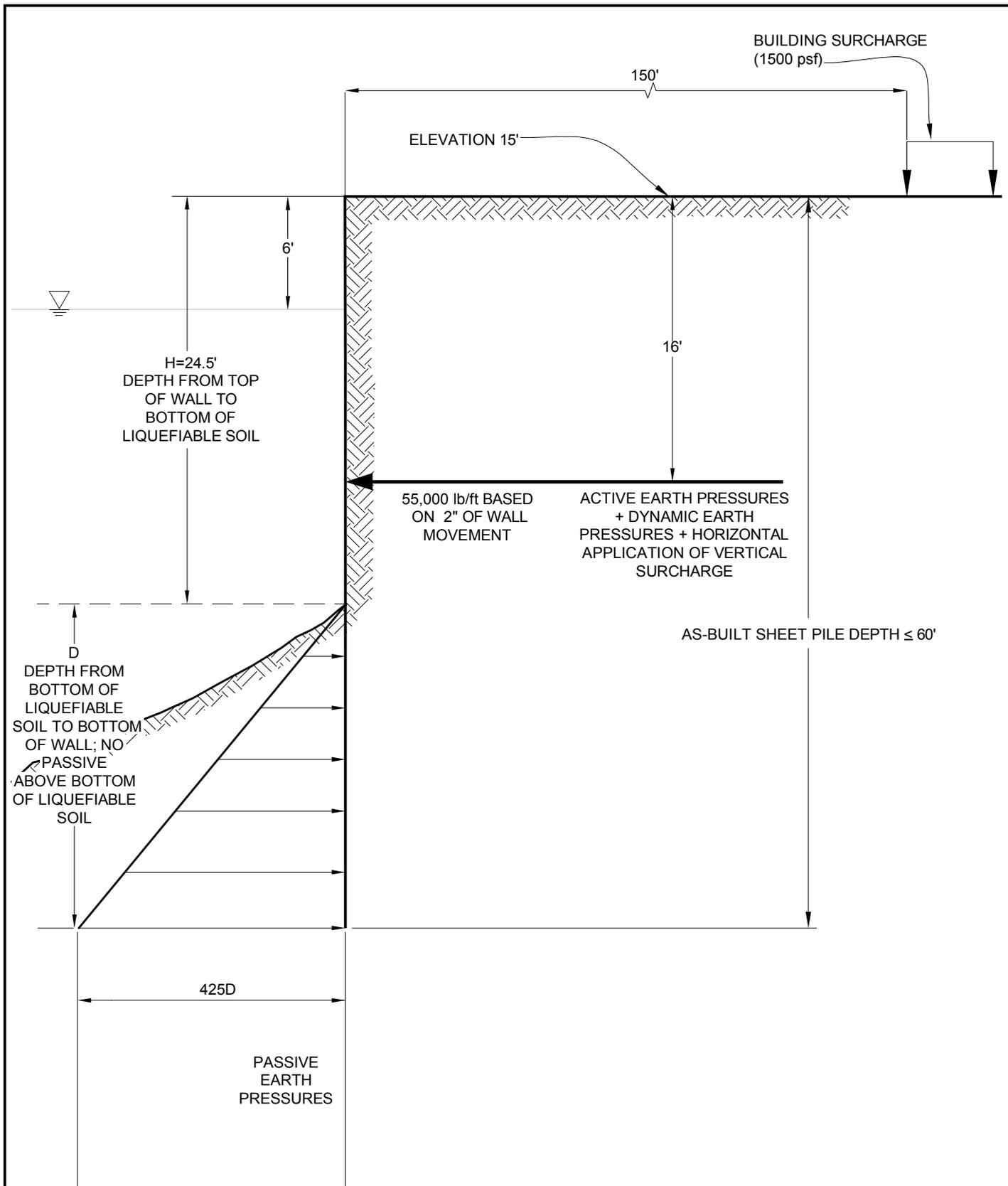


EXPLANATION	
◆	MAGNITUDE 7+
⬠	MAGNITUDE 6-7
●	MAGNITUDE 5-6
— (Red)	HISTORIC FAULT
— (Blue)	HOLOCENE FAULT
— (Green)	QUATERNARY FAULT
///	HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE:  
 COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION  
 U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010  
 U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

	REGIONAL FAULTING AND SEISMICITY ALASKA BASIN BULKHEAD ALAMEDA, CALIFORNIA		PROJECT NO.: 9769.000.001 SCALE: AS SHOWN DRAWN BY: LL    CHECKED BY: JAF	FIGURE NO. <b>3</b>
				ORIGINAL FIGURE PRINTED IN COLOR

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EARTH PRESSURE DIAGRAM  
ALASKA BASIN BULKHEAD  
ALAMEDA, CALIFORNIA

PROJECT NO.: 9769.000.001

SCALE: NO SCALE

DRAWN BY: SRP

CHECKED BY: JAF

FIGURE NO.

4



## **APPENDIX A**

### **CPT LOGS**

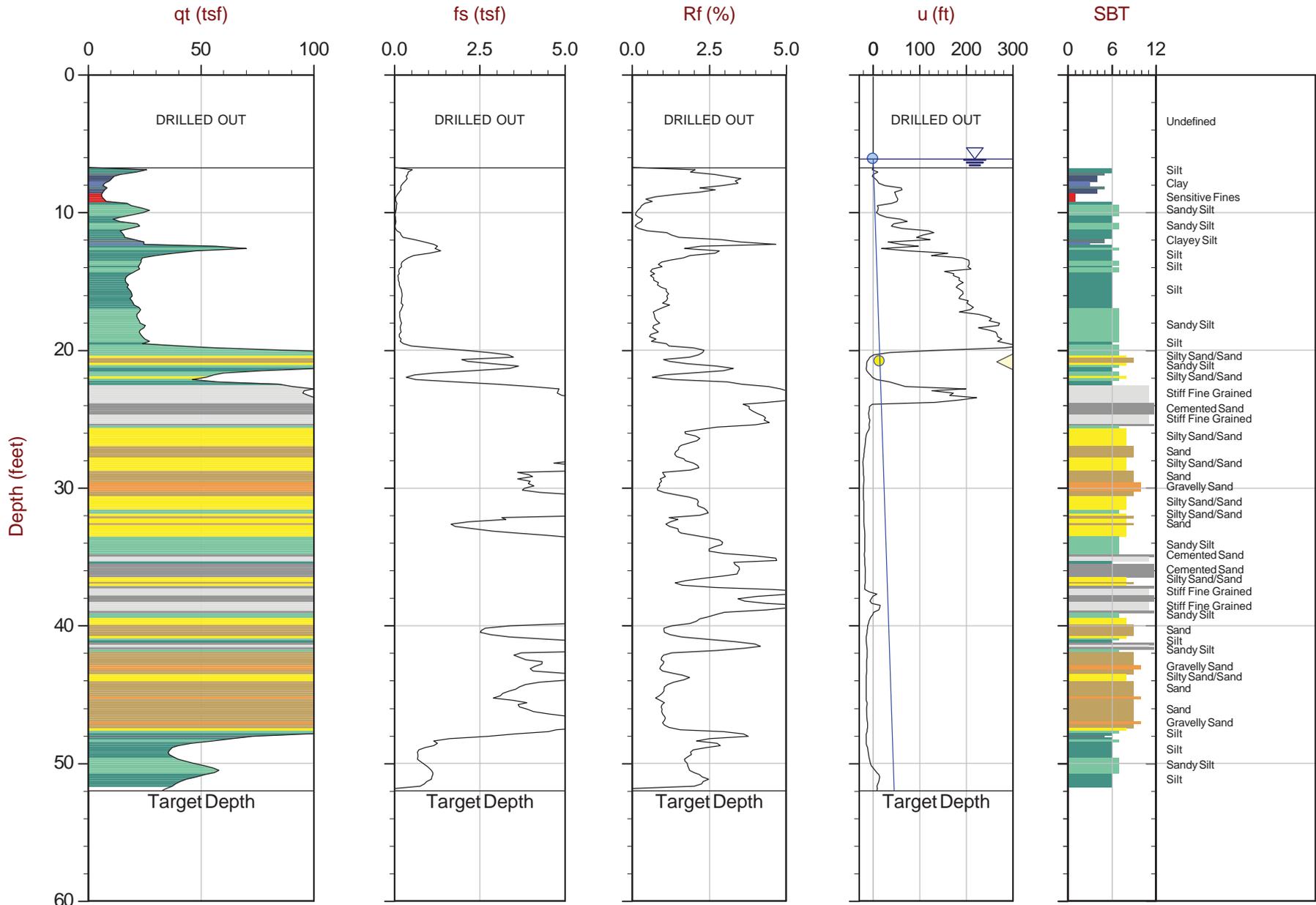




**ENGEO Inc.**

Job No: 16-56086  
 Date: 11:08:16 07:46  
 Site: Warehouse 48

Sounding: 3-CPT-2  
 Cone: 448:T1500F15U500



Max Depth: 15.850 m / 52.00 ft  
 Depth Inc: 0.050 m / 0.164 ft  
 Avg Int: Every Point

File: 16-56086\_CP02.COR  
 Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986  
 Coords: UTM10N N: 4181431m E: 565169m  
 Page No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    
 ● Assumed Ueq    
 ◀ Dissipation, Ueq achieved    
 ◀ Dissipation, Ueq not achieved    
 — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



## **APPENDIX B**

### **LABORATORY TEST DATA**

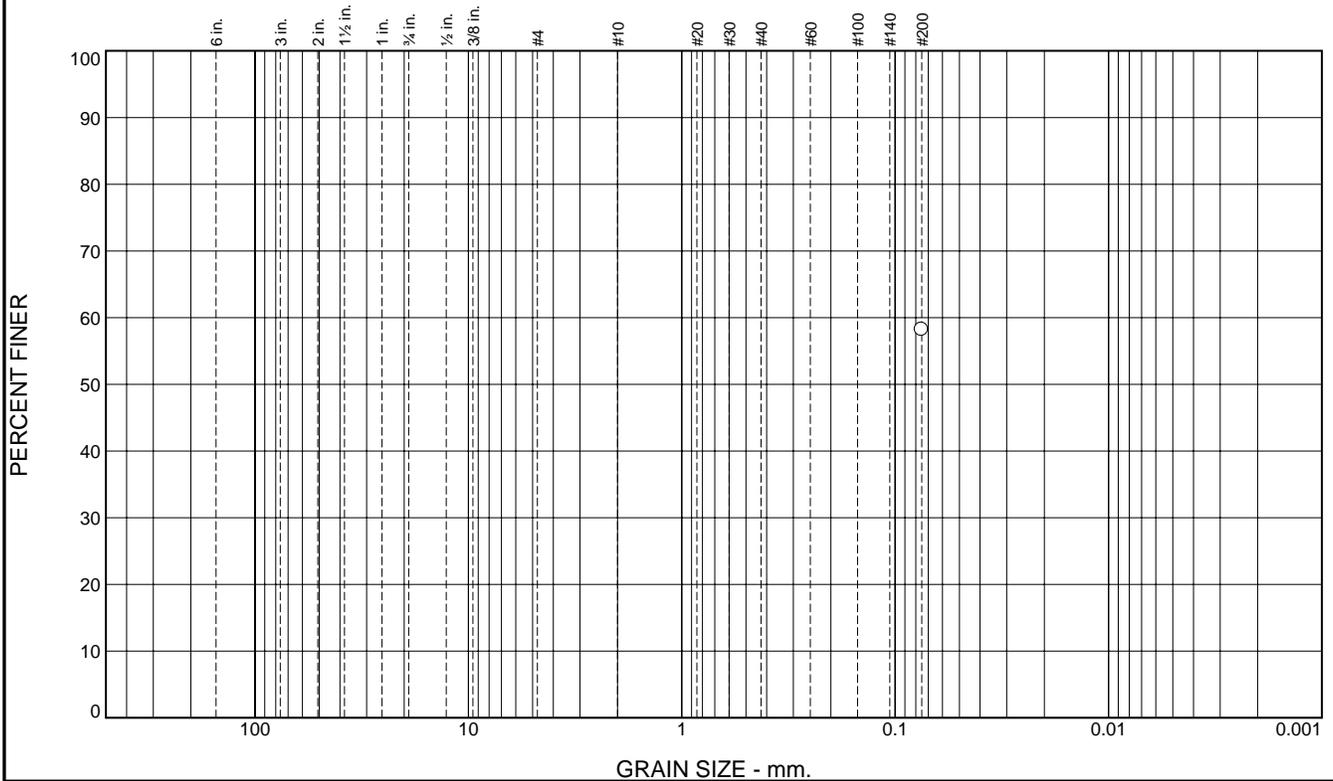
---

**LABORATORY TEST DATA**  
**ENGEO**

Select samples recovered during drilling activities were tested to determine various soil characteristics as presented on the following table.

<b>SOIL CHARACTERISTIC</b>	<b>TESTING METHOD</b>
Grain Size Distribution	ASTM D1140
Moisture Content Determination	ASTM D2216
Liquid and Plastic Limits Test	ASTM D4318 ASTM D1140

# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						58.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	58.2		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL= 15      LL= 23      PI= 8

**Coefficients**

D<sub>90</sub>=      D<sub>85</sub>=      D<sub>60</sub>=  
D<sub>50</sub>=      D<sub>30</sub>=      D<sub>15</sub>=  
D<sub>10</sub>=      C<sub>u</sub>=      C<sub>c</sub>=

**Classification**

USCS=      AASHTO=

**Remarks**

GS: ASTM D1140  
PI: ASTM D4318, Wet method

\* (no specification provided)

Sample Number: 3-CPT1 @ 15

Depth: 15.0-16.0 feet

Date: 11/15/16

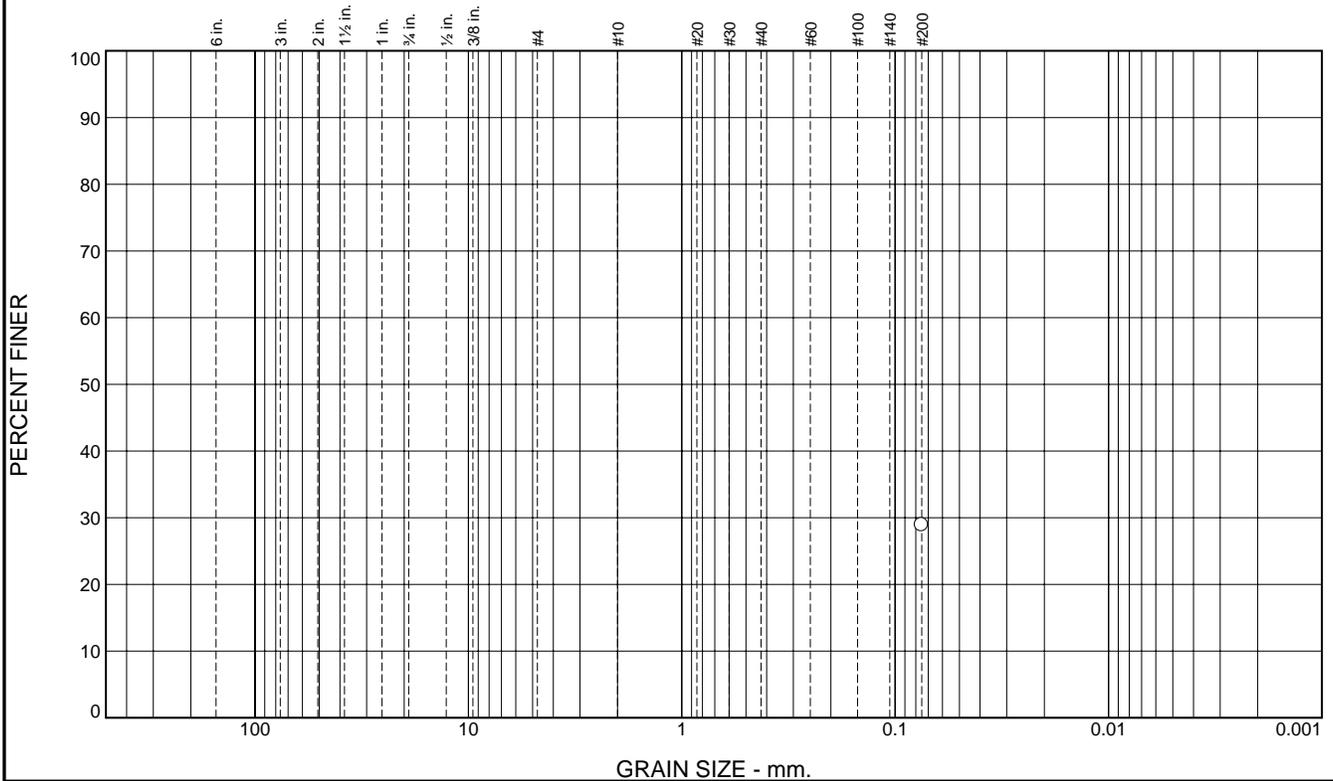


**Client:** STL Company, LLC  
**Project:** Del Monte Warehouse  
**Project No:** 9769.000.001

Tested By: M. Quasem

Checked By: D. Seibold

# Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						28.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	28.9		

**Soil Description**

See exploration logs

**Atterberg Limits**

PL= 14      LL= 16      PI= 2

**Coefficients**

D<sub>90</sub>=      D<sub>85</sub>=      D<sub>60</sub>=  
D<sub>50</sub>=      D<sub>30</sub>=      D<sub>15</sub>=  
D<sub>10</sub>=      C<sub>u</sub>=      C<sub>c</sub>=

**Classification**

USCS=      AASHTO=

**Remarks**

GS: ASTM D1140  
PI: ASTM D4318, Wet method

\* (no specification provided)

**Sample Number:** 3-CPT1 @ 19

**Depth:** 19.0-19.5 feet

**Date:** 11/15/16



**Client:** STL Company, LLC  
**Project:** Del Monte Warehouse  
**Project No:** 9769.000.001

**Tested By:** M. Quasem

**Checked By:** D. Seibold

# MOISTURE CONTENT DETERMINATION

ASTM D2216

BORING/SAMPLE ID	3-CPT1	3-CPT1						
DEPTH (ft)	15.0-16.0	19.0-19.5						
Method A or B	B	B						
%MOISTURE	18.5	17.5						

**PROJECT NAME: Del Monte Warehouse**

**DATE: 11/14/16**

**PROJECT NUMBER: 9769.000.001**

**CLIENT: STL Company, LLC**

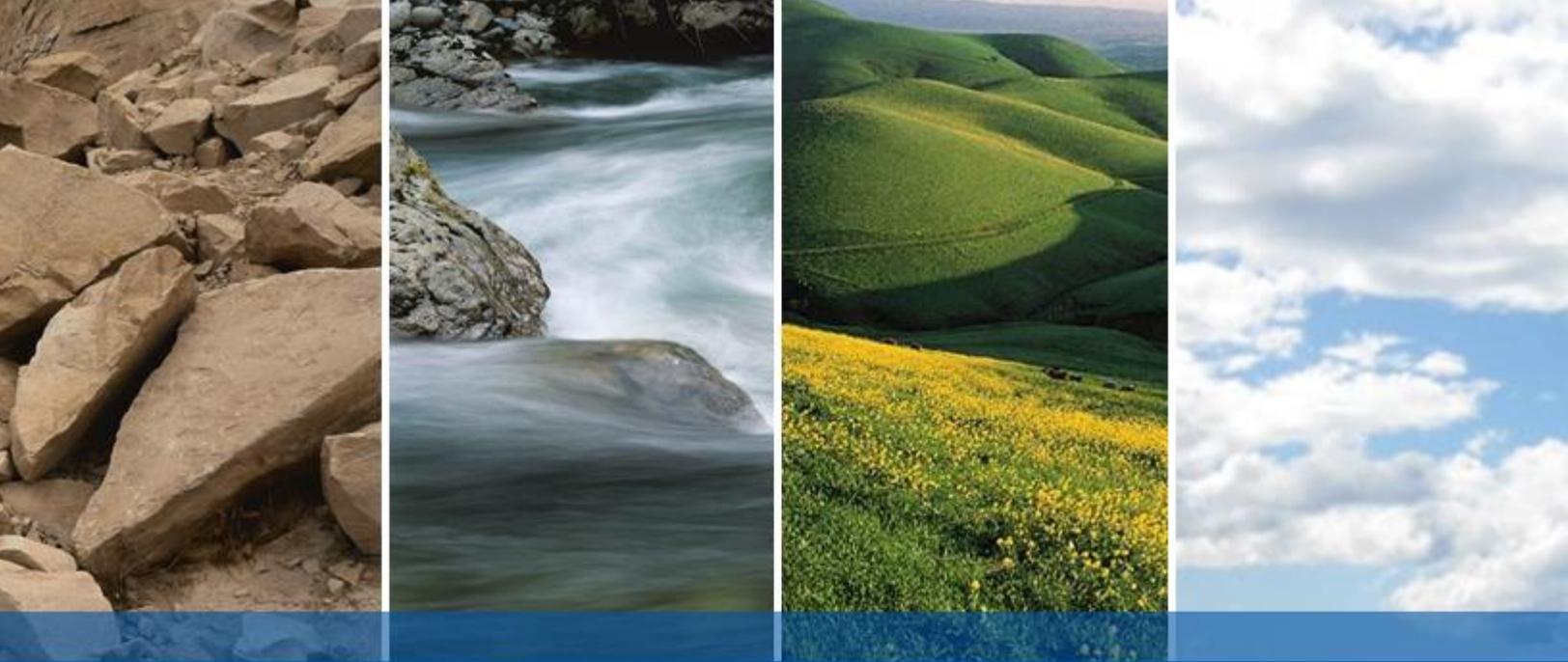
**PHASE NUMBER: 016**



Tested by: M. Quasem

Reviewed by: G. Criste





## **APPENDIX C**

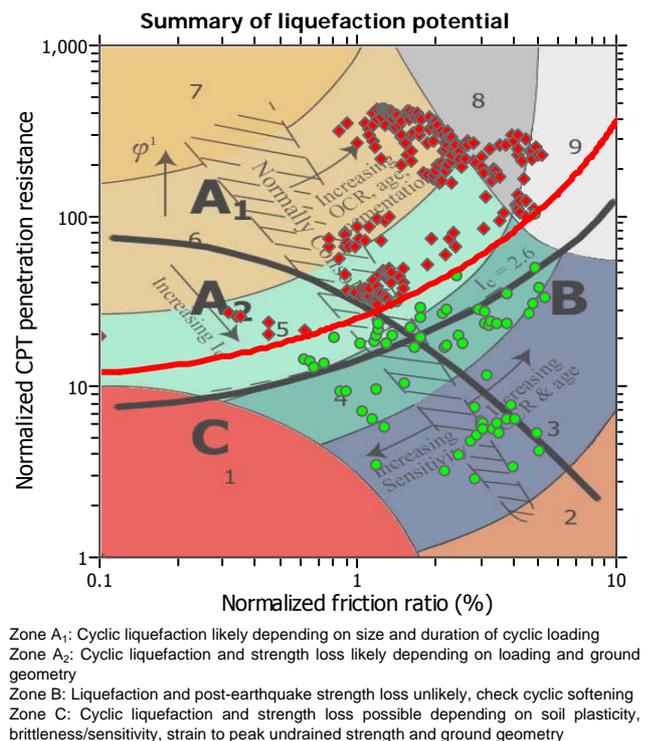
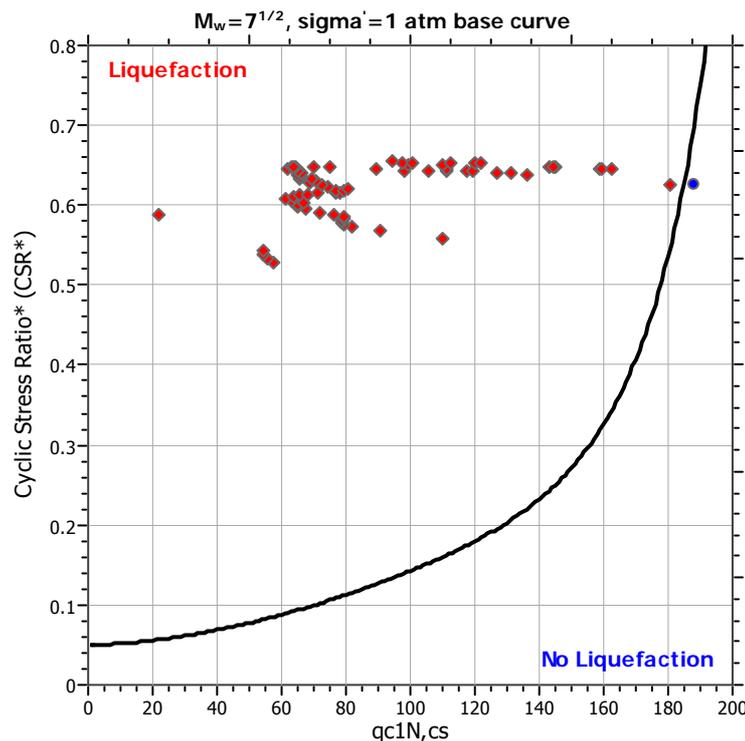
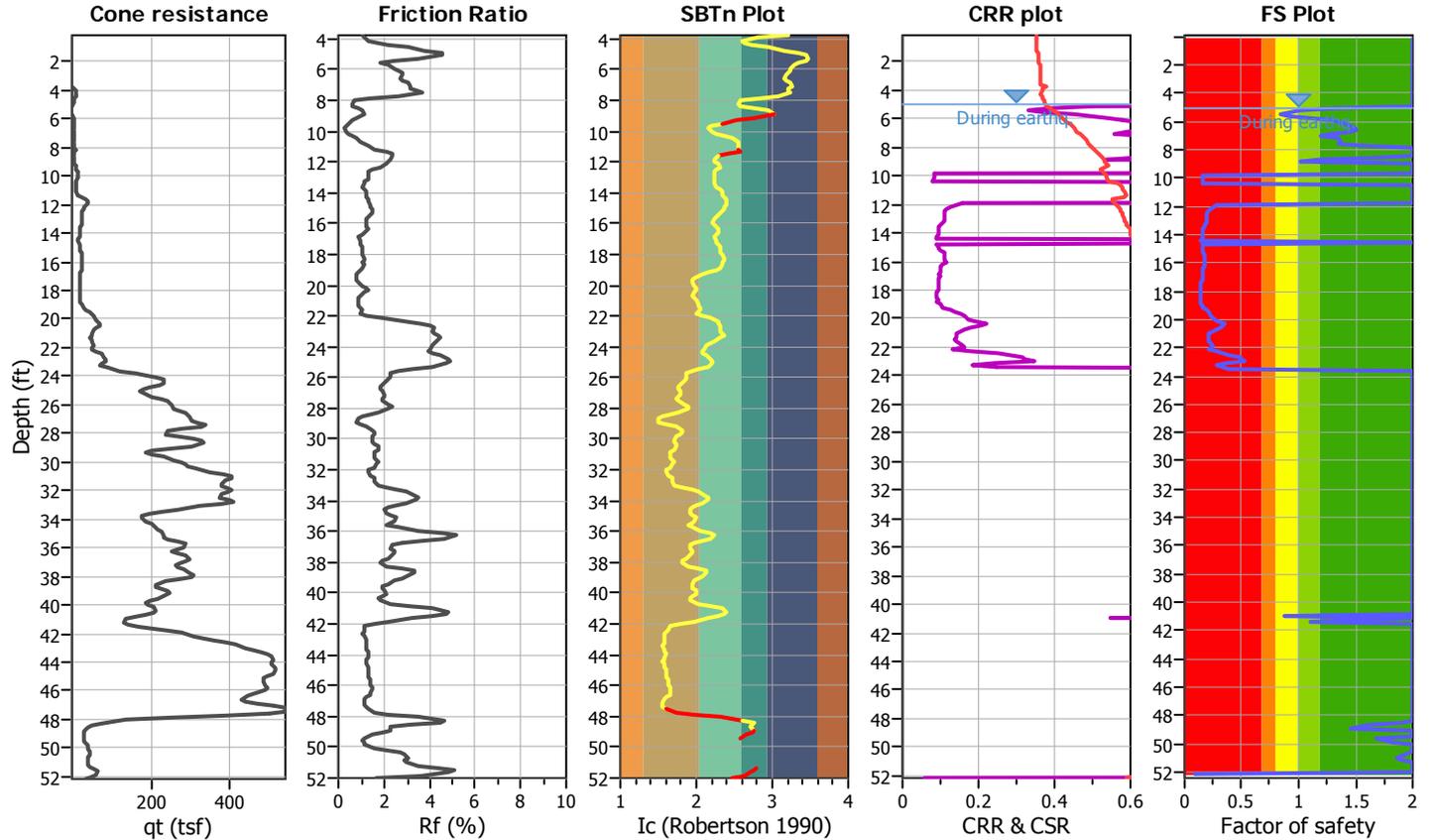
### **LIQUEFACTION ANALYSIS**

LIQUEFACTION ANALYSIS REPORT

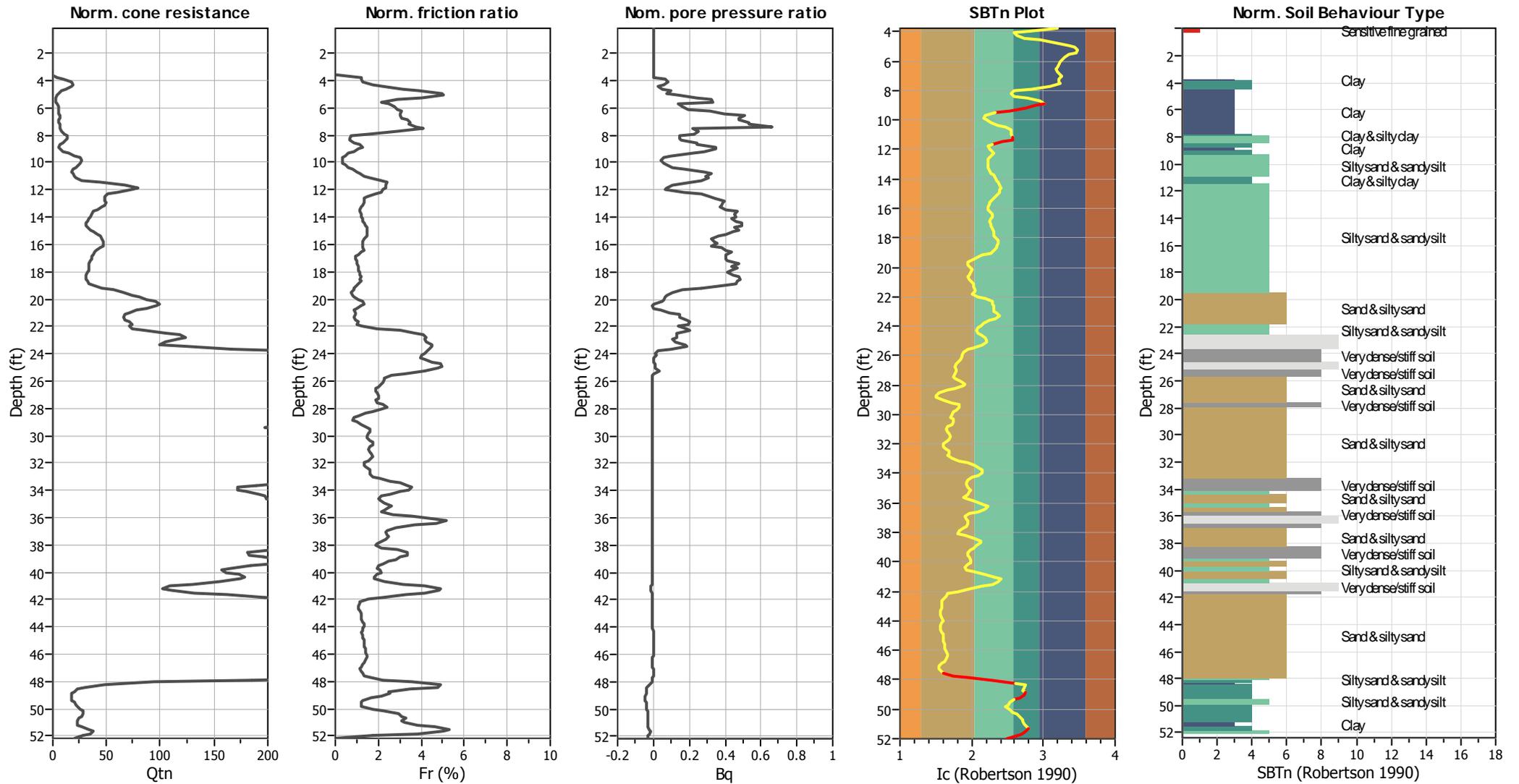
Project title : Alaska Basin Bulkhead Liquefaction Analysis      Location : 1501 Buena Vista Avenue, Alameda, CA  
CPT file : 3-CPT-1

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude $M_w$ :	7.33	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	$K_\sigma$ applied:	Yes		



### CPT basic interpretation plots (normalized)



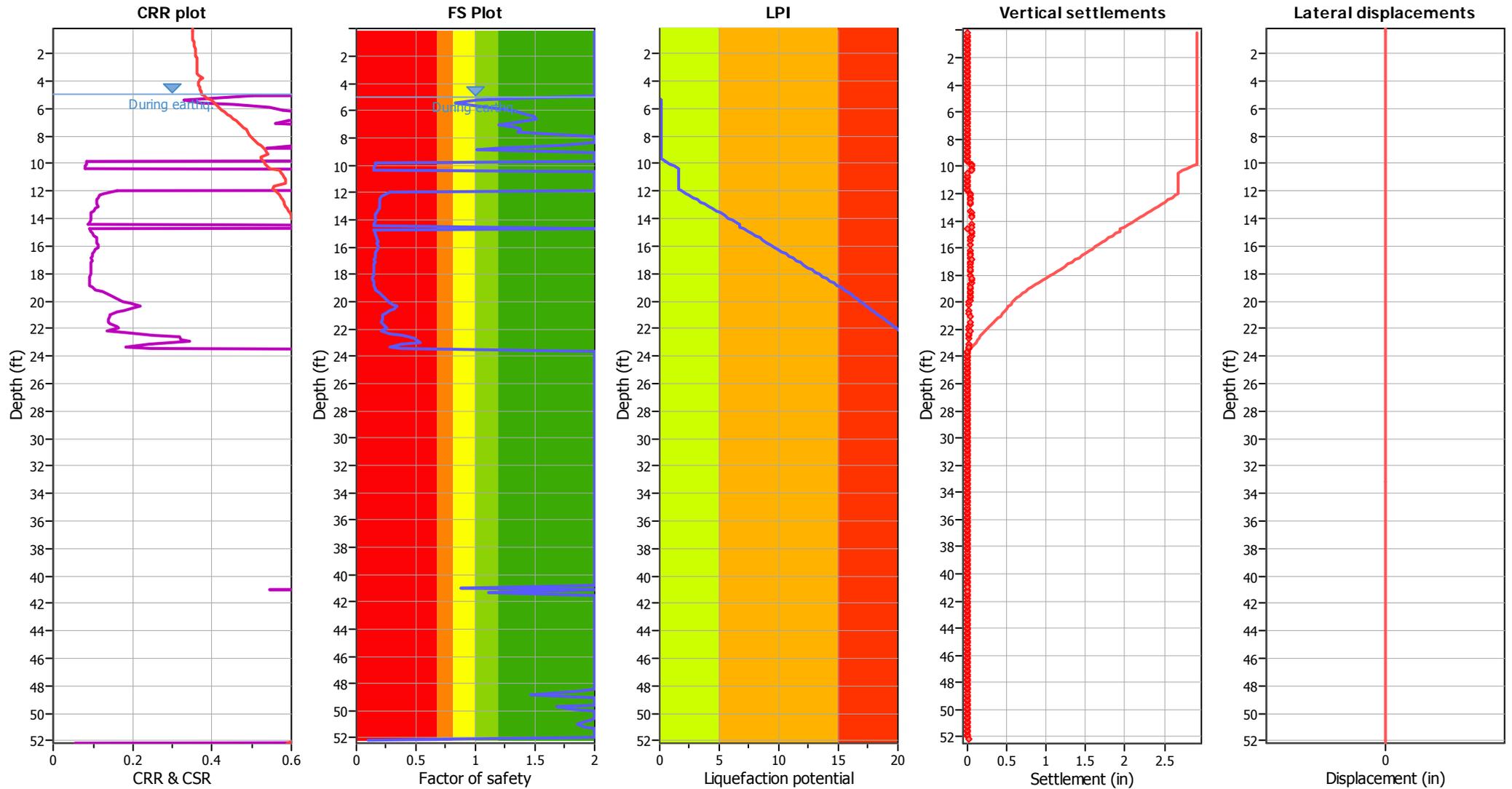
#### Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.40	K <sub>q</sub> applied:	Yes
Earthquake magnitude M <sub>w</sub> :	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

#### SBTn 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: I&B (2008)  
 Fines correction method: I&B (2008)  
 Points to test: Based on I<sub>c</sub> value  
 Earthquake magnitude M<sub>w</sub>: 7.33  
 Peak ground acceleration: 0.62  
 Depth to water table (insitu): 5.00 ft

Depth to GWT (erthq.): 5.00 ft  
 Average results interval: 3  
 I<sub>c</sub> cut-off value: 2.40  
 Unit weight calculation: Based on SBT  
 Use fill: No  
 Fill height: N/A

Fill weight: N/A  
 Transition detect. applied: Yes  
 K<sub>σ</sub> applied: Yes  
 Clay like behavior applied: Sand & Clay  
 Limit depth applied: No  
 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**

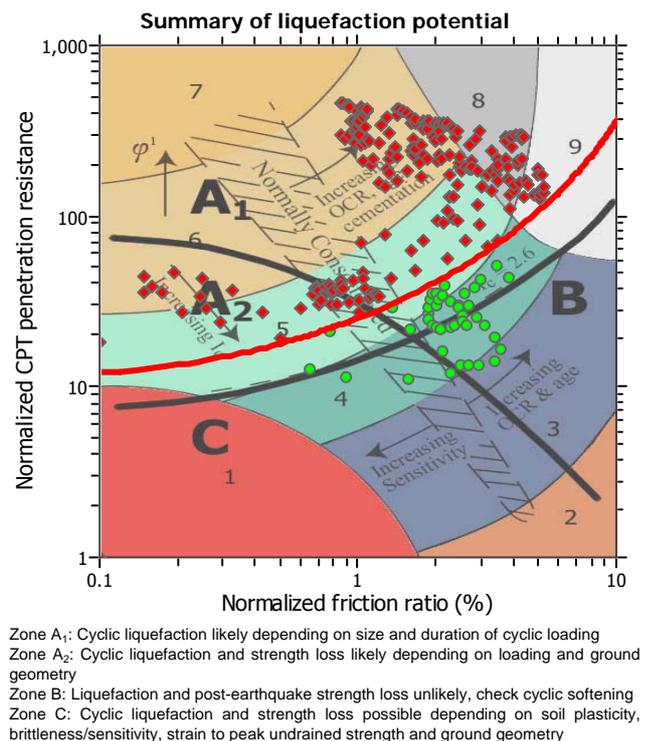
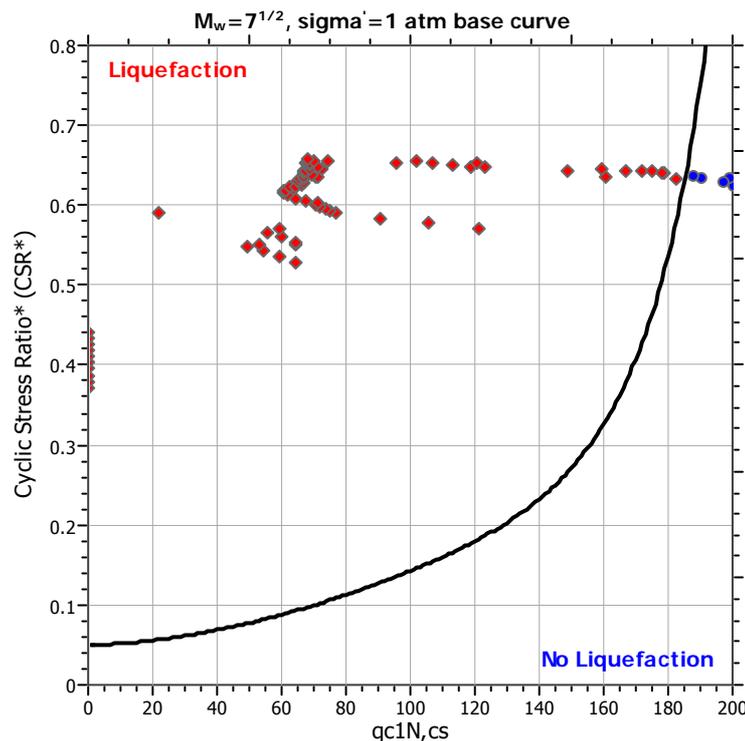
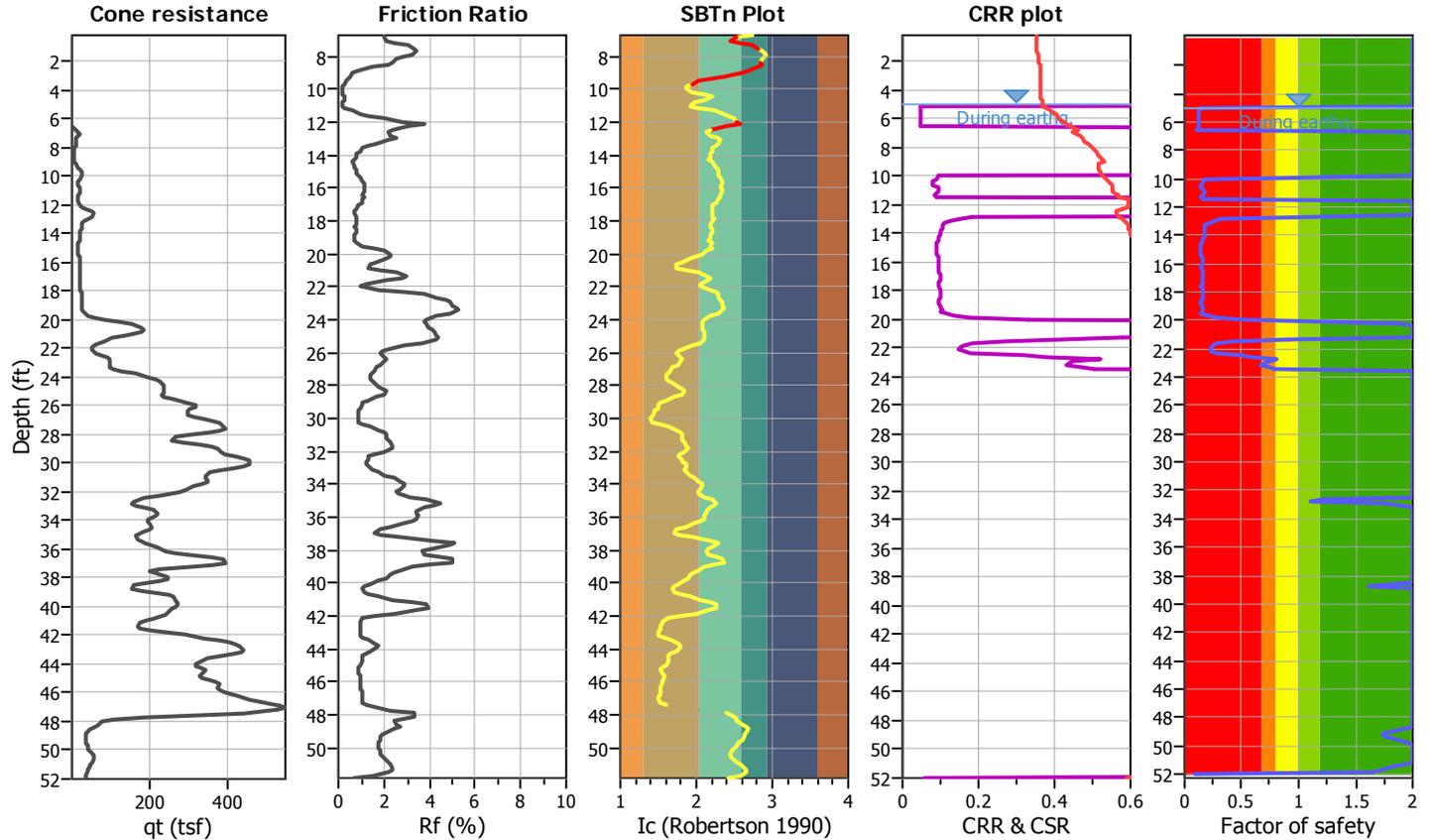
- Very high risk
- High risk
- Low risk

LIQUEFACTION ANALYSIS REPORT

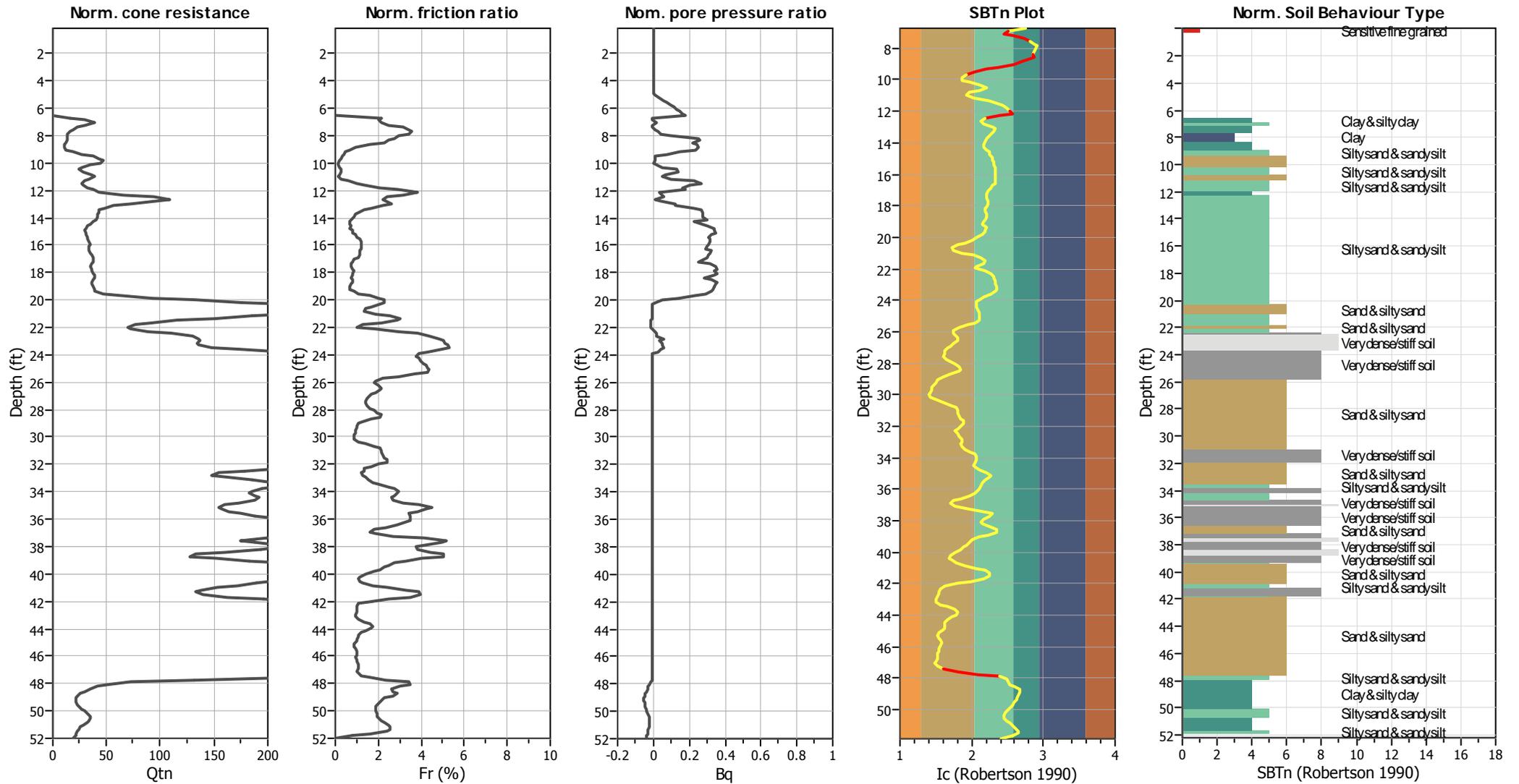
Project title : Alaska Basin Bulkhead Liquefaction Analysis      Location : 1501 Buena Vista Avenue, Alameda, CA  
CPT file : 3-CPT-2

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude $M_w$ :	7.33	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	$K_G$ applied:	Yes		



### CPT basic interpretation plots (normalized)



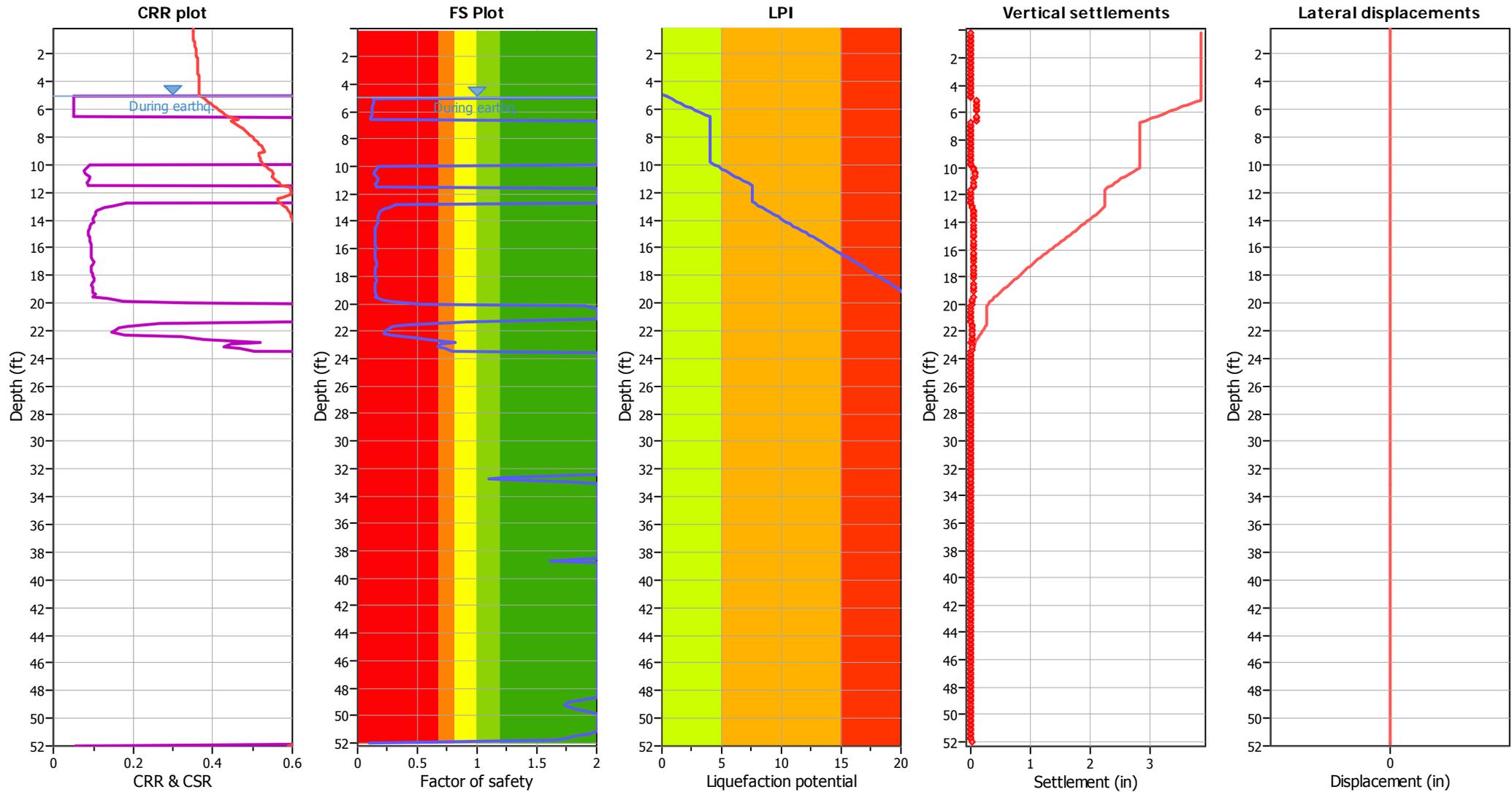
#### Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.40	K <sub>q</sub> applied:	Yes
Earthquake magnitude M <sub>w</sub> :	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

#### SBTn 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:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.40	$K_{\sigma}$ applied:	Yes
Earthquake magnitude $M_w$ :	7.33	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 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

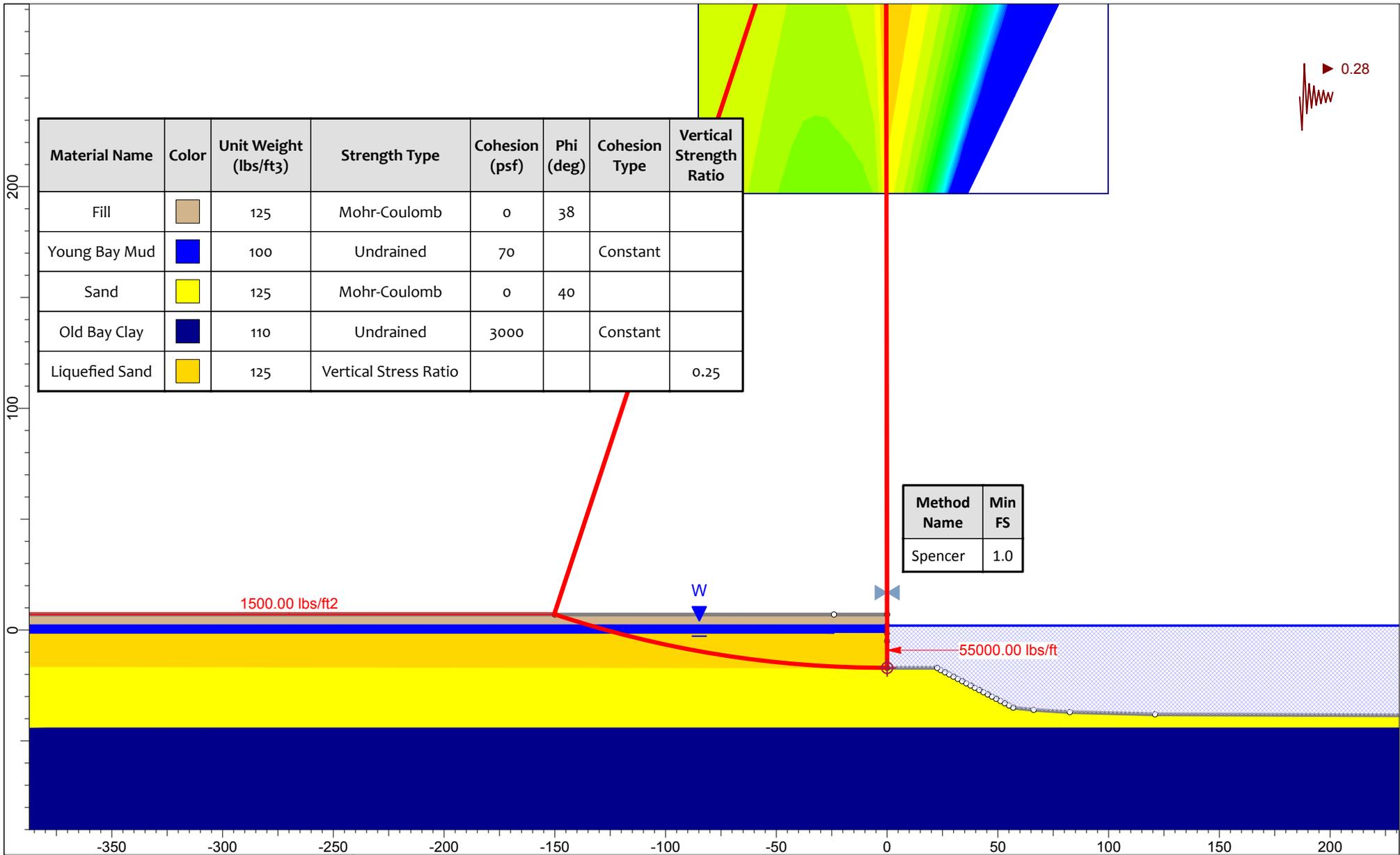
- Very high risk
- High risk
- Low risk



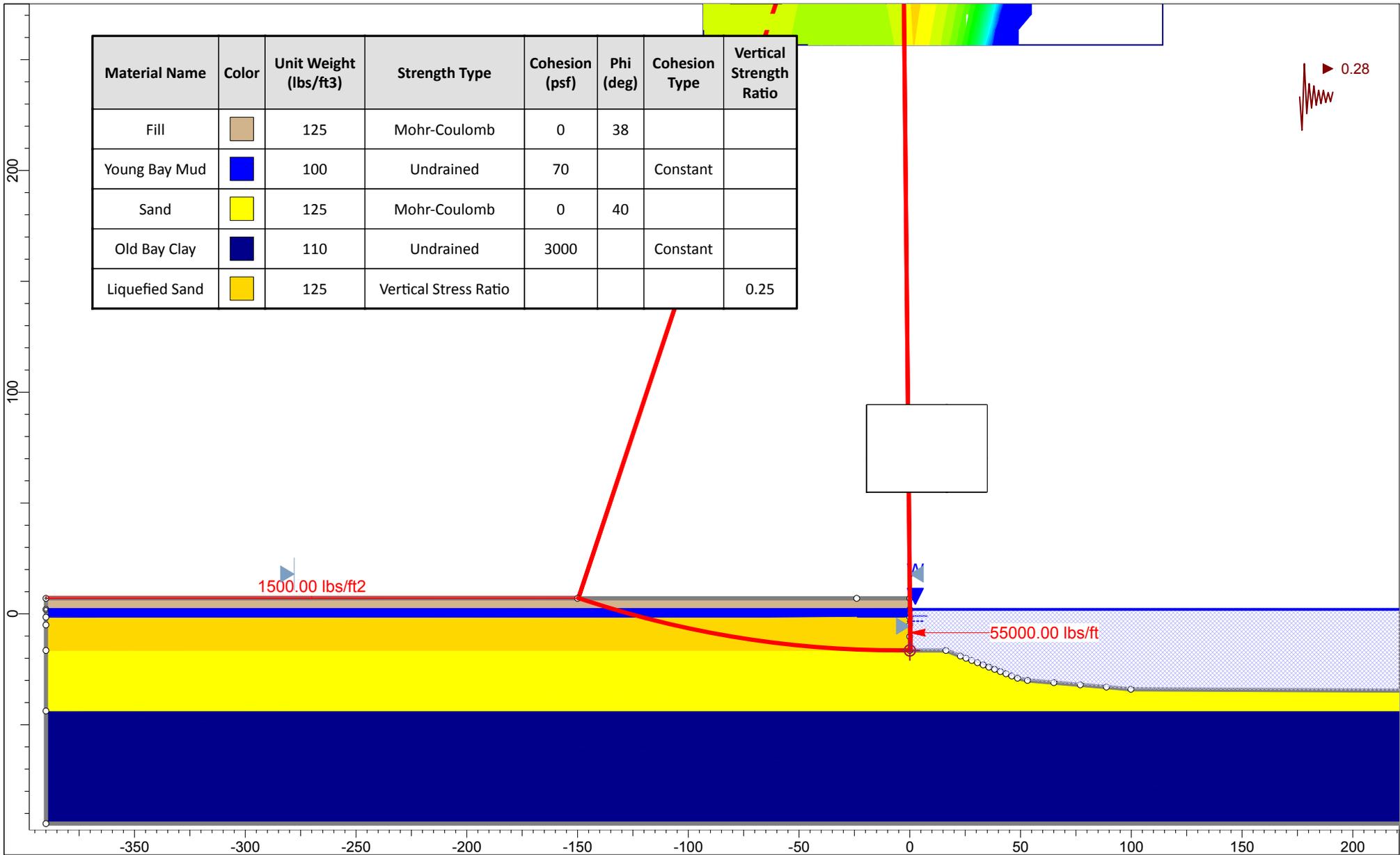
- 
- 
- 
- 

## **APPENDIX D**

### **EARTH PRESSURE ANALYSIS**



	Project			Del Monte Warehouse		
	Analysis Description			Pseudo-Static		
	Drawn By	SOS	Scale	1:720	Company	ENGEO
	Date	04/20/2017		File Name	Bulkhead A-A' - DSM - Pseudo-Static force for wall.slim	



Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Vertical Strength Ratio
Fill		125	Mohr-Coulomb	0	38		
Young Bay Mud		100	Undrained	70		Constant	
Sand		125	Mohr-Coulomb	0	40		
Old Bay Clay		110	Undrained	3000		Constant	
Liquefied Sand		125	Vertical Stress Ratio				0.25

0.28

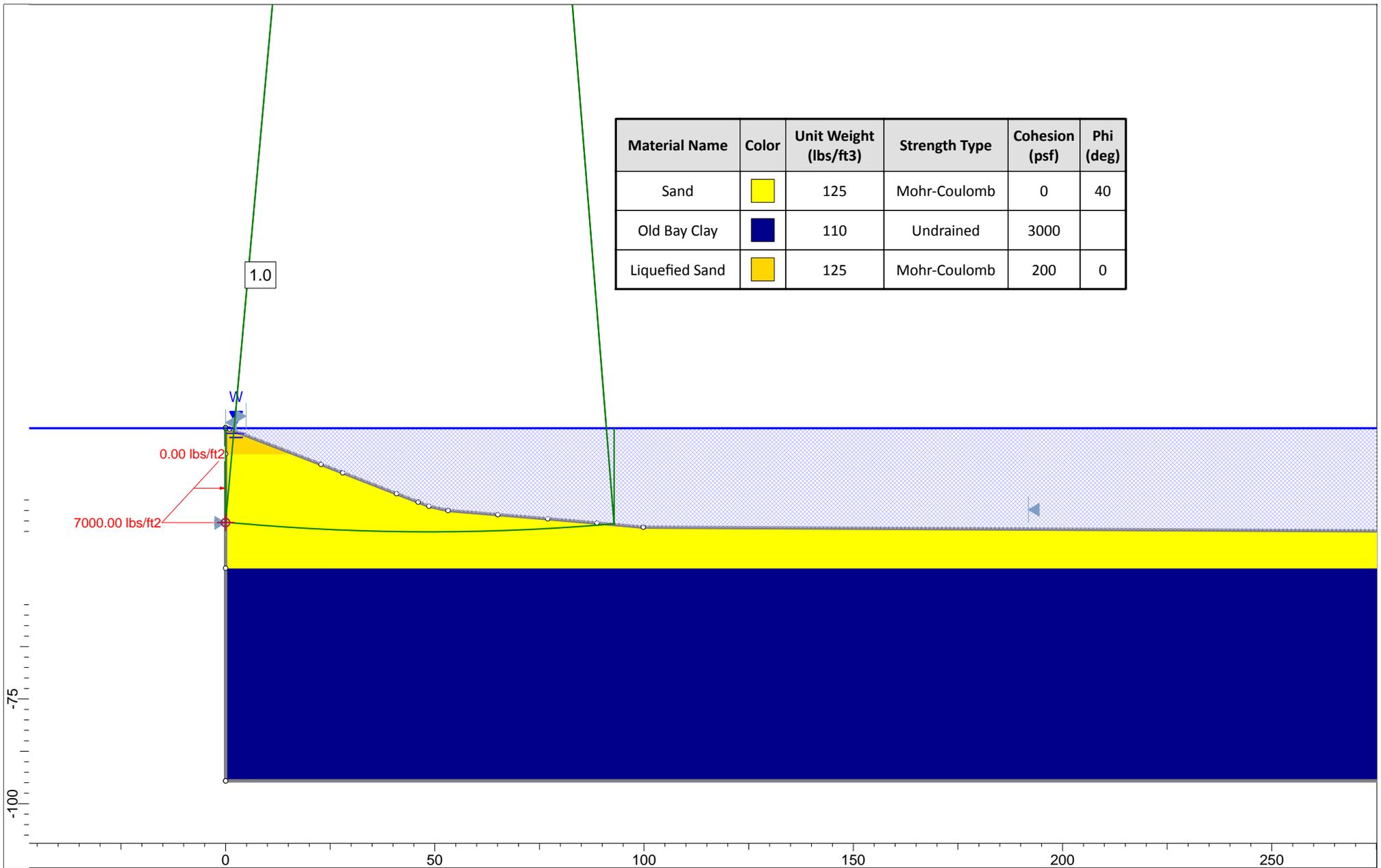
1500.00 lbs/ft<sup>2</sup>

55000.00 lbs/ft

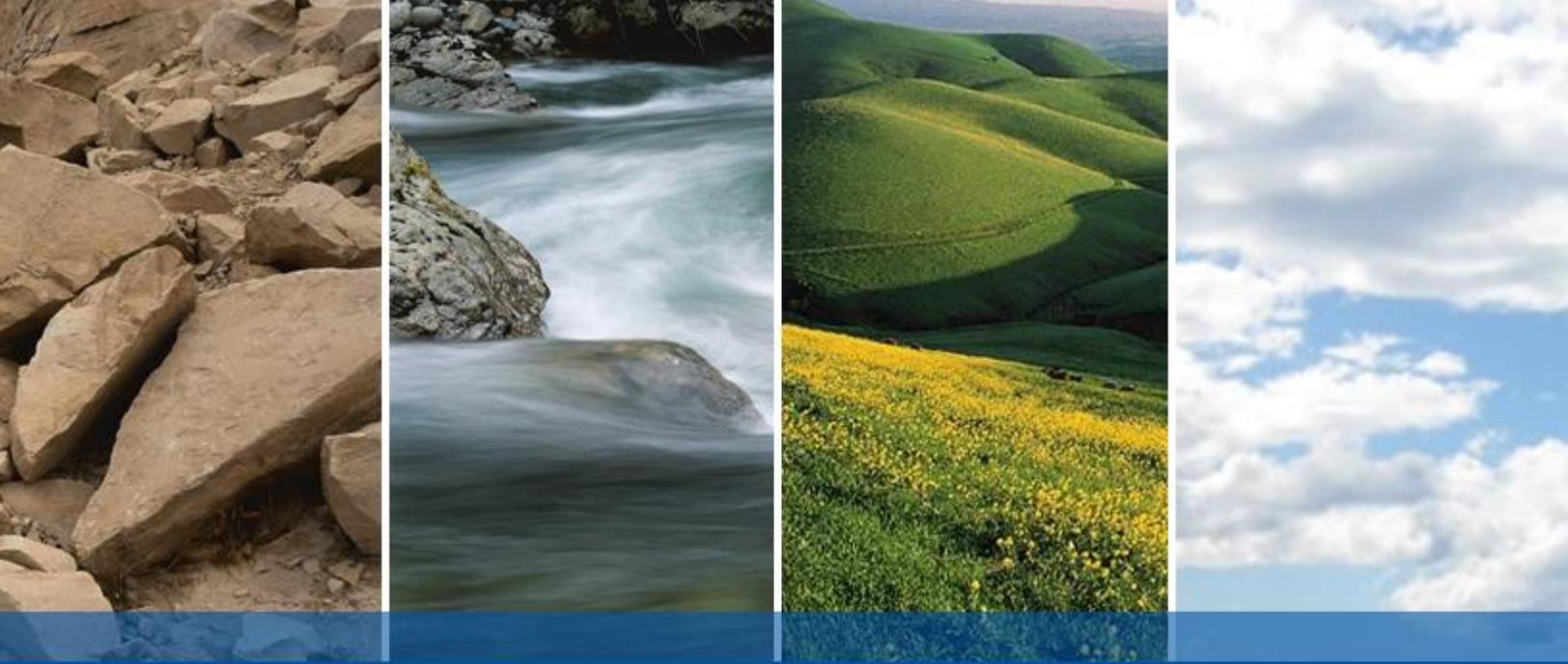


Project				Del Monte Warehouse			
Analysis Description				Pseudo-Static			
Drawn By		SOS		Scale		1:720	
Date				04/20/2017		Company	
						ENGEO	
				File Name		Bulkhead B-B" - DSM - Pseudo-Static force for wall.slim	

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sand	Yellow	125	Mohr-Coulomb	0	40
Old Bay Clay	Dark Blue	110	Undrained	3000	
Liquefied Sand	Light Yellow	125	Mohr-Coulomb	200	0

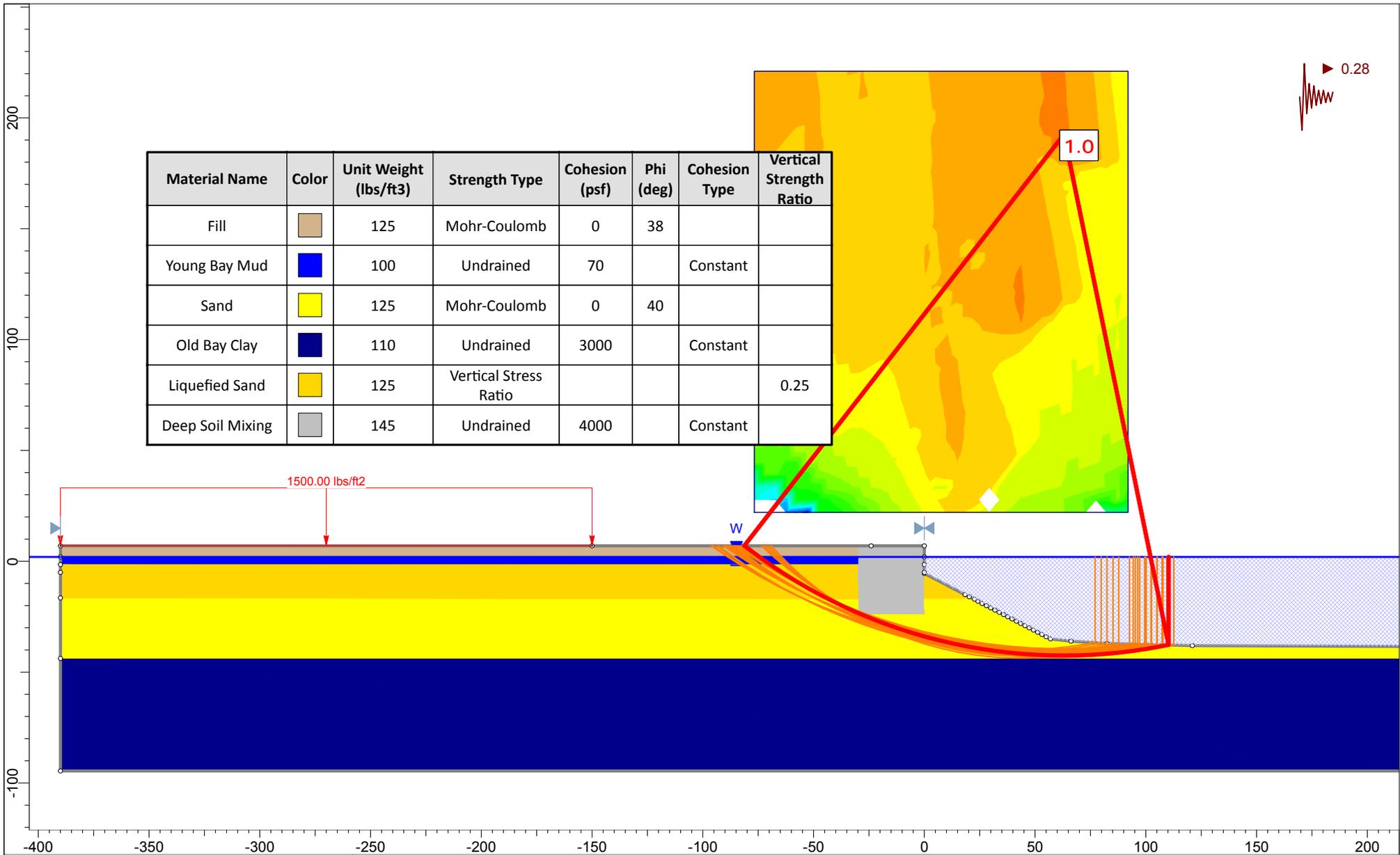


Project			Alaska Basin - Passive Pressures on Bulkhead - GLE Method		
Scale	1:375	Author	TMK	Project No.	9769.000.001
Date	December 1, 2016				



## **APPENDIX E**

### **DSM ANALYSES**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Vertical Strength Ratio
Fill		125	Mohr-Coulomb	0	38		
Young Bay Mud		100	Undrained	70		Constant	
Sand		125	Mohr-Coulomb	0	40		
Old Bay Clay		110	Undrained	3000		Constant	
Liquefied Sand		125	Vertical Stress Ratio				0.25
Deep Soil Mixing		145	Undrained	4000		Constant	

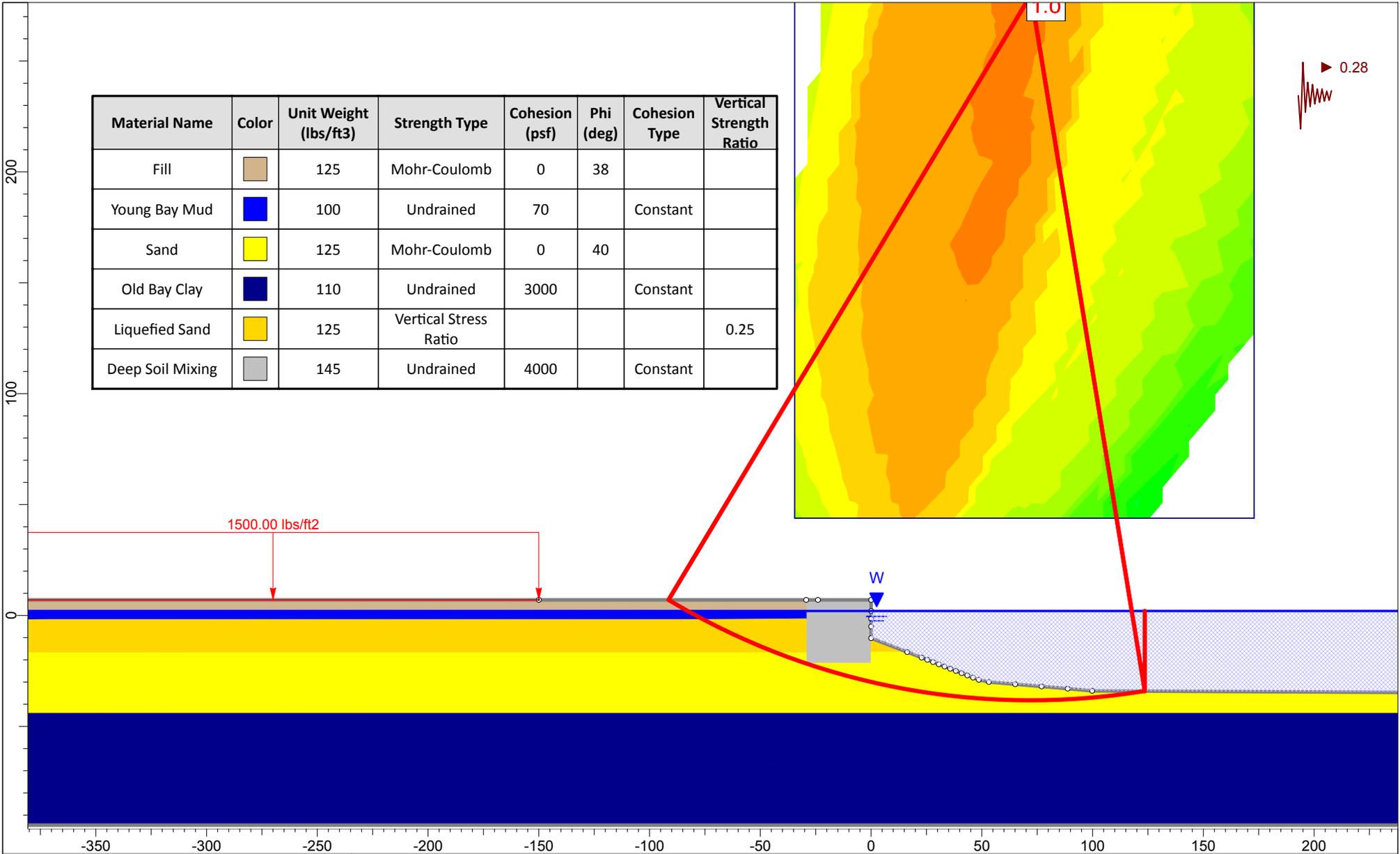
1500.00 lbs/ft2

1.0

0.28



Project		Del Monte Warehouse	
Analysis Description		Pseudo-Static	
Drawn By	SOS	Scale	1:720
Date	04/20/2017	Company	ENGEO
		File Name	Bulkhead A-A' - DSM - Pseudo-Static.slim



	Project			Del Monte Warehouse		
	Analysis Description			Pseudo-Static		
	Drawn By	SOS	Scale	1:720	Company	ENGEO
	Date	04/20/2017		File Name	Bulkhead B-B" - DSM - Pseudo-Static.slim	

