Feasibility-Level Geotechnical Investigation

Former Crowley Marine Property
1441 – 1551 Embarcadero
Oakland, California

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East Bay Regional Park District Land Acquisition
2950 Peralta Oaks Court, Oakland, California 94605

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1.0 INTRODUCTION

This report presents the results of our feasibility-level geotechnical investigation for the proposed Former Crowley Marine Property improvements to be constructed at the existing vacant lot located at 1441 – 1551 Embarcadero in Oakland, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide geotechnical recommendations for design of the proposed project.

For our use we received the following documents:

- Preliminary Title Report Order No. 1117017057 dated September 23, 2016
- Soil Testing Report by Northgate Environmental Management, Inc. dated March 17, 2016
- Remedial Action Completion Certification by Alameda County Environmental Protection dated January 7, 2000
- Fuel Leak Site Case Closure Letter by Alameda County Environmental Protection dated January 7, 2000
- Update to Risk Assessment Report for the Former Pacific Dry Dock and Repair Company Yard I Site in Oakland, California by Risk-Based Decisions, Inc, dated July 6, 1998
- Rescission of Cleanup and Abatement Order 96-111 for the Properties at 1441 Embarcadero (Yard I) and 321 Embarcadero (Yard II), Oakland, Alameda County dated April 22, 1998
- Cleanup and Abatement Order 96-111 for the Properties at 1441 Embarcadero (Yard I) and 321 Embarcadero (Yard II), Oakland, California, Alameda County dated July 15, 1996
- Preliminary Investigation and Evaluation Report, Pacific Dry Dock and Repair Yard I, Western Section, Oakland, California by Versar, Inc. dated May 6, 1992
- Summary of Tank Removal Activities, Pacific Dry Dock Yard I, 1441 Embarcadero, Oakland, California by Versar, Inc. dated January 14, 1992
- A report titled "1441 Embarcadero Shoreline Redevelopment – Feasibility Study – DRAFT" Dated October 27, 2017, prepared by Mott MacDonald

1.1 Project Description

We understand that East Bay Regional Park District (EBRPD) is considering a long-term lease with the Port of Oakland for the property and constructing a public park at the Site with a path along the waterside boundary of the Site. The site is bordered by Embarcadero to the northeast, Oakland Estuary to the southwest and southeast, and an existing facility to the northwest. EBRPD's conceptual plans include redevelopment as a possible "pocket park" with paved parking, picnic tables, trees and landscaping, and a public path along the waterfront. The path will include construction of a 12-foot-wide paved trail with 5-foot-wide unpaved shoulder along the waterfront portions of the Site that will be part of the Oakland Waterfront Trail and San Francisco Bay Trail.

We understand that there are three alternatives being considered for the site development. They are replace the existing bulkhead with a new similar tied-back sheet pile structure, remove the existing bulkhead and construct a rock revetment, and no action (safe setback) which would not touch the existing bulkhead and provide a setback distance to mitigate any potential ground movement as a result of failure of the existing bulkhead.

Proposed surface improvements will consist of hardscape and/or landscape elements and associated underground utilities.

1.2 Scope of Services

Our scope of services was presented in our agreement with you dated September 8, 2017. To accomplish this work, we have provided the following services:

- Exploration of subsurface conditions by drilling three borings in the area of the proposed development and retrieving soil samples for observation and laboratory testing.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analysis to evaluate site earthwork, retaining walls, slabs-on-grade, and pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Site Reconnaissance

Our Senior Project Manager performed a reconnaissance of the site on September 11, 2017. The Site comprises two (2) parcels of land (APN 18-475-2 and 18-430-3) that totals approximately 1.44 acres. The Site has a rectangular shape (approximately 385 feet by 165 feet). The site is currently vacant land covered with gravel and asphalt or concrete pavement, with a seawall extending approximately 330 linear feet along the southwestern portion of the site consisting of steel sheet piles. The project area is relatively level, with minor ground surface slopes. Historical site operations included a marine dry dock (Pacific Dry Dock) and repair yard operated by Crowley Marine Services, Inc.

Documents provided to TRC included no construction details or other design information regarding the onsite fill, compaction, if any, and/or design for the seawall. Accordingly, it is unknown whether any tie-backs were installed or how deep the sheet pile were installed. At the southern terminus of the sheet piles, concrete debris over soil/sediment is present as erosion control extending approximately 50 feet southward to the southeastern corner of the Site. Wooden piles were observed along this portion of the Site.

The seawall appears to be failing at several locations as a result of corrosion to the steel sheet piles and/or soil settlement. There are areas where the sheet piles are fully corroded through and tilting outward toward the Oakland Estuary, separating from concrete caps by approximately 6 to 10 inches. Additionally, significant soil settlement was observed within 3 feet of the sheet piles, which has undermined some concrete stringers by as much as 12 to 18 inches. It is also not clear whether wooden poles bolted to the sheet piles were driven into the submerged mud and are now rotted through or were used as a batten to protect the sheet pile from docked boats.

Along the southern portion of the site, settlement was observed that has undermined the surficial concrete and asphalt pavement by 4 to 6 feet. Wooden piles were also observed along this portion of the Site. Portions of the concrete stringer inboard of the wooden piles along the southern edge of the Site have failed. Concrete debris over soil/sediment is present along the southern portion of the Site (approximately 165 feet) as erosion control extending approximately 50 feet southward to the southeastern corner of the Site.

2.2 Exploration Program

Subsurface exploration was performed on September 11, 2017 using truck-mounted, hollow-stem auger drilling equipment to investigate, sample and log subsurface soils. Three 8-inch diameter hollow-stem auger exploratory borings were drilled to a depth of up to 50 feet. The borings were permitted and backfilled in accordance with Alameda County Public Works Agency Water Resources. The approximate location of the borings are shown on the Site Plan, Figure 2. The logs of the borings and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

2.3 Subsurface Conditions

Boring EB-1 encountered a pavement section consisting of 3½ inches of asphalt concrete (AC) over 4½ inches of aggregate base (AB). Below the pavement section, EB-1 encountered fill to a depth of approximately 15 feet consisting of medium dense to dense clayey gravel to a depth of approximately 5 feet, underlain by loose clayey sand. Below the fill, interbedded layers consisting of medium stiff to stiff elastic silt (known locally as Bay Mud), loose silty sand, medium dense to very dense clayey sand, and medium stiff to very stiff sandy lean clay were encountered to a depth of about 36½ feet. Below this depth, medium stiff to very stiff lean clay was encountered to 50 feet, the maximum depth explored.

Boring EB-2 encountered fill to a depth of 5 feet consisting of medium stiff lean clay and medium stiff sandy lean clay. Below the fill, generally medium stiff to very stiff lean clay and medium stiff to stiff sandy lean clay was encountered to a depth of approximately 46½ feet, with some thin interbedded layers consisting of soft to medium stiff elastic silt. Below this depth, dense clayey sand was encountered to a depth of 50 feet.

Boring EB-3 encountered fill to a depth of approximately 17 feet consisting of interbedded layers of loose to dense clayey sand and medium dense clayey gravel. Below the fill, soft to medium stiff elastic silt was encountered to a depth of about 26½ feet, underlain by interbedded layers of stiff gravelly lean clay, stiff to very stiff lean clay, very stiff sandy lean clay, and dense silty sand.

2.4 Ground Water

Free ground water was encountered in all of our borings at depths ranging from 8½ to 10½ feet below the ground surface. Based on the depth to historically high ground water map prepared by the California Geological Survey for the Oakland East Quadrangle (CGS, 2003), the depth to historically high ground water levels in the site vicinity is on the order of 5 feet below the existing ground surface (bgs). We judge a ground water depth of 5 feet to be appropriate for design and construction. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Maximum Estimated Ground Shaking

Based on Equation 11.8-1 of ASCE 7-10, we judge a maximum considered earthquake geometric mean peak ground acceleration of 0.61g to be appropriate for geotechnical analyses for the project site.

3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (WGCEP, 2014) estimates there is a 72 percent chance of at least one magnitude 6.7 earthquake occurring in the San Francisco Bay region between 2014 and 2044. This result is an important outcome of WGCEP's work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.4 Liquefaction

3.4.1 General Background

The site is located in an area that has been mapped by the State of California for the potential for seismically induced liquefaction hazards (CGS, 2003). During cyclic ground shaking, such as earthquakes, cyclically-induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to moderately dense, saturated, non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

3.4.2 Analysis and Results

Based on our explorations, design ground water levels of 5 feet below the existing site grades were used for our liquefaction analysis. As discussed in the subsurface description above, several sand and silt layers were encountered below the design ground water depth. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed structures. No liquefaction analyses were performed on layers above the design ground water depths.

Our liquefaction analysis followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for SPT analysis update simplified procedures presented by Seed and Idriss (1971). In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. We corrected the field measured SPT blow counts for the overburden, stress reduction versus depth, fine-grained soil content, hammer energy ratio, boring diameter, rod length and sampling method (SPT sampler without liners). The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65\,(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

Where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. We evaluated the liquefaction potential of the medium dense sand and silt strata encountered below the design ground water depth using a peak ground acceleration of 0.61g (based on Equation 11.8-1 of ASCE 7-10) and moment magnitude of 7.5 (USGS 2014).

The factor of safety (FS) against liquefaction can be expressed as the ratio of the CRR to CSR. If the FS for a soil layer is less than 1.0, the soil layer is considered liquefiable during a moderate to large seismic event.

$$FS = CRR/CSR$$

Soils that have corrected SPT blow counts greater than 30 blows per foot are considered too dense to liquefy. Such soil layers have been screened out of the analysis and are not presented below. A summary of our SPT analysis is presented in the table below.

Table 1. Results of Liquefaction Analyses – SPT Method

Boring Number	Depth to Top of Sand Layer (feet)	Layer Thickness (feet)	SPT* $(N_1)_{60CS}$	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)
EB-1	5.0	2.8	29	0.9	Likely	0.3
	7.8	4.0	22	0.5	Likely	0.7
	11.8	3.0	25	0.5	Likely	0.5
	14.8	2.0	25	0.5	Likely	0.3
					Total =	**1.8**
EB-3	7.5	3.3	22	0.5	Likely	0.7
	11.8	5.0	25	0.5	Likely	0.6
					Total =	**1.3**

Our analyses indicate that some granular layers encountered below the design ground water depth may theoretically liquefy, resulting in approximately 1¼ to 1¾ inches of total settlement. Volumetric change and settlement were estimated using the Zhang, Robertson, and Brachman (2002) method.

We estimate differential settlements from liquefaction will be on the order of ½-inch in 50 horizontal feet. A detailed discussion of estimated settlements is presented in the "Foundations" section of this report.

3.4.3 Potential for Ground Rupture/Sand Boils

The methods of analysis used to estimate the total settlement assume that there is no possibility of surface ground rupture. For liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must be large enough to break through the surface layer. There is approximately 5 to 7½ feet of non-liquefiable material overlying the potentially liquefiable soils for borings EB-1 and EB-3, respectively. Based on the work by Youd and Garris (1995), it is our opinion that there is not adequate non-liquefiable material capping the site to prevent ground rupture. If ground rupture and sand venting were to occur, significantly higher ground deformation could occur. It is our opinion that ground rupture at the site should be considered during development. Ground rupture could potentially result is vertical and horizontal ground movements on the order of several feet on a localized basis.

3.5 Dry Seismic Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. Our borings encountered some medium dense cohesionless soils above the design ground water depth in borings EB-1 and EB-3. However, our calculations indicated that the medium dense sand layers encountered in the borings may densify and settle on the order of approximately less than ¼-inch. Therefore, we judge the probability significant differential settlement of non-saturated granular layers at the site to be low.

3.6 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable since it is difficult to evaluate where the first tension crack will occur.

The Oakland Estuary is located adjacent to the vacant land. Loose to medium dense sands with potential for liquefaction were encountered to depths ranging from approximately 14 to 17 feet. If liquefaction were to occur, the potential for lateral spreading would be moderate to high in localized areas. Lateral spreading could damage retaining walls, pavements and underground utilities on the site.

3.7 Overall Bulkhead Stability

The Mott MacDonald report indicated that a failure zone could extend approximately 30 feet behind the wall on the land side as a result of liquefaction and reduced soil strength. They recommended a setback distance of at least 30 feet behind the bulkhead wall if it is not replaced.

Based on the existing subsurface conditions identified in the borings, we have evaluated the global stability of the existing bulkhead and estimated the depth of penetration of the new bulkhead to provide overall factors of safety against ground movement failure. The depth of penetration of the existing sheet pile bulkhead wall below the mud line is unknown. We assume a depth of 10 feet in our evaluation.

The lateral stability of a slope is influenced by the composition, inclination, and height of a slope. Stability is usually expressed as a ratio of resisting moments and forces divided by driving moments and forces termed the factor of safety (FS). Factors of safety are calculated for static and seismic conditions.

The stability of the existing bulkhead structure was evaluated using the computer program Slope/W utilizing the Spencer method of analysis. Input parameters for the analysis include slope geometry, soil layers or zones, soil unit weights and strength parameters, and groundwater conditions.

The slope is first analyzed to establish the minimum factor of safety under static conditions. Once this failure surface is located, an additional, horizontal force acting in the direction of potential failure is imposed on the sliding mass for the "pseudo-static" method of analysis. This additional force is equal to the potential landslide mass multiplied by a seismic coefficient.

The seismic stability was evaluated in accordance with recommended procedures provided in Special Publication 117A (California Geological Survey, 2008). This method, which is known as a screening analysis, involves calculating a seismic coefficient and evaluating the stability. A slope is considered globally stable under seismic conditions when the analyses show a minimum FS greater than or equal to 1.0. The seismic coefficient is dependent on the contributing earthquake magnitude, distance to the fault, and the peak ground acceleration (PGA). The seismic coefficient used in our analysis. The minimum allowable FS with respect to slope stability generally ranges from 1.5 to 2.0 for static conditions and 1 to 1.3 for seismic conditions. DMG SP-117A recommends minimum factors of safety of 1.5 and 1.0 for static and seismic analyses, respectively (DMG SP-117A, 2008) when using the screening method.

The stability of the existing adjacent slope will be directly related to the characteristics of the soil materials encountered in the borings. For the clay soils present on the site, we used a unit weight of 120 pounds per cubic foot (pcf), a cohesion of 500 pounds per square foot (psf) and a friction angle of 0 degrees. For sands below the ground water level of 5 feet, we assumed an effective unit weight of 55 pcf and a residual friction angle of 15 degrees.

Due to the uncertainties of the depth of the bulkhead, the integrity of the bulkhead and tieback anchors, the overall factors of safety for stability of the bulkhead are likely less than the required 1.5 for static conditions and 1.5 for seismic conditions. For a new bulkhead structure, we recommend a minimum penetration of the bulkhead of at least 15 feet below the mudline.

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted three samples collected during our subsurface investigations to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The results of this test is summarized in Table 2 below.

Table 2. Results of Corrosivity Testing

Sample	Depth (feet)	Chloride (mg/kg)	Sulfate (mg/kg)	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity	Estimated Corrosivity Based on Sulfates
EB-1, 2A	4.0	409	313	7.4	723	Very Severely	Negligible
EB-2, 4A	9.5	1,406	117	7.5	289	Very Severely	Negligible
EB-3, 8B	24.5	10,244	222	8.3	92	Very Severely	Negligible

Notes: 1. mg/kg = milligrams per kilogram.

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 3 below.

Table 3. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table 4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 4.

Table 4. Relationship Between Sulfate Concentration and Sulfate Exposure (Table 4.2.1 of ACI)

Water-Soluble Sulfate (SO_4) in soil, ppm	Sulfate Exposure
0 to 1,000	Negligible
1,000 to 2,000	Moderate[1]
2,000 to 20,000	Severe
over 20,000	Very Severe

[1]= seawater

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity be with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 2, the soil resistivity results ranged from 92 to 723 ohm-centimeters. Based on this result and the resistivity correlations presented in Table 3, the corrosion potential to buried metallic improvements may be characterized as very severely corrosive. We recommend that a corrosion protection engineer be consulted about appropriate corrosion protection methods for buried metallic materials if necessary.

Based on our previous experience and Table 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered negligible for the native subsurface materials sampled.

5.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering viewpoint, the proposed improvements may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

5.1 Primary Geotechnical Concerns

The primary geotechnical and geologic concerns at the site are as follows:

- Strong seismic shaking
- Potential for liquefaction, ground rupture, and lateral spreading
- Shallow ground water
- Corrosion potential of the near-surface soils
- Fill soils
- Moderately expansive near surface soils
- Bulkhead stability

We have prepared a brief description of the issues and present typical approaches to manage potential concerns associated with the long-term performance of the development.

5.1.1 Strong Seismic Shaking

We recommend that, as a minimum, the proposed improvements be designed in accordance with the seismic design criteria presented in Table 5.

5.1.2 Potential for Liquefaction, Ground Rupture, and Lateral Spreading

Our analyses indicate that some layers theoretically can liquefy, ranging from 1¼ to 1¾ inches of total liquefaction induced settlement in the upper 50 feet. We estimate differential settlements from liquefaction will be on the order of ½-inch in 50 horizontal feet.

There is also a high potential for ground rupture to occur as there is not an adequate cap of non-liquefiable material overlying the liquefiable layers at the site. In addition, there is also high potential for liquefaction-induced lateral spreading to occur due to the close proximity of the Oakland Estuary. These phenomena could result in up to several feet or more of vertical and lateral movement of the ground surface. This could cause significant adverse effects to the proposed improvements, and short of site-wide ground improvement to reduce the liquefaction potential, there is no real mitigation method to reduce this potential. To help mitigate the potential effects of ground rupture, any underground utility connections will need to be flexible to accommodate potential ground movement.

The potentially liquefiable soils will greatly reduce the soil strength for the design of tie back anchors, anchor blocks or deadman for the bulkhead. Specific recommendations are presented in Section 7.0.

We understand that the proposed revetment section would not be designed to resist liquefaction and lateral spread during earthquake shaking, but would be reconstructed if movement occurred. Therefore, the potential for liquefaction and lateral spreading would not be a factor in design, construction or performance of the revetment section.

5.1.3 Shallow Ground Water

As discussed in Section 2.4, ground water was encountered in our borings at depths of approximately 8½ to 10½ feet below the existing ground surface. We recommend a design depth to ground water of 5 feet. Therefore, ground water potentially can be encountered during excavations and/or utility installations. If excavations for the new underground utilities extend near or into ground water, localized dewatering and soil stabilization may be required.

5.1.4 Corrosion Potential of Near-Surface Soils

As discussed above, the corrosion potential to buried metallic improvements constructed within the fill and native soils may be characterized as very severely corrosive. A qualified corrosion engineer should be contacted to provide specific recommendations regarding corrosion protection for buried metal pipe or buried metal pipe-fittings.

5.1.5 Fill Soils

As discussed above, fill was encountered in all of our borings to depths ranging from approximately 5 to 17 feet below the ground surface. Based on the lab results, these soils do not appear to be relatively well compacted so over-excavation is recommended prior to construction of the proposed improvements. However, they can be reused as compacted fill if desired.

5.1.6 Moderately Expansive Soils

To reduce the potential for damage to the planned improvements due to the presence of moderately expansive surficial soils, we recommend slabs-on-grade have sufficient reinforcement and be supported on a layer of non-expansive fill and that any shallow foundations extend below the zone of seasonal moisture fluctuation. Detailed recommendations are presented in the following sections of this report.

5.1.7 Bulkhead Stability

As discussed above the global stability factors of safety of the existing bulkhead wall are likely less than acceptable values. The Mott Macdonald report recommended a setback distance of 30 feet if the existing bulkhead will remain. If a new bulkhead is constructed, it should extend to a depth of at least 15 feet below the existing mud line in front of the wall.

5.2 Design-Level Geotechnical Investigation

Once final plans have been prepared, we recommend that our firm perform a design-level geotechnical investigation for the project. Review of the of the geotechnical aspects of the project design for general conformance with our recommendations should also be performed. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical aspects of the project construction.

The following sections provide feasibility-level recommendations for the proposed improvements.

6.0 EARTHWORK

6.1 Clearing and Site Preparation

The proposed project area should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, utilities, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

After clearing, any vegetated areas within the proposed improvements should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

6.2 Removal of Undocumented Fill

As discussed above, fill was encountered to a depths of up to 17 feet, and does not appear to be well compacted. We recommend the upper two feet (below subgrade or finish grade-whichever is lower) of existing, should be over-excavated to at least 5 feet beyond the proposed footprint and replaced with engineered fill. Before placing fill, the exposed soils should be scarified to a depth of 8 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report. If the excavated fill material meets the requirements in the "Material for Fill" section, it may be reused as engineered fill.

6.3 Abandoned Utilities

Abandoned utilities within the proposed project areas should be removed in their entirety. As an alternative, it may be feasible to abandon underground utilities in-place within the proposed project areas provided the utility does not conflict with new improvements, is completely grouted, and previous fills associated with the utility do not pose a risk to the structures. Existing underground utilities outside the proposed project areas may be removed or abandoned in-place by grouting or plugging the ends with concrete. The decision to abandon in-place versus removal should be based on the level of risk associated with the particular utility line.

Fills associated with underground utilities abandoned in-place may have an increased potential for settlement, and partially grouted or plugged pipelines will have a potential risk of collapse that may result in ground settlement, soil piping and leakage of pipeline constituents. The potential risks are relatively low for small diameter pipes (4 inches or less) above the ground water table and increasingly higher with increasing diameter.

6.4 Subgrade Preparation

After the site has been properly cleared, stripped, and necessary excavations have been made, exposed surface soils in those areas to receive fill, or slabs on grade should be scarified to a depth of 8 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and relatively non-yielding under the weight of compaction equipment.

6.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Import fill material should be inorganic, have a PI of 15 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Non-expansive fill (NEF) should have a PI of 15 or less. Samples of the proposed import fill should be submitted to us at least 10 working days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

6.6 Reuse of On-site Recycled Materials

If it is desired, the existing asphalt concrete and portland cement concrete may be used as engineered fill provided the materials are ground up and thoroughly mixed with on-site or import soil. In general, recycled concrete should be ground down to less than 4 inches in greatest dimension, with no more than 25 percent larger than 2½ inches. Recycled material should be thoroughly mixed with a sufficient amount of soil, such that there is no more than 40 percent by weight of recycled material in the final mix. Recycled material containing asphalt concrete should not be placed below habitable structures.

Recycled concrete could possibly be used as aggregate base or subbase material provided suitable crushing and/or screening equipment is used to process the concrete. Laboratory tests should be performed on samples of recycled aggregate material prior to placement in street or parking areas. Consideration should be given to potential issues associated with the environmental characteristics of the recycled materials, especially beneath buildings and other habitable locations.

6.7 Compaction

All fill, as well as scarified surface soils in those areas to receive fill, should be uniformly compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content near the laboratory optimum, except for the native expansive clays. The native expansive clays should be compacted to between 87 and 92 percent relative compaction at a moisture content at least 3 percent over optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and relatively non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition), except for the native expansive clays, which should be compacted as noted above. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum moisture content.

6.8 Wet Soils and Wet Weather Conditions

Earthwork such as subgrade preparation, fill placement and trench backfill may be difficult for soil containing high moisture content or during wet weather. If the soil is significantly above its optimum moisture content, it will become soft, yielding, and difficult to compact. Based on the results of our laboratory tests, the in-situ moisture contents of the near surface soils are generally near to above optimum moisture contents. If saturated soils are encountered, aerating or blending with drier soils to achieve a workable moisture content may be required. We recommend that earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.

There are several alternatives to facilitate subgrade preparation, fill placement and trench backfill if the soil is wet or earthwork is performed during the wet winter season.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,
- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,
- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

6.9 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements of the governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. If expansive soil is used for trench backfill, it should be compacted to between 87 to 92 percent at a moisture content at least 3 percent over optimum. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas, and coming into contact with subgrade soils.

6.10 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards. On a preliminary basis, site soils can be classified as Type C based on soil classification by OSHA. Therefore a maximum slope 1.5:1 (horizontal:vertical) should be anticipated. A TRC representative should be retained to verify soil conditions in the field at the time of the excavation.

6.11 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscaping and 0.5 percent in pavement areas, should be provided to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

6.12 Landscaping Considerations

We recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or drip irrigation systems
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeters.

We recommend that the landscape architect consider these items when developing the landscaping plans.

7.0 FOUNDATIONS

We recommend that any proposed structures be supported on shallow foundations, provided the estimated settlements discussed below are acceptable. Recommendations for shallow foundations are presented in Section 7.2.

7.1 2016 California Building Code Site Class and Site Seismic Coefficients

Chapter 16 of the 2016 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on our exploration, the site is generally underlain by soft to very stiff fine-grained soils, and loose to very dense sands which corresponds to a soil profile type E. Based on the above information and local seismic sources, the site may be characterized for design using the information in Table 5 below.

Table 5. 2016 CBC Site Class and Site Seismic Coefficients

Latitude: 37.7871 N **Longitude: -122.2486 W**	**CBC Reference**	**Factor/ Coefficient**	**Value**
Soil Profile Type	Section 1613.3.2	Site Class	E
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.3.1(1)	S_s	1.77
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.3.1(2)	S_1	0.70
Site Coefficient	Table 1613.3.3(1)	F_a	0.90
Site Coefficient	Table 1613.3.3(2)	F_v	2.40
Adjusted MCE Spectral Response Parameter	Equation 16-37	S_{MS}	1.59
Adjusted MCE Spectral Response Parameter	Equation 16-38	S_{M1}	1.69
Design Spectral Response Acceleration Parameter	Equation 16-39	S_{DS}	1.06
Design Spectral Response Acceleration Parameter	Equation 16-40	S_{D1}	1.12

7.2 Footings

Any proposed structures should be supported on conventional isolated or strip footing foundations bearing on engineered fill. All footings should have a minimum width of at least 12 inches and footing bottoms should extend at least 18 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

Continuous strip footings and isolated spread footings constructed on native soil or engineered fill in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. It is essential that we observe the all footing excavations before reinforcing steel is placed. If loose soils are encountered, they should be removed from the bottom of the footings.

7.2.1 Footing Foundation Settlement

Based on the estimated loads and the maximum allowable bearing pressures recommended above, we estimate that total static settlement for shallow footings will be approximately ½-inch, with differential settlements of ¼-inch over a horizontal distance of 25 feet. There is a potential for up to 1¾-inch of liquefaction induced settlement at the site with associated differential settlement on the order of ½-inch across a horizontal distance of 50 feet is also possible.

7.2.2 Lateral Loads on Footings

Lateral loads may be resisted by friction between the bottom of footings and the supporting subgrade. A maximum allowable frictional resistance of 0.30 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against footings poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

7.3 Bulkhead Design

As discussed above, significant horizontal and vertical soil movement could occur at the site as a result of liquefaction during earthquake shaking. Reduced soil strengths are anticipated due to liquefaction, and tie back anchors or deadman anchors may not mobilize enough strength to resist lateral movements.

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards. On a preliminary basis, site soils can be classified as Type C based on soil classification by OSHA. Therefore a maximum slope 1.5:1 (horizontal:vertical) should be anticipated. A TRC representative should be retained to verify soil conditions in the field at the time of the excavation.

The new bulkhead structure should penetrate at least 15 feet below the existing mud line along the wall. We should have the opportunity to review the final design.

7.4 Permanent Bulkhead Support System

As previously discussed, the existing sheet pile bulkhead may be replaced. The bulkhead height will likely be on the order of 15 feet above the mudline. The excavations could potentially be permanently supported by several methods including tiebacks, soil nailing, braced shoring, permanent slopes if space is adequate, or potentially other methods. Where shoring is required, restrained shoring will most likely be necessary to limit deflections. It has been our experience that cantilever shoring might be feasible for temporary shoring to a height of about 10 feet where allowable deflections are limited. The choice of shoring method should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice of support.

The permanent shoring should be designed for additional surcharges due to adjacent loads such as from construction vehicles and street traffic. To prevent excessive surcharging of the walls, we recommend that heavy loads such as construction equipment and stockpiles of materials be kept at least 15 feet from the top of the excavations. If this is not possible, the shoring must be designed to resist the additional anticipated lateral loads. Shoring systems should be designed with sufficient rigidity to prevent detrimental lateral displacements. Minimum geotechnical parameters for design of a permanent shoring system are given in Table 2.

Table 6. Permanent Bulkhead System Design Parameter

Design Parameter	Design Value (psf)
Minimum Lateral Wall Surcharge[1]	120 psf
Earth Pressure – Cantilever Wall	45 pcf
Earth Pressure – Restrained Wall[2] From ground surface to H/4 (ft)	Increase from 0 to 25H psf
Earth Pressure – Restrained Wall Below H/4 (ft)	Uniform pressure of 25H psf
Passive Pressure[3]	200 pcf up to 600 psf max

Note: 1 For the upper 5 feet (minimum for incidental loading)
2 Where H equals height of excavation
3 Can assume to act over 2 times the diameter of soldier piles, neglecting the upper foot

To limit potential movements of the shoring system, the shoring designer and contractor should consider several design and construction issues. For the movements of shoring to be reduced, the designer will have to provide for a uniform and timely mobilization of the soil pressures. Tiebacks or internal bracing should be loaded to the design loads prior to excavation of the adjacent soil so that load induced strains in the retaining system will not result in the system moving toward the excavation. In addition, a relatively stiff shoring system should be designed to limit deflections under loading. In general, we recommend designing a shoring system to deflect less than 1-inch.

In addition, ground subsidence and deflections can be caused by other factors such as voids created behind the shoring system by over-excavation, soil sloughing, erosion of sand or silt layers due to perched water, etc. All voids behind the shoring system should be filled as soon as feasible by grouting to minimize potential problems during installation of the shoring system.

Since the borings were drilled with hollow-stem auger drilling equipment, we are not able to evaluate the potential for caving of on-site soils, which may become a factor during soldier pile and/or tieback installation. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes using proposed production drilling equipment may be prudent, to evaluate possible excavation difficulties such as caving soils, cobbles, boulders and/or other excavation difficulties.

The contractor is also responsible for site safety and the means and methods of construction, including temporary shoring. Temporary and permanent shoring must be designed by a licensed California Civil or Structural Engineer. Prior to construction, we recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review.

Tie Backs/Anchors

Because of the shallow depth to ground water and the potentially liquefiable soils, anchors or tiebacks will have relatively low strength to resist lateral loads. In fact, they may not be able to mobilize sufficient lateral capacity to support the bulkhead.

The bonded length of the tie-backs should begin at least 5 feet behind a plane inclined at an angle of 35 degrees from vertical originating at the base of the wall.

The ultimate bond strength should be 2 psi to account for the residual strength of the potentially liquefiable soils. Minimum bond length should be 10 feet. Diameter of the drill hole should be at least 8 inches.

Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 4 times their nominal diameters on center, no reduction in the capacity of the anchors needs to be considered due to group action.

For deadman anchors, we recommend a passive soil resistance of 200 psf be used.

Below are the L-Pile parameters for each of the three borings we drilled at the site for design of the lateral capacity of the bulkhead wall.

Boring EB-1			Boring EB-2			Boring EB-3		
Depth	Material	Strength	Depth	Material	Strength	Depth	Material	Strength
0-5	Sand	34°	0-10	Clay	500 psf	0-5	Sand	33°
5-10	Sand	34°	10-20	Clay	500 psf	5-17	Sand	15°
10-18	Sand	15°	20-27	Clay	1,000 psf	17-30	Clay	500 psf
18-20	Clay	750 psf	27-30	Clay	750 psf	30-40	Clay	2,000 psf
20-27	Sand	15°	30-40	Clay	2,000 psf	40-50	Clay	1,000 psf
27-34	Clay	1,500 psf	40-50	Clay	750 psf			
34-37	Sand	15°						
37-45	Clay	1,000 psf						
45-50	Clay	2,000 psf						

Depth is in feet below existing ground surface
Groundwater is assumed to be at a depth of 5 feet below ground surface
Soil unit weights (unsaturated) are Clay 115 pcf, Sand 110 pcf
Strength is soil cohesion in psf for clays and friction angle in degrees for sands. Sands below 5 feet have residual value representing reduced value resulting from liquefaction.

7.5 Exterior Concrete Flatwork and Sidewalks

Due to the presence of moderately expansive surficial soils, we recommend exterior concrete flatwork such as pedestrian walkways and sidewalks be supported on at least 6 inches of non-expansive fill (NEF). NEF may include aggregate base, crushed rock, quarry fines or import soil having a PI of 15 or less. The upper 6 inches of the NEF should consist of Class 2 aggregate base compacted to at least 90 percent relative compaction. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Exterior Portland Cement Concrete Pavements" section of this report. The subgrade soil should be moisture conditioned near the laboratory optimum and compacted to at least 90 percent relative compaction, except for the native expansive clays which should be compacted as described in the compaction section of this report. We recommend that exterior slabs be isolated from adjacent foundations and that adequate construction and control joints be used in design of the concrete slabs to control cracking inherent in concrete construction.

A pavement section for trash enclosures, if proposed, should consist of 6 inches of Portland cement concrete over 6 inches of Class 2 aggregate base rock, compacted to at least 95 percent relative compaction.

9.0 PAVEMENTS

9.1 Asphalt Concrete

Based on the near-surface soils encountered during our exploration, which consisted of medium dense clayey sand and clayey gravel, and moderately expansive soils, we judged an R-value of 5 to be applicable for design based on a subgrade consisting of untreated native soils. Using estimated traffic indices for various pavement-loading requirements and untreated native soils, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 7.

Table 7. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 5

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking Channel	5.0	3.0	10.0	13.0
	5.5	3.0	12.0	15.0
Truck Access & Parking Areas	6.0	3.5	12.5	16.0
	6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. If the pavement subgrade soils are expansive, increased maintenance and reduction in pavement life can be expected. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study. We recommend that R-value testing be performed on the actual subgrade soils once the grades have been raised. If testing indicates a significantly higher R-value, it may be feasible to reduce the design pavement sections.

Because the full thickness of asphalt concrete is frequently not placed prior to construction traffic being allowed to use the street, rutting and pavement failures can occur prior to project completion. To reduce this occurrence, we recommend that either the full design pavement section be placed prior to use by construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

In addition, it has been our experience that asphalt concrete pavements constructed over expansive soils and adjacent to non-irrigated open space areas may experience cracking parallel to the edge of the pavement. This is typically caused by seasonal shrinkage and swelling adjacent to non-irrigated edges of the pavement. The cracks typically occur within the first few years of construction, and are typically located within a few to several feet of the edge of the pavement. The cracks, if they occur, can be filled with a bituminous sealant. Otherwise, a moisture barrier would need to be installed to a depth of at least 24 inches to reduce the potential for shrinkage of the pavement subgrade soils.

9.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

10.0 LIMITATIONS

This report has been prepared for the sole use of East Bay Regional Park District Land Acquisition, specifically for feasibility-level design of the proposed improvements described in the report at 1441 – 1551 Embarcadero in Oakland, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC's report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

11.0 REFERENCES

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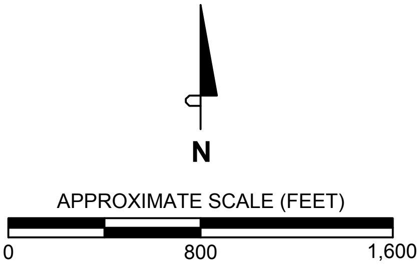
SOURCE AERIAL PHOTO: Google Earth, March 2017.



VICINITY MAP

Former Crowley Marine Property
1441-1551 Embarcadero
Oakland, California

TRC | 243805.0004 | **FIGURE 1**

FILE NAME: Z:\Shared\CAD_DRAW\Current\Former Crowley Marine Property\Geotechnical Report\Fig1_Vicinity Map.dwg | Layout Tab: 8x11

SOURCE AERIAL PHOTO: Google Earth, March 2017.

LEGEND

⊕ Approximate location of exploratory boring

N

APPROXIMATE SCALE (FEET)

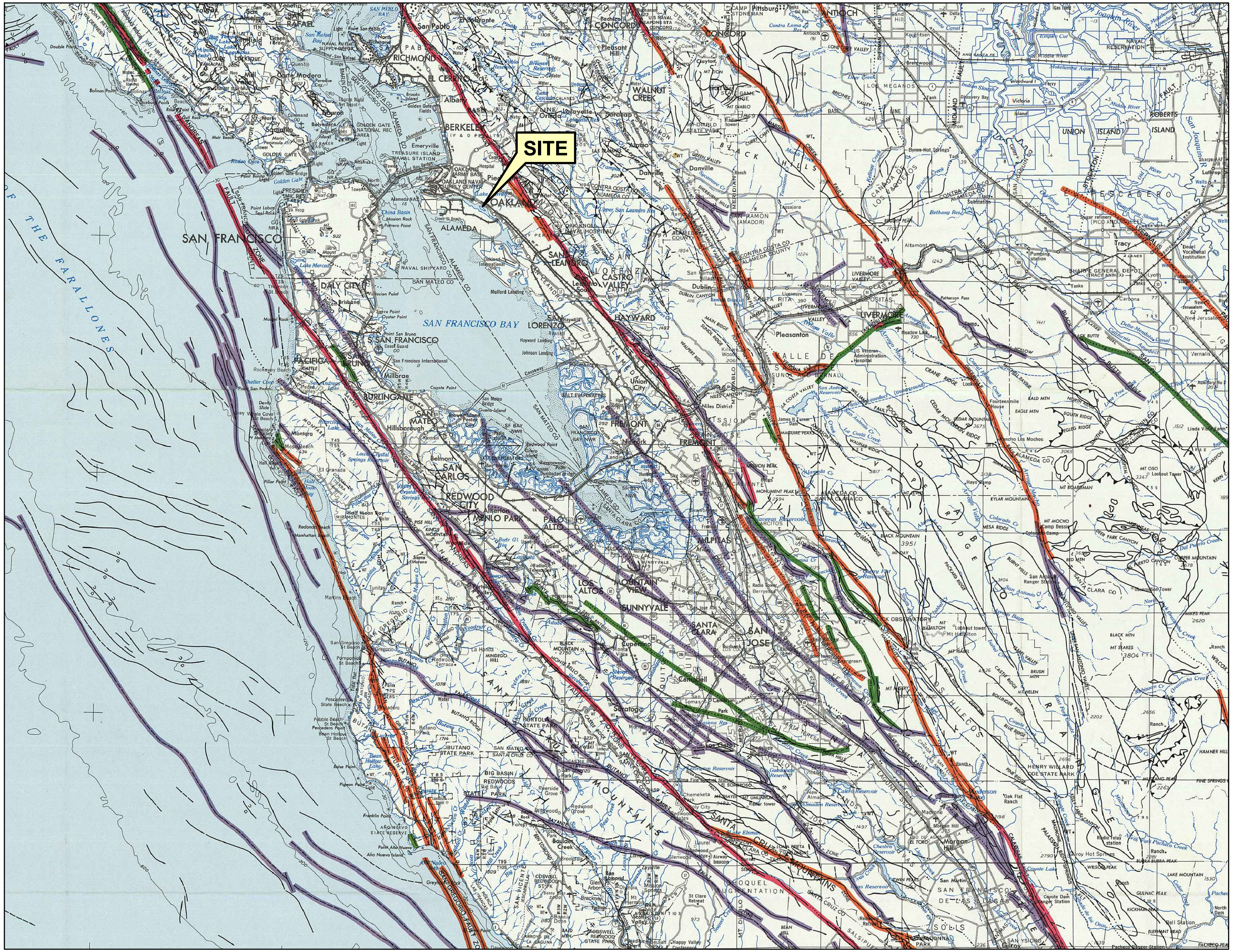
0 100 200

SITE PLAN

Former Crowley Marine Property
1441-1551 Embarcadero
Oakland, California

TRC 243805.0004 **FIGURE 2**

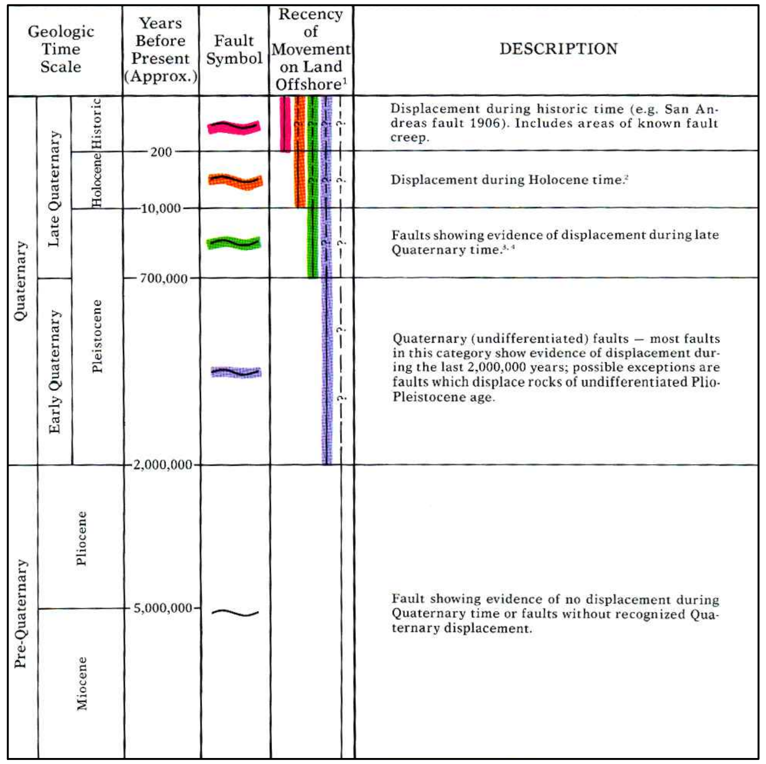
FILE NAME: Z:\Shared\CAD_DRAW\Current\Former Crowley Marine Property\Geotechnical Report\Fig2_Site Plan.dwg | Layout Tab: 8x11



SCALE: 1:500,000

APPROXIMATE GRAPHICAL SCALE (MILES)

0 10 20



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	200	🟧		Displacement during Holocene time.[2]
		Pleistocene	10,000	🟩		Faults showing evidence of displacement during late Quaternary time.[3, 4]
	Early Quaternary		700,000	🟪		Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		2,000,000			
	Miocene		5,000,000	~		Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

NOTES:

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

REGIONAL FAULT MAP

Former Crowley Marine Property
1441-1551 Embarcadero
Oakland, California

TRC

243805.0004

FIGURE 3

FILE NAME: Z:\Shared\CAD_DRAW\Current\Former Crowley Marine Property\Geotechnical Report\Fig3_Regional Fault Map.dwg | Layout Tab: 11x17

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mobile hollow-stem auger drilling equipment. Three 8-inch diameter exploratory borings were drilled on September 11, 2017 to a maximum depth of 50 feet. The approximate location of the exploratory borings are shown on Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The location of the borings were approximately determined by pacing from existing site boundaries. Elevation of the borings were not determined. The location should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the boring at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Relatively undisturbed samples were also obtained with 2.875-inch inside diameter Shelby Tube sampler, which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the sampler the last two 6-inch increments. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The attached boring logs and related information depict subsurface conditions at the location indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at the boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	●	Well graded gravels, gravel–sand mixtures, little or no fines
			GP	○	Poorly graded gravels or gravel–sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	◌	Silty gravels, gravel–sand–silt mixtures, plastic fines
			GC	▨	Clayey gravels, gravel–sand–clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	⁘	Well graded sands, gravelly sands, little or no fines
			SP	░	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	▥	Silty sands, sand–silt–mixtures, non–plastic fines
			SC	▨	Clayey sands, sand–clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 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 silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH	▥	Inorganic 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 and other highly organic soils

DEFINITION OF TERMS

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

	200	40	10	4	3/4"	3"	12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

	0.08	0.4	2	5	19	76mm

GRAIN SIZES

☒	TERZAGHI SPLIT SPOON STANDARD PENETRATION
⧗	MODIFIED CALIFORNIA
▯	ROCK CORE
◩	PITCHER TUBE
○	NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0–4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0–1/4	0–2
SOFT	1/4–1/2	2–4
MEDIUM STIFF	1/2–1	4–8
STIFF	1–2	8–16
VERY STIFF	2–4	16–32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2–inch O.D. (1–3/8 inch I.D.) split spoon (ASTM D–1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D–1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D–2487)

TRC

FIGURE A–1

EXPLORATORY BORING: EB-1

Sheet 1 of 2

DRILL RIG: TRUCK MOBILE B-56	PROJECT NO: 243805
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: FORMER CROWLEY MARINE PROPERTY
LOGGED BY: AC	LOCATION: OAKLAND, CA
START DATE: 9-11-17 FINISH DATE: 9-11-17	COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	0	■	**3.5" of AC over 4.5" of AB**	AC/AB						
			CLAYEY GRAVEL WITH SAND [FILL] (GC) dense, moist, yellowish brown, medium plasticity, fine to coarse sand, fine gravel (sub-angular/rounded)	GC	48	⧗	9		16.6	
			medium dense		25	⧖				
	5		**CLAYEY SAND WITH GRAVEL [FILL] (SC)** loose, moist, reddish brown, medium plasticity, fine to coarse gravel (sub-angular/rounded)	SC	15	⧗	19		29.5	
▽			wet, gray		8	⧖	20			
	10									
			CLAYEY SAND WITH GRAVEL [FILL] (SC) loose, wet, gray, medium plasticity, fine to coarse sand, fine gravel (sub-angular/rounded), organics	SC	10	⧖				
			ELASTIC SILT [BAY MUD] (MH) stiff, moist, dark gray, high plasticity, trace organics and shells	MH						
	15		**SILTY SAND (SM)** loose, wet, grayish brown, low plasticity, fine to medium sand, trace shells	SM						
			SANDY LEAN CLAY (CL) medium stiff, moist, gray, medium plasticity, fine sand, trace shells, 100 psi	CL	12	⧗				△
	20					■	46	68	54.4	
			very stiff, greenish gray, no trace shells		42	⧗	13			
	25		**CLAYEY SAND (SC)** medium dense, moist, bluish gray, medium plasticity, fine sand, trace fine gravel (sub-angular/rounded)	SC						
			SANDY LEAN CLAY (CL) stiff, moist, olive gray, medium plasticity, fine to coarse sand, trace fine gravel (sub-angular/rounded)	CL	17	⧖	21			
	30									

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 9.0 FEET

▼ : FREE GROUND WATER MEASURED FOLLOWING DRILLING AT 37.0 FEET

LA_CORP.GDT 10/31/17 MV, CA*

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: TRUCK MOBILE B-56	PROJECT NO: 243805
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: FORMER CROWLEY MARINE PROPERTY
LOGGED BY: AC	LOCATION: OAKLAND, CA
START DATE: 9-11-17 FINISH DATE: 9-11-17	COMPLETION DEPTH: 50.0 FT.

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Undrained Shear Strength (ksf)

○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30	**SANDY LEAN CLAY (CL)** stiff, moist, olive gray, medium plasticity, fine to coarse sand, trace fine gravel (sub-angular/rounded)	CL					
		CLAYEY SAND WITH GRAVEL (SC) very dense, wet, brown, medium plasticity, fine to coarse sand, fine gravel (sub-angular/rounded)	SC	33 50/6			18.2	
	35							
▼		**LEAN CLAY WITH SAND (CL)** stiff, moist, olive gray, medium plasticity, trace fine sand	CL	10	26			
	40							
		LEAN CLAY (CL) medium stiff, moist, light olive gray, medium plasticity, shells	CL	15	50			○
	45							
		LEAN CLAY WITH SAND (CL) very stiff, moist, reddish gray, medium plasticity, fine sand	CL	36	30			○
	50	Bottom of boring at 50 feet						
	55							
	60							

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 9.0 FEET

▼ : FREE GROUND WATER MEASURED FOLLOWING DRILLING AT 37.0 FEET

EXPLORATORY BORING: EB-2

Sheet 1 of 2

DRILL RIG: TRUCK MOBILE B-56
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 9-11-17 FINISH DATE: 9-11-17

PROJECT NO: 243805
PROJECT: FORMER CROWLEY MARINE PROPERTY
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 50.0 FT.

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ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression 1.0 2.0 3.0 4.0
	0		**LEAN CLAY WITH SAND [FILL] (CL)** medium stiff, moist, dark olive gray, medium plasticity, fine sand, trace organics	CL	7		58	62		○
			SANDY LEAN CLAY [FILL] (CL) medium stiff, moist, greenish gray, medium plasticity, fine to coarse sand, few fine gravel (sub-angular/rounded), trace shells	CL	5		29		62.2	
	5		**ELASTIC SILT [BAY MUD] (ML)** soft, moist, dark gray, high plasticity, trace shells	MH	8		53	69		△
			LEAN CLAY WITH SAND (CL) medium stiff, moist, greenish gray, medium plasticity, fine sand, shells	CL						
			SANDY LEAN CLAY (CL) medium stiff, moist, olive gray, medium plasticity, fine sand, trace shells	CL	4		31			△
▼	10									
					5		17			
	15									
			SANDY LEAN CLAY WITH GRAVEL (CL) stiff, moist, olive gray, medium plasticity, fine sand, fine gravel (sub-angular/rounded)	CL	8		32	90		△
	20									
					8					△
▽	25									
			LEAN CLAY WITH SAND (CL) medium stiff, moist, dark gray, moderate plasticity, 100-120 psi	CL			27	97		△▲
			LEAN CLAY (CL) medium stiff, moist, dark gray, medium plasticity	CL	23		22			○
	30									

Continued Next Page

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 25.0 FEET
▼ : FREE GROUND WATER MEASURED FOLLOWING DRILLING AT 10.5 FEET

EXPLORATORY BORING: EB-2 Cont'd

Sheet 2 of 2

DRILL RIG: TRUCK MOBILE B-56	PROJECT NO: 243805
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: FORMER CROWLEY MARINE PROPERTY
LOGGED BY: AC	LOCATION: OAKLAND, CA
START DATE: 9-11-17 FINISH DATE: 9-11-17	COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0 2.0 3.0 4.0)

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30	▨	**SANDY LEAN CLAY (CL)** stiff, moist, dark gray, medium plasticity, fine sand	CL						
	35	▨	**LEAN CLAY WITH SAND (CL)** very stiff, moist, olive gray, medium plasticity, fine sand, few fine gravel (sub-angular/rounded)	CL	26	⧖	34			
	40	▨	**LEAN CLAY (CL)** stiff, moist, light olive brown, medium plasticity, trace fine gravel (sub-angular/rounded)	CL	16	⧗				
	45	▨	**LEAN CLAY (CL)** stiff, moist, light tan brown, medium plasticity, shells	CL	9	⧗				○
		▨	**CLAYEY SAND (SC)** dense, moist, brown, medium plasticity, fine sand, few fine gravel (sub-angular/rounded), trace medium to coarse sand	SC	63	⧗			29.3	
	50		Bottom of boring at 50 feet							
	55									
	60									

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 25.0 FEET

▼ : FREE GROUND WATER MEASURED FOLLOWING DRILLING AT 10.5 FEET

LA_CORP.GDT 10/31/17 MV, CA*

EXPLORATORY BORING: EB-3

Sheet 1 of 2

DRILL RIG: TRUCK MOBILE B-56
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 9-11-17 FINISH DATE: 9-11-17

PROJECT NO: 243805
PROJECT: FORMER CROWLEY MARINE PROPERTY
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf)
- ○ Pocket Penetrometer
- △ Torvane
- ● Unconfined Compression
- ▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

SURFACE ELEVATION:

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	0	**CLAYEY SAND WITH GRAVEL [FILL] (SC)** dense, moist, strong brown, medium plasticity, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	SC	49	12		31.2	
		medium dense		21				
	5	**CLAYEY GRAVEL WITH SAND [FILL] (GC)** medium dense, moist, olive brown, medium plasticity, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GC	24	10			
▽		**CLAYEY SAND WITH GRAVEL [FILL] (SC)** loose, wet, greenish gray, medium plasticity, fine sand, fine gravel (sub-angular/rounded), trace medium to coarse sand	SC	8				
	10							
		CLAYEY GRAVEL WITH SAND [FILL] (GC) medium dense, wet, dark gray, medium plasticity, fine sand, fine gravel (sub-angular/rounded)	GC	10				
	15							
		ELASTIC SILT [BAY MUD] (MH) medium stiff, moist, dark gray, high plasticity, trace shells		8	84	51		△
	20	soft	MH		88	50		▲△
		medium stiff, trace shells		7	90	49		△
	25							
		LEAN CLAY WITH SAND (CL) stiff, moist, bluish gray, medium plasticity, fine sand, trace medium to coarse sand and fine gravel (sub-angular/rounded)	CL	14	18	113		
	30							

Continued Next Page

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 8.5 FEET
▼ : FREE GROUND WATER MEASURED FOLLOWING DRILLING AT 35.0 FEET

EXPLORATORY BORING: EB-3 Cont'd

Sheet 2 of 2

DRILL RIG: TRUCK MOBILE B-56
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 9-11-17 FINISH DATE: 9-11-17

PROJECT NO: 243805
PROJECT: FORMER CROWLEY MARINE PROPERTY
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30	▨	**GRAVELLY LEAN CLAY (CL)** stiff, moist, bluish gray, medium plasticity, trace fine sand, fine gravel (sub-angular/rounded)	CL						
		▨	**LEAN CLAY (CL)** very stiff, moist, brown, medium plasticity, trace fine sand	CL	29	⧖				
▼	35	▨								
		▨	**LEAN CLAY WITH SAND (CL)** stiff, moist, light olive brown, medium plasticity, fine sand	CL	26	⧗	28			○
	40	▨								
		▨	**LEAN CLAY (CL)** stiff, moist, light brown, medium plasticity, shells	CL	17	⧗				○
	45	▨								
		▨	**SANDY LEAN CLAY (CL)** very stiff, moist, brown, medium plasticity, fine sand	CL	39	⧖	20	97		
	50		**SILTY SAND (SM)** dense, moist, olive brown, medium plasticity, fine sand Bottom of boring at 50 feet	SM						
	55									
	60									

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 8.5 FEET
▼ : FREE GROUND WATER MEASURED FOLLOWING DRILLING AT 35.0 FEET

LA_CORP.GDT 10/31/17 MV, CA*

APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was measured (ASTM D2216) on samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density tests (ASTM D2937) were performed on samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was performed on samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Unconsolidated Undrained Triaxial Shear Test: Unconsolidated Undrained Triaxial Shear tests were performed to find the undrained shear strength of clayey samples. Results of these tests are shown on the logs at the appropriate depths and included as part of this appendix.

* * * * * * * * * * *

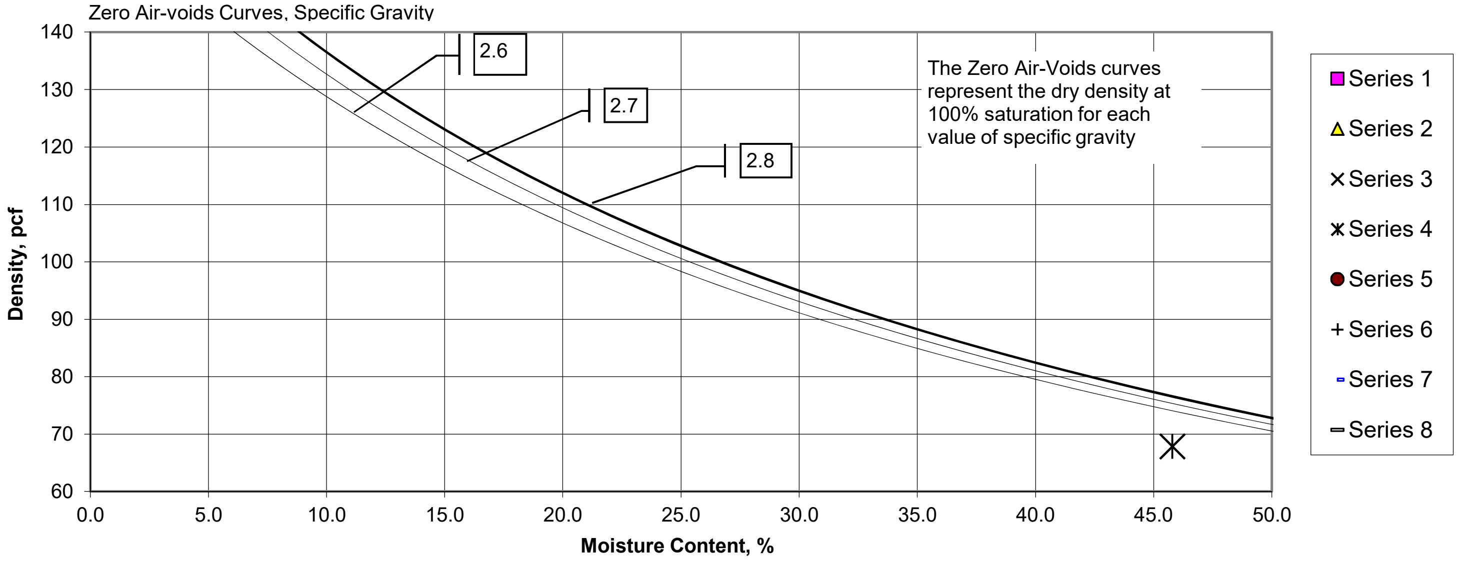
Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No: 028-2722a **Project No.** 243805 **By:** RU
Client: TRC **Date:** 09/28/17
Project Name: Former Crowley Marine **Remarks:**

Boring:	EB-1	EB-1	EB-1	EB-1	EB-1	EB-1	EB-1	EB-1
Sample:	1B	3B	4A	7A	8A	9A	11A	12A
Depth, ft:	2.0	6.0	9.5	21.0	24.5	29.5	39.5	44.5
Visual Description:	Yellowish Brown Clayey GRAVEL w/ Sand	Reddish Brown Clayey SAND w/ Gravel	Gray Sandy CLAY w/ Gravel	Gray Sandy CLAY	Bluish Gray Clayey SAND w/ Gravel	Olive Gray Sandy CLAY	Olive Gray CLAY w/ Sand	Light Olive Gray CLAY w/ shells
Actual G_s								
Assumed G_s				2.70				
Moisture, %	9.2	18.9	20.1	45.8	13.3	21.3	25.9	49.9
Wet Unit wt, pcf				98.9				
Dry Unit wt, pcf				67.8				
Dry Bulk Dens.ρb, (g/cc)				1.09				
Saturation, %				83.2				
Total Porosity, %				59.8				
Volumetric Water Cont,Θw,%				49.7				
Volumetric Air Cont., Θa,%				10.1				
Void Ratio				1.49				
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

Moisture-Density



The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

Moisture-Density-Porosity Report

Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No: 028-2722b **Project No.** 243805 **By:** RU

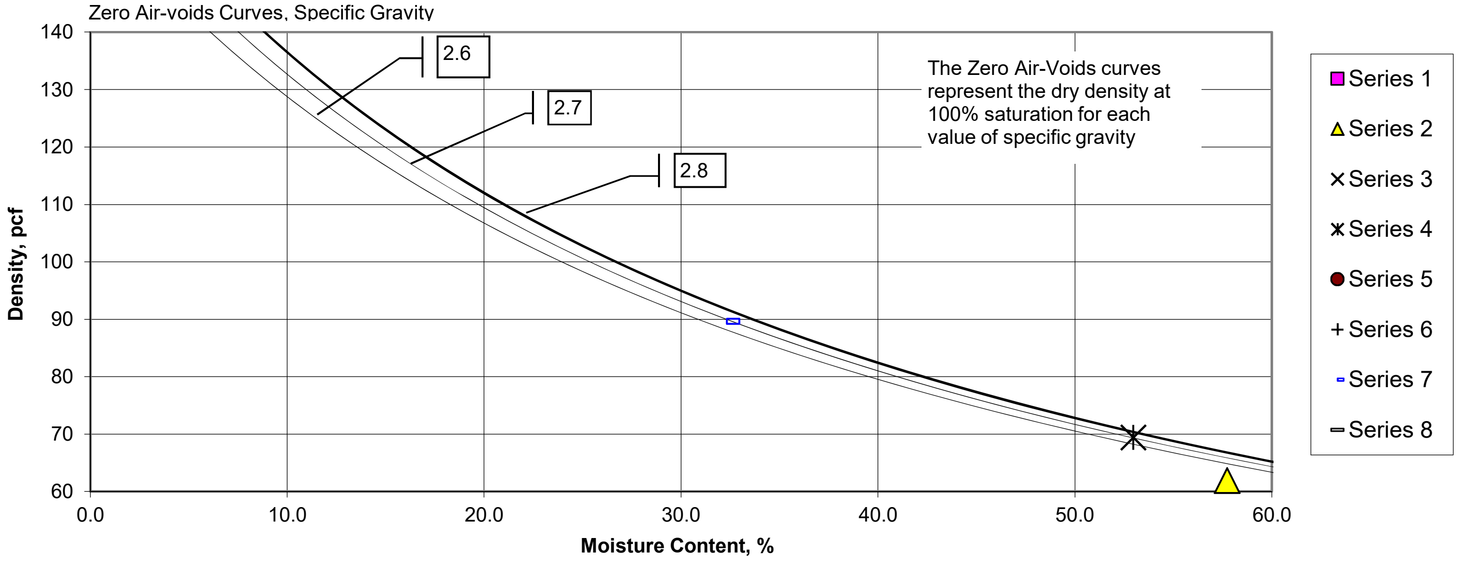
Client: TRC **Date:** 09/28/17

Project Name: Former Crowley Marine **Remarks:**

Boring:	EB-1	EB-2	EB-2	EB-2	EB-2	EB-2	EB-2	EB-2
Sample:	13B	1A	2A	3B	4A	5A	6B	9A
Depth, ft:	49.5	2.0	4.0	6.0	9.5	14.5	19.5	29.5
Visual Description:	Reddish Gray CLAY w/ Sand	Black Peat grading to Dark Olive Gray CLAY w/ Sand	Greenish Gray Sandy CLAY	Dark Gray CLAY w/ Sand	Olive Gray Sandy CLAY	Olive Gray Clayey SAND	Olive Gray Sandy CLAY w/ Gravel	Dark Gray Sandy CLAY
Actual G_s								
Assumed G_s		2.70		2.70			2.70	
Moisture, %	29.9	57.7	28.9	53.0	31.1	17.4	32.3	21.6
Wet Unit wt, pcf		97.6		106.2			118.6	
Dry Unit wt, pcf		61.9		69.4			89.6	
Dry Bulk Dens.ρb, (g/cc)		0.99		1.11			1.44	
Saturation, %		90.2		100.0			99.0	
Total Porosity, %		63.3		58.9			46.9	
Volumetric Water Cont,Θw,%		57.1		58.8			46.4	
Volumetric Air Cont., Θa,%		6.2		0.0			0.5	
Void Ratio		1.73		1.43			0.88	
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

Moisture-Density

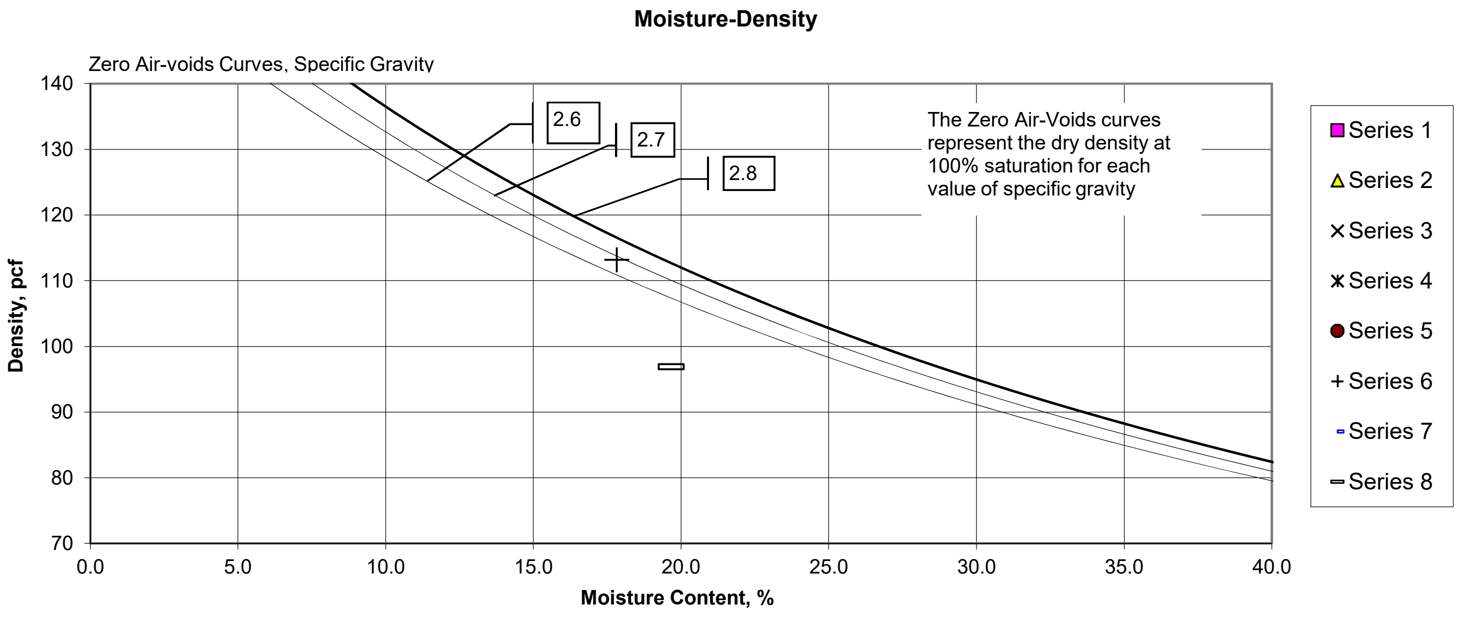


Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No: 028-2722c **Project No.** 243805 **By:** RU
Client: TRC **Date:** 09/28/17
Project Name: Former Crowley Marine **Remarks:**

Boring:	EB-2	EB-3	EB-3	EB-3	EB-3	EB-3	EB-3	EB-3
Sample:	10A	1B	3A	6B	8B	9A	11A	13A
Depth, ft:	36.0	2.0	6.0	19.5	24.5	29.5	39.5	49.5
Visual Description:	Olive Gray CLAY w/ Sand	Strong Brown Clayey SAND w/ Gravel	Olive Brown Clayey GRAVEL w/ Sand	Gray CLAY	Gray CLAY	Bluish Gray Clayey SAND w/ Gravel	Light Olive Brown CLAY w/ Sand	Olive Brown Silty SAND
Actual G_s								
Assumed G_s				2.70	2.70	2.70		2.70
Moisture, %	33.6	11.9	9.5	84.3	89.5	17.8	27.9	19.7
Wet Unit wt, pcf				94.4	93.2	133.3		116.0
Dry Unit wt, pcf				51.2	49.2	113.1		96.9
Dry Bulk Dens.ρb, (g/cc)				0.82	0.79	1.81		1.55
Saturation, %				99.3	99.5	98.1		71.7
Total Porosity, %				69.6	70.8	32.9		42.6
Volumetric Water Cont,Θw,%				69.2	70.5	32.3		30.5
Volumetric Air Cont., Θa,%				0.5	0.4	0.6		12.0
Void Ratio				2.29	2.43	0.49		0.74
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.



COOPER
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#200 Sieve Wash Analysis
ASTM D 1140

Job No.: 028-2722
Client: TRC
Project: Former Crowley Marine
Project No.: 243805
Date: 10/2/2017
Run By: MD
Checked By: DC

Boring:	EB-1	EB-1	EB-1	EB-1	EB-2	EB-2	EB-3	
Sample:	1B	3B	7A	10B	2A	13A	1B	
Depth, ft.:	2.0	6.0	21.0	34.0	4.0	49.5	2.0	
Soil Type:	Yellowish Brown Clayey GRAVEL w/ Sand	Reddish Brown Clayey SAND w/ Gravel	Gray Sandy CLAY	Yellowish Brown Clayey SAND w/ Gravel	Greenish Gray Sandy CLAY	Yellowish Brown Clayey SAND	Strong Brown Clayey SAND w/ Gravel	
Wt of Dish & Dry Soil, gm	713.6	559.7	617.0	530.0	464.7	572.4	590.9	
Weight of Dish, gm	171.0	171.3	174.2	176.6	176.5	175.0	175.6	
Weight of Dry Soil, gm	542.6	388.4	442.9	353.4	288.2	397.4	415.4	
Wt. Ret. on #4 Sieve, gm	303.9	122.7	9.1	88.6	37.5	43.2	133.4	
Wt. Ret. on #200 Sieve, gm	452.6	273.8	201.8	289.0	109.1	281.0	285.9	
% Gravel	**56.0**	**31.6**	**2.1**	**25.1**	**13.0**	**10.9**	**32.1**	
% Sand	**27.4**	**38.9**	**43.5**	**56.7**	**24.8**	**59.8**	**36.7**	
% Silt & Clay	**16.6**	**29.5**	**54.4**	**18.2**	**62.2**	**29.3**	**31.2**	

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

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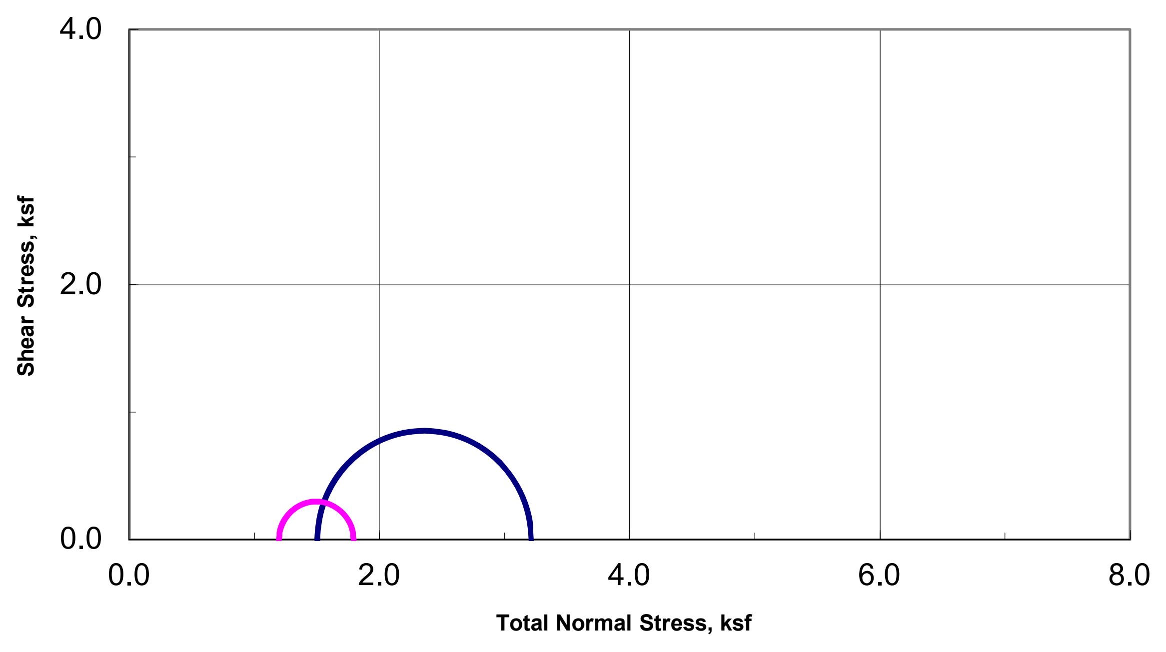
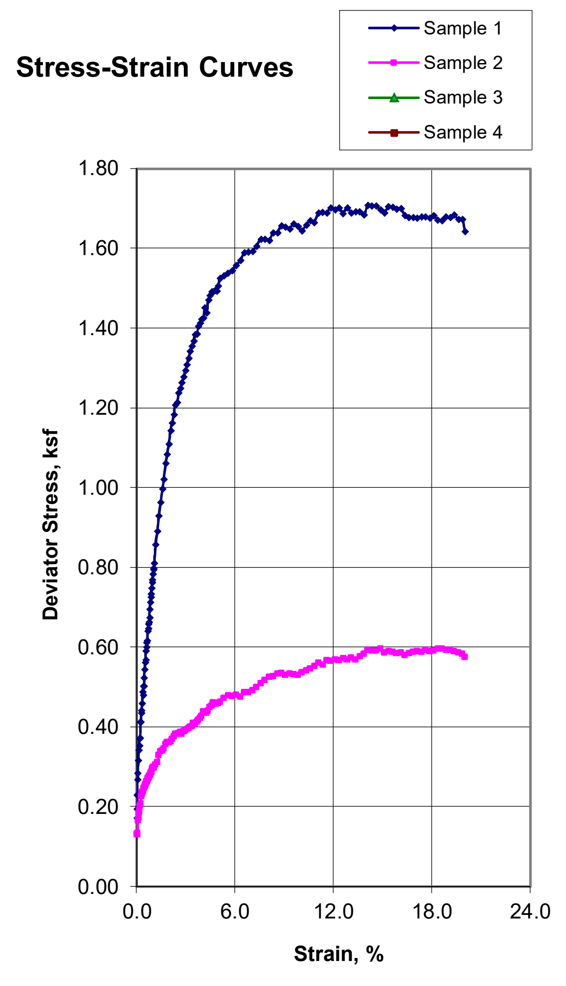
Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data

	1	2	3	4
Moisture %	27.3	87.6		
Dry Den,pcf	96.5	49.8		
Void Ratio	0.748	2.386		
Saturation %	98.6	99.1		
Height in	6.04	6.08		
Diameter in	2.87	2.86		
Cell psi	10.4	8.3		
Strain %	14.09	15.00		
Deviator, ksf	1.708	0.596		
Rate %/min	0.99	1.00		
in/min	0.059	0.061		
Job No.:	**028-2722**			
Client:	**TRC**			
Project:	**Former Crowley Marine - 243805**			
Boring:	EB-2	EB-3		
Sample:	8A	7A		
Depth ft:	25.5	20.5		

Visual Soil Description

Sample #	
1	Very Dark Gray CLAY w/ Sand
2	Gray CLAY w/ shells (Bay Mud)
3	
4	

Remarks:

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

Corrosivity Tests Summary

CTL # 028-2722 **Date:** 9/29/2017 **Tested By:** PJ **Checked:** PJ

Client: TRC **Project:** Former Crowley Marine **Proj. No:** 243805

Remarks:

Sample Location or ID			**Resistivity @ 15.5 °C (Ohm-cm)**			**Chloride**	**Sulfate**		**pH**	**ORP**		**Sulfide**	**Moisture**	**Soil Visual Description**
			As Rec.	**Min**	**Sat.**	**mg/kg**	**mg/kg**	**%**		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	**%**	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-1	2A	4.0	-	-	723	409	313	0.0313	7.4	-	-	-	13.7	Olive Brown Clayey SAND w/ Gravel
EB-2	4A	9.5	-	-	289	1,406	117	0.0117	7.5	-	-	-	31.1	Olive Gray Clayey SAND/ Sandy CLAY
EB-3	8B	24.5	-	-	92	10,244	222	0.0222	8.3	-	-	-	89.5	Gray CLAY