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ENGINEERING GROUP

October 17, 2017
File: 2174.008altr.doc

Mid- Cal Construction
2716 E. Miner Ave., Suite S
Stockton, CA 95205

Attn: Mr. Greg Pederson

Re: Geotechnical Investigation
Foundation Upgrade
12887 Sir Francis Drake
Inverness, California

Introduction

This letter summarizes the results of our Phase 1 Geotechnical Investigation for the planned foundation upgrades to lift the existing structure at project at 12887 Sir Francis Drake in Inverness, California. A site location map is shown on Figure 1. The purpose of our services is to evaluate existing geologic and geotechnical conditions and prepare geotechnical recommendations for use in project planning and design.

The scope of our Phase 1 services is outlined in our proposal letter dated May 2, 2017, and includes review of readily-available geotechnical and geologic reference material, one day of subsurface exploration with 1 or 2 soil borings, laboratory testing of recovered samples, engineering analysis, and preparation of this report. Issuance of this report completes our Phase 1 scope of services. Future phases or work are anticipated to include geo-civil design and construction observation and testing.

Project Description

The proposed project consists of lifting the existing structures approximately 3 feet in order to raise the structure out of the flood range. The existing structure appears to be supported on shallow foundations. A site plan showing the proposed improvements is presented on Figure 2.

Regional Geology

Marin County lies within the Coast Ranges geomorphic province of California, a region characterized by active seismicity and abundant landsliding and erosion. The regional basement bedrock geology consists of sedimentary, igneous, and metamorphic rock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex and marine sedimentary strata of the Great Valley Sequence, which is of similar age. Within central and northern California, the Franciscan and Great Valley rocks are locally overlain by a variety of Late Cretaceous and Tertiary-age sedimentary and volcanic rocks which have been deformed by various episodes of folding and faulting. The youngest geologic units in the region are Quaternary-age (last 1.8 million years) sedimentary deposits. These unconsolidated deposits partially fill many of the valleys of the region.

Regional geologic mapping[1] indicates that the project site is underlain by alluvial deposits, which are typically comprised of unconsolidated, moderately- to poorly-sorted clay, silt, sand, and gravel deposited in stream channel, terrace, or fan environments. A regional geologic map is shown on Figure 3.

Subsurface Exploration and Laboratory Testing

Subsurface exploration at the site was performed on August 30, 2017 with 2 soil borings excavated at the approximate locations shown on Figure 2. Borings were excavated to maximum explored depths of 31.5-feet below the ground surface by use of a portable hydraulic drill rig equipped with 6-inch hollow stem augers. Materials encountered were examined and logged by our geologist, and select samples were retained for laboratory testing. A brief explanation of the terms and methodology used in logging earth materials are shown on the Soil Classification Chart and Rock Classification Chart, Figures A-1 and A-2, respectively. Exploratory boring logs are shown on Figures A-3 through A-6.

Laboratory testing included determination of moisture content, dry density, unconfined compressive strength, percentage of particles passing the No. 200 (75-µm) sieve, and plasticity index, in general accordance with applicable ASTM procedures. Laboratory test results are presented on the boring logs, excepting the plasticity index results, which are shown on Figure A-7. The subsurface exploration and laboratory testing programs are discussed in further detail in Appendix A.

Subsurface Conditions

Boring 1 was drilled next to the lower level deck in the front of the residence as shown on Figure 2. Boring 1 encountered alluvial deposits composed of loose sand with clay to a depth of about 6-feet. At 6-feet, medium stiff, sandy silt was encountered. Boring 1 was terminated at a maximum explored depth of 24.0-feet below the ground surface and did not encounter bedrock.

Boring 2 was drilled on the back deck in between the workshop and the pier as shown on Figure 2. Boring 2 encountered very loose sand with clay to a depth of about 15.0-feet. The sand grades medium dense below 15.0-feet. At 26.5-feet, sandstone bedrock was encountered, and Boring 2 was terminated at a maximum explored depth of 31.5-feet below the ground surface.

Groundwater was encountered during our exploration at depths of about 3.0- and 1.0-feet in Borings 1 and 2, respectively. Since the borings were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. It is anticipated that the groundwater level will generally correspond to water levels in Tomales Bay; we judge groundwater should generally be expected within about 1.0-feet of the ground surface during the winter months and following periods of heavy rain.

[1] Thomas E. Gay Jr, T.C., “Geology of the Tomales Bay Study Area”, California Department of Conservation, Division of Mines and Geology Open-File Report 77-15 S.F. (Plate 2), Map Scale 1:12,000.

Geologic Hazards Evaluation

Based on our site reconnaissance, subsurface exploration, and literature review, we have evaluated commonly-considered geologic hazards that may affect the proposed project. Based on our review, the primary hazards which may affect the proposed improvements are strong seismic ground shaking, liquefaction, tsunami, flooding and related hazards (lateral spreading, lurching, and ground cracking), slope/bank instability, settlement, and erosion/scour. Other hazards, such as fault surface rupture, expansive soil, and others are judged less than significant and are not discussed in detail. Our evaluations and conceptual mitigation measures for geologic hazards judged significant at the site are discussed in greater detail in the following sections.

Seismic Ground Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 4, could cause moderate to strong ground shaking at the site. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical relations developed from data collected during previous earthquakes to provide estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the site, their maximum credible magnitude, closest distance to the project area, and probable peak accelerations is provided in Table A.

TABLE A
ESTIMATED SEISMIC GROUND MOTIONS
Foundation Upgrade
12887 Sir Francis Drake
Inverness, California

Fault	Moment Magnitude for Characteristic Earthquakes[1]	Closest Estimated Distance[1]	Median Peak Ground Acceleration[1,2]
San Andreas	8.0	0.2 km	0.42 g
San Gregorio	7.4	27 km	0.17 g
Rodgers Creek	7.3	32 km	0.14 g
Hayward	7.3	42 km	0.12 g
Maacama	7.4	50 km	0.11 g

1) Caltrans ARS Online, Version 2.3.06 (web-based acceleration spectra calculator tool), accessed October 4, 2017.
2) Values determined using Vs_{30} = 180 m/s for soft soil conditions (Site Class “E”) in accordance with 2016 California Building Code.

The potential for strong seismic shaking at the project site is high. The San Andreas Fault is the closest and most likely source for a future earthquake. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*
Mitigation: *New foundations should be designed in accordance with the latest edition of the California Building Code (2016 CBC), and retaining structures should be designed with a seismic surcharge load. Seismic design criteria for new foundations and retaining walls are presented in the Conclusions and Recommendations section of this report.*

Liquefaction Potential and Related Impacts

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high fines content (clayey and silty materials that pass the #200 sieve) provided the fines exhibit a relatively low plasticity (plasticity index less than 12), a liquid limit less than 37 and a moisture content greater than 85% of the liquid limit. Regional mapping indicates the project site lies in a zone of “high” liquefaction susceptibility as shown on Figure 5.

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation. The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum credible earthquake peak ground acceleration (PGA) and depth. The soil resistance to liquefaction is based on the relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with Standard Penetration Test (SPT) blow count data measured in the field and corrected for hammer efficiency, overburden, and fines content to determine the $(N1)_{60CS}$ value. Liquefaction analyses (Seed et. al., 2003) which consider a magnitude 8.0 earthquake producing a peak ground acceleration of 0.42 g (based on deterministic seismic analysis discussed above) indicate that the sand layer between 1.0 and 15.0 feet below the ground surface in Boring 2 are liquefiable during the maximum credible earthquake, as shown on Figure 6.

Liquefaction of deeper soils may be manifested in the form of settlement and/or damage to improvements at the ground surface. Ishihara (1985) and Youd (1995) have published empirical relationships to correlate the thickness of overlying non-liquefiable soil layers, the thickness of liquefiable layer, and the potential for ground-surface deformations during liquefaction. The relationships developed by Ishihara and Youd are based on empirical data gathered around the world at sites where liquefaction has occurred in historic times. Our analysis indicates the potential for damaging settlements to occur at the ground surface is moderate to high. Based on our calculations, total post-liquefaction settlements of roughly 6 inches may be expected. The magnitude of post-liquefaction settlements is expected to vary across the site as a result of variations in soil composition and the lateral extents of liquefiable horizons.

Lateral spreading, lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where dense/stiff soils are underlain by soft deposits or along steep slopes or channel banks. Lateral spreading occurs where liquefiable soils move freely towards a free face, such as a creekbank. Based on our exploration, conditions conductive to lurching, lateral spreading, and ground cracking exist throughout the site. Lateral movement is estimated on the order 6 to 18 inches towards Tomales Bay during a strong earthquake.

Based on the results of our subsurface exploration, laboratory testing, and engineering analyses, it is our professional opinion that the likelihood of liquefaction occurring at the project site following a strong seismic event is high. In addition, if liquefaction were to occur, the potential for damaging settlements at the ground surface is moderate to high. When combining these factors, the overall probability of significant liquefaction damage at the property is high.

Evaluation: *Less than significant with mitigation*
Mitigation: *Minimum mitigation measures should include designing the foundations to account for the potential for some settlement due to liquefaction of localized sand layers. We recommend that deep foundations bearing on firm materials beneath any liquefiable horizons be utilized. Alternatively, rigid shallow foundations, designed to span areas of non-uniform support, could be considered if post-liquefaction differential settlement is acceptable. Project designers should also consider the potential effects of lateral spreading, lurching, and ground cracking during a seismic event and provide mitigation if warranted. Recommendations for foundation design are presented in the Conclusions and Recommendations section of this report.*

Settlement

Significant settlement can occur when new loads such as buildings or fill are placed at sites that are located over soft compressible soils such as bay mud. Settlement can also occur or continue if existing fill has not been in-place long enough to allow all of the settlement to occur. The length of time that settlement occurs is influenced by the thickness of the bay mud layer and the distance to permeable drainage layers.

The site is underlain by loose granular soils and compressible silt. Therefore, the risk of damage due to settlement is judged moderate.

Evaluation: *No significant Impact.*
Mitigation: *Settlement is inevitable in areas underlain by compressible soils and will be increased if new fill or building loads are applied to the existing ground surface. Alternatives to reduce new settlement/subsidence could include minimizing the amount of new fill loads, supporting the structure on deep foundations, or designing a shallow foundation system capable of withstanding some total and differential settlement. Flexible utility connections should be provided to reduce the risk of damage and some periodic maintenance to repair offset flatwork and other improvements where they abut the buildings should be anticipated. Additional discussion regarding anticipated settlements, mitigation measures, and optional foundation systems is presented in the Conclusions and Recommendations section of this report.*

Erosion and Scour
Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. The work area is relatively level. Therefore, excess erosion is not considered to be a significant long-term geologic hazard. However, care should be taken during construction to prevent excess erosion when the subsurface soils will likely be exposed.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation measures include designing a site drainage system to collect surface water and discharging it into an established storm drainage system. The project Civil Engineer or Architect is responsible for designing the site drainage system and, an erosion control plan could be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook (2003).*

Flooding
The adverse impact from flooding is water damage to structures. The project site is located within a FEMA 100-year flood zone as shown on Figure 7. Therefore, the risk of damage to improvements due to large scale flooding is moderate.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation measures should include designing the finished floor elevations above flood elevation minimums. Estimates of expected future settlements should also be considered in evaluating flood potential. Additional discussion and geotechnical recommendations for site drainage are presented in the Conclusions and Recommendations section of this report.*

Conclusions and Recommendations

Based on the results of our geotechnical investigation, we conclude that the proposed project is feasible from a geotechnical perspective. The primary geotechnical considerations for the project are providing uniform foundation support for the existing structure and appropriate mitigation for potential liquefaction, settlement, lateral spreading and erosion.

The Owner will need to consider the cost vs. benefit of designing the planned foundation improvements for static or seismic conditions. Under static conditions, shallower and less expensive foundations could be used to lift and support the existing structures. However, under seismic conditions significant vertical and laterals settlement could occur. Much deeper and stronger foundations would be required to significantly reduce the predicted seismic deformations. The cost and desire of upgrading existing structures to current seismic code may influence the foundation design.

Recommendations and design criteria to address these and other geotechnical items are presented in the following sections.

Site Preparation and Grading
Site grading will consist primarily of excavation to facilitate construction of the foundation upgrades. Site preparation, excavation, and backfill should be performed in accordance with the following recommendations and criteria.

1. Excavations - Based on the results of our subsurface exploration, excavations for the proposed construction will generally extend through loose sandy soils and soft clayey soils. We anticipate that the majority of the excavations can likely be accomplished with “conventional” equipment, such as excavators or backhoes. Excavations are anticipated to yield clayey to sandy mixtures that should be suitable for re-use as fill provided they can be processed to meet the gradation requirements discussed below. Excavations that extend below the groundwater table are expected to be unstable. Temporary shoring will likely be required for the planned foundations.

   Excavations having a depth of 5 feet or more must be sloped and/or benched in accordance with OSHA regulations. Pursuant to OSHA classifications, the onsite alluvial soils would be classified as Type “C”. For Type “C” soils, excavations must be sloped no steeper than 1½:1 (horizontal:vertical). If vertical slopes are required, they must be shored or braced to a minimum of 18-inches above the top of the vertical slope. The Contractor should be responsible for site safety and should select and maintain and appropriate shoring system for the site conditions and in accordance with OSHA requirements.

   Performance of temporary cut slopes will be heavily dependent on the amount of time the cut is unsupported, seepage and surface runoff over the face, soil materials, and other factors.

3. Fill Compaction – Any fill placed should be conditioned to a moisture content within 3 percent of the optimum moisture content. Properly moisture-conditioned soils should be placed in loose horizontal lifts of 8 inches thick or less and uniformly compacted to at least 90 percent relative compaction. In pavement areas, the upper 12-inches should be compacted to a minimum of 95 percent relative compaction.

   The fill material should consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40, (3) have a Plasticity Index less than 20, (4) have a maximum particle size of 4 inches, and (5) have a minimum R-Value of 20.

4. Fill Slope Construction – Any permanent cut and fill slopes (if planned) should ideally be inclined no steeper than 3:1. If steep slopes are required, they should incorporate synthetic geogrid reinforcement to improve stability.

   New fill slopes should be founded on keyways and benches excavated into stable soil along the sides of the new culvert. Keyways should extend a minimum of 2-feet laterally beyond the outside edge of the new culvert (at footing elevation). Keyway depths will need to be determined during construction, but we anticipate keyways will extend a minimum of 3-feet into firm soil.

Seismic Design

Minimum mitigation of seismic ground shaking includes design of new structures in conformance to the provisions of the most recent edition (2016) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity of the San Andreas Fault, we recommend the CBC coefficients and site values shown in Table B below to calculate the design base shear of the new construction. To determine site seismic coefficients, we used the USGS Earthquake Ground Motion Parameters Java application, Version 5.1.0, using the latitude and longitude shown on Figure 4.

TABLE B
2016 CBC SEISMIC DESIGN FACTORS
Foundation Upgrade
12887 Sir Francis Drake
Inverness, California

Factor Name	Coefficient	CBC Table/ Figure	Site Specific Value[1]
Site Class[2]	$S_{A,B,C,D,E,\text{ or }F}$	1613.5.2	S_E
Spectral Acc. (short)	S_s	1613.5(3)	2.556 g
Spectral Acc. (1-sec)	S_1	1613.5(4)	1.228 g
Site Coefficient	F_a	1613.5.3(1)	0.9
Site Coefficient	F_v	1613.5.3(2)	2.4

1) Values determined in accordance with the 2010 ASCE-7 standard.
2) Soil Profile Type S_E Description: Soft Clay Soil, Shear Wave Velocity less than 600 feet per second, Standard Penetration blow counts less than 15, and undrained shear strength less than 1,000 psf.

The effects of earthquake shaking (i.e., protection of life safety) can be mitigated by close adherence to the seismic provisions of the current edition of the CBC. However, some structural damage may still occur during strong ground shaking.

Foundation Design

Deep foundations are recommended to provide “superior” performance and should result in minor total and differential settlements.

If decided by owner, we judge that a rigid, shallower foundation system could be considered provided they are designed to span over localized areas of differing support conditions and some post-construction and future post-seismic settlements are deemed acceptable. Various deep and shallow foundation options are discussed below.

Deep Foundations

Helical Piles or Torque-Down Piles - Helical piles could be used to support the foundation. Helical piles are hollow, steel piles (typically 3” to 6” diameter) with helical fins at the base and are “screwed” into place gaining their vertical (compressive and tensile) capacities primarily from the helices. The piles should be installed and corrosion-protected in accordance with the manufacturer's specifications. The piles should derive support from competent native soils. Helical piles can be installed with small construction equipment and battered to provide resistance to lateral loads.

Helical piers extending into approved competent soils and extending below a depth of 40 feet may be designed using an estimated allowable bearing capacity of 30 kips for dead plus live loads. The actual depth and bearing capacity of the anchors should be evaluated based on measured torque values obtained during installation. Helical piers should be interconnected with a foundation footing / grade beams to support structural loads.

Torque-down piles (TDP) are a displacement pile system consisting of steel pipe with helical tips and cutting teeth to assist in pile installation. The piles are installed with a specialized drill rig using a combination of torque and downward pressure. This system also allows additional sections of pile to be welded on in the field as necessary to reach suitable bearing strata. Once the torque down pile reaches the design depth, the steel shell is filled with reinforced concrete. Based on our exploration and laboratory testing, TDP could be expected to develop capacities on the order of 70-kips at depths of about 50-feet.

Torque-down piles and helical piles minimize site disturbance and the quantity of spoils brought to the surface. The torque-down piles would need to be structurally designed to resist the vertical and lateral loads imposed by the existing structure. Elements which are not supported on the piles may experience minor future settlement, especially during a strong seismic event. Therefore, some minor differential settlements should be anticipated between the structure and driveway.

Drilled Piers - Drilled, cast-in-place, reinforced concrete piers should be at least 18 inches in diameter, and should extend at least 30 feet deep. Design pier depths and diameters should be calculated by the Project Structural Engineer using the criteria presented below. The soils encountered in pier excavations should be evaluated by our representative in the field during drilling.

The portion of the piers extending at least 24 inches below subgrade can impose a passive equivalent fluid pressure of 300 pounds per cubic foot (pcf) acting over 2 pier diameters. Vertical dead plus real live loads should be supported by piers designed using 1,000 pounds per square foot (psf) in skin friction from the ground surface to a depth of 10 feet, 300 psf between 10 and 30 feet, and 600 psf below 30 feet. End bearing should not be utilized. These values may be increased by 1/3 for seismic and wind loads, but should not be increased for determining uplift resistance. For seismic conditions, skin friction should be neglected in the soils located above a depth of 22 feet, and end bearing should be neglected due to the uncertainty of mobilizing end bearing and skin friction simultaneously.

Groundwater will be encountered, and it will therefore be necessary to dewater the holes and/or place concrete by the tremie method. Caving soils will likely be encountered, and it will likely be necessary to case the holes. Casing should be carefully maintained ahead of the drill to avoid causing settlement of adjacent areas. Casing should be removed from the holes simultaneous with concrete placement.

Shallow Foundations

Rigid shallow foundations may be considered for the existing structure provided some post-construction seismic settlement on the order of 6 inches due to liquefaction of loose sand layers. Shallow foundations should be designed in accordance with the criteria shown in Table C:

TABLE C
SHALLOW FOUNDATION DESIGN CRITERIA
Foundation Upgrade
12887 Sir Francis Drake
Inverness, California

Spread Footings:	
Minimum embedment (1):	3 feet
Minimum width(2):	2 feet
Allowable bearing pressure:	
Dead plus live loads(3):	500 psf
Base friction coefficient:	0.30
Lateral passive resistance(3,4):	300 pcf
Maximum unsupported interior span(5):	10 feet
Maximum unsupported edge (corner) cantilever(5):	5 feet

Notes:

(1) Maintain a minimum of 7-feet horizontal distance from base of foundation to face of nearest slope. Depth to be below potential scour zone to be determined by others.
(2) Design shallow foundations to similar bearing pressures, i.e. size footing widths to maintain uniform bearing loads.
(3) For compacted fill or soft to medium stiff native soils, may increase design values by 1/3 for total design loads including seismic.
(4) Neglect potential scour zone. Equivalent Fluid Pressure, not to exceed 3,000 psf.
(5) Assumes rigid slab behavior with idealized fixed end conditions.

If soft, wet, expansive, or otherwise unsuitable soils are encountered in shallow foundation excavations, these soils should be removed, stabilization fabric placed, and replaced with properly-conditioned and compacted select fill as described in Site Grading.

Site Foundation and Drainage

Most of the site is elevated above Tomales Bay. In the front yard, new grading could result in adverse drainage patterns and water ponding around buildings. Careful consideration should therefore be given to design of finished grades at the site, and designers of site grading and drainage systems should account for predicted future site settlements. We recommend that landscaped areas adjoining the structures be sloped downward at least 0.25 feet for 5 feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent). Roof gutter downspouts may discharge onto the pavements, but should not discharge onto any landscaped areas. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout discharges into a tight pipe collection system. Site drainage improvements should be connected to the existing municipal storm drain system.

Trench Backfill
Following construction and backfill, utilities may need to be restored. We recommend minimum of 6 inches of non-corrosive sand (or other approved pipe bedding material) be placed in the bottom of trench excavations. The bedding material should be continuous around the utility pipe and extend at least 6 inches above the top of pipe. The bedding material over the pipe should be compacted prior to placement of intermediate backfill.

Intermediate trench backfill above the bedding material and up to the subgrade elevation may be select fill material or aggregate base, unless otherwise specified. Native soil and rock materials derived from excavations at the site are likely suitable for re-use as select fill, provided they are properly processed to conform to the fill criteria discussed in the Site Preparation and Grading/Fill Compaction section above.

Intermediate backfill should be moisture-conditioned to near the optimum-moisture content and compacted to at least 90 percent relative compaction. Within pavement or other structural areas, the uppermost 12-inches should be compacted to at least 95 percent relative compaction, in general accordance with ASTM D-1557. The compacted surface must also be non-yielding when proof-rolled with heavy construction equipment. Refer to the applicable utility district Standard Specifications for additional utility trench backfill requirements.

Pavement Sections
Subgrade preparation for asphalt-paved areas should be performed in accordance with the recommendations shown in the Site Preparation section above. The base rock should consist of a minimum of 6-inches of compacted Class 2 Aggregate Base (Caltrans, 2015), be conditioned to near optimum moisture content, placed in lifts no more than six inches thick, and compacted to achieve at least 95 percent relative compaction and a non-yielding surface when proof-rolled with heavy construction equipment. The subgrade should also be maintained at near-optimum moisture content prior to placement of aggregate base rock. Areas of soft or saturated soils encountered during construction should be excavated and replaced with properly moisture conditioned fill or aggregate base. If asphalt concrete is part of the pavement section it should be a minimum of 2-inches thick.

Supplemental Services

We should review project plans as they near completion to ensure that the intent of our recommendations has been sufficiently incorporated. Additionally, we should be present during construction to verify that actual conditions encountered are consistent with our recommendations and design criteria.

We trust that this letter includes the information you require at this time. Please do not hesitate to contact us should there be any questions or concerns.

Yours very truly,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY

Monica Thornton
Staff Engineer

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/19)

Attachments: Figures 1 through 7; Appendix A

SITE: LATITUDE, 38.1034°
LONGITUDE, -122.8578°

SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2017

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FILENAME: 2174.008 Figures.dwg

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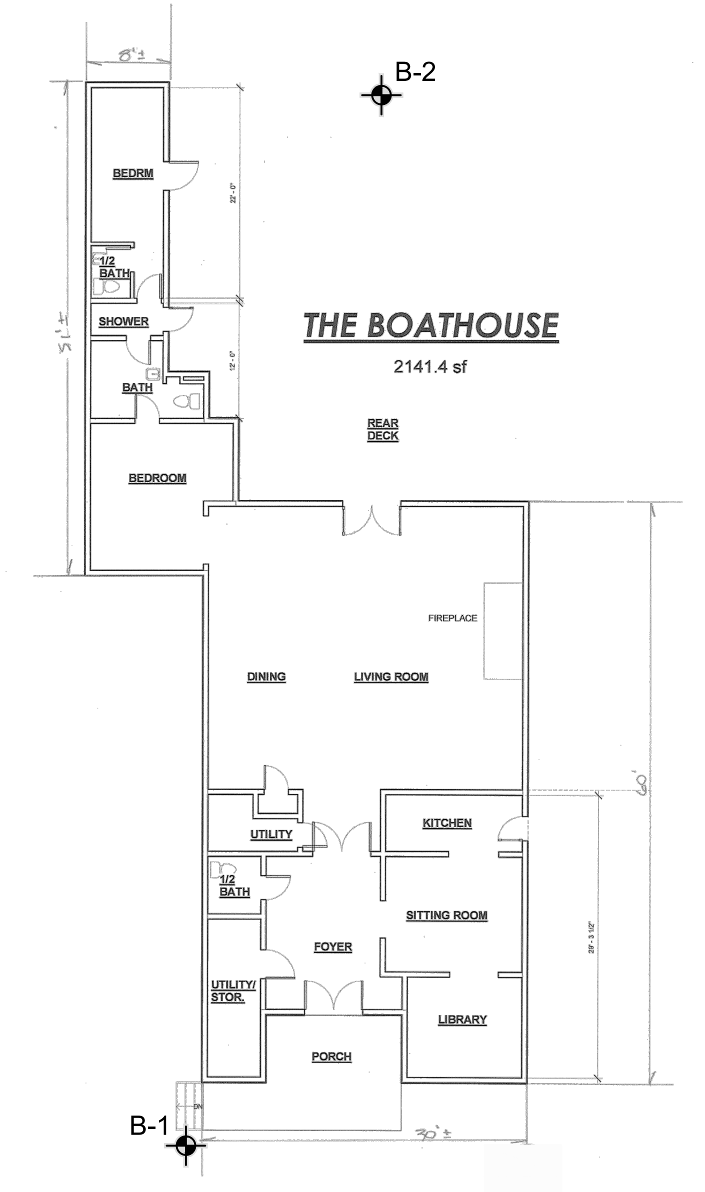
SITE LOCATION MAP

Mid-Cal Construction
12887 Sir Francis Drake Blvd
Inverness, California

Project No. 2174.008 Date: 9/7/2017

Drawn MMT
Checked

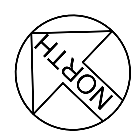
1
FIGURE



SITE PLAN

SCALE

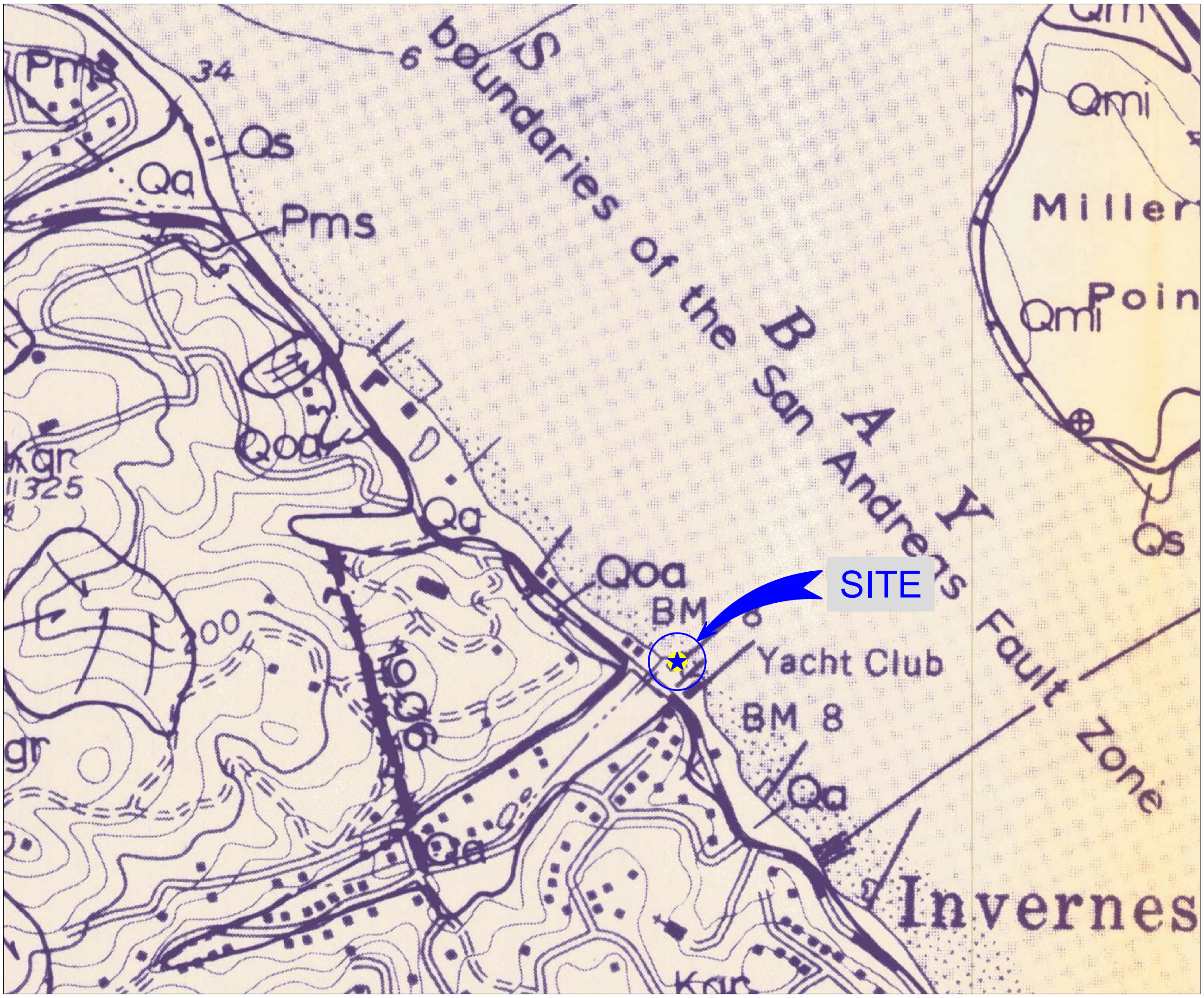
0 10 20 40 FEET



Approximate location of boring completed by MPEG, 2017

REFERENCE: Plans provided by Client.

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE PLAN		
A CALIFORNIA CORPORATION, © 2017, ALL RIGHTS RESERVED FILENAME: 2174.008 Figures.dwg		Mid-Cal Construction 12887 Sir Francis Drake Blvd Inverness, California Project No. 2174.008 Date: 9/7/2017	Drawn MMT Checked	2 FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

Qm — BAY MUD
Marshlands, former marshlands, and mudflats bordering Tomales Bay. These are deposits of unconsolidated, low-density, highly compressible, highly impermeable silt clay that occur at or below mean sea level.

Qa — ALLUVIUM
Unconsolidated deposits of silt, sand, and gravel transported and deposited by streams.

Qoa — OLDER ALLUVIUM
Uplifted and dissected deposits of loosely consolidated sand and pebble to cobble gravel which occur in stream canyons.

DEBRIS FLOW LANDSLIDES
Predominantly deposits of unconsolidated and unsorted soil and rock debris (colluvium) that have moved downslope en masse or in increments by flow or creep processes

BLOCK SLUMP LANDSLIDES
Masses of relatively intact to highly disrupted bedrock that have moved downslope by rotational slip along deep concave slip planes, or rarely, by translational slip along planar surface. Commonly flanked by, and succeeded downslope by, debris flow deposits.

REFERENCE: Thomas E. Gay Jr, T.C., "Geology of the Tomales Bay Study Area", California Department of Conservation, Division of Mines and Geology Open-File Report 77-15 S.F. (Plate 2), Map Scale 1:12,000

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REGIONAL GEOLOGIC MAP

Mid-Cal Construction
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Project No. 2174.008 Date: 9/7/2017

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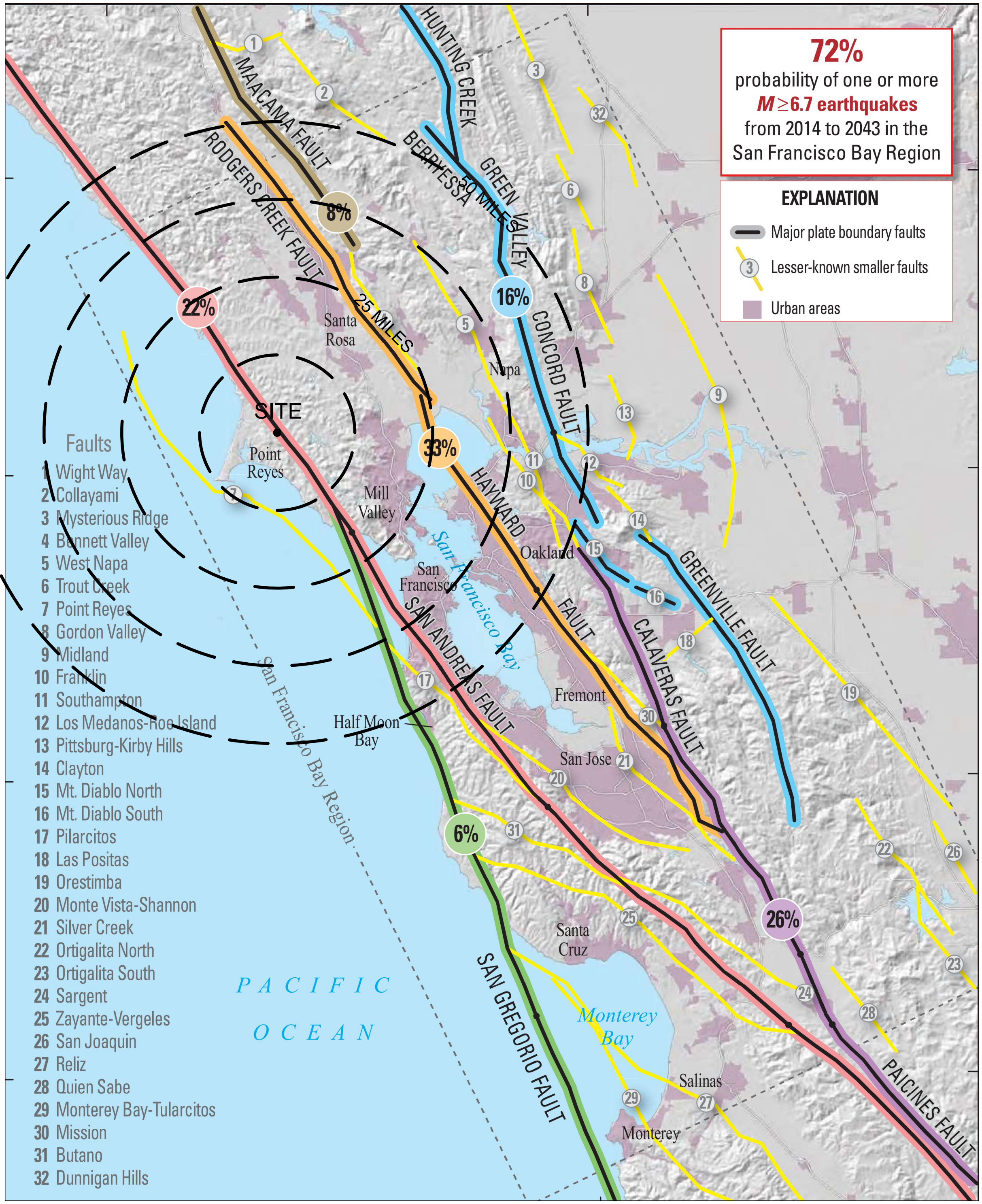
FIGURE 3



SITE COORDINATES
LAT. 38.1034°
LON. -122.8578°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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ACTIVE FAULT MAP

Mid-Cal Construction
12887 Sir Francis Drake Blvd
Inverness, California

Project No. 2174.008 Date: 9/7/2017

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4
FIGURE

Susceptibility Level:

No Scale

Very High
Moderate
Very Low
Local Road
High
Low
Major Road

NORTH

Map Reference: ABAG Geographic Information System.

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LIQUEFACTION SUSCEPTIBILITY MAP

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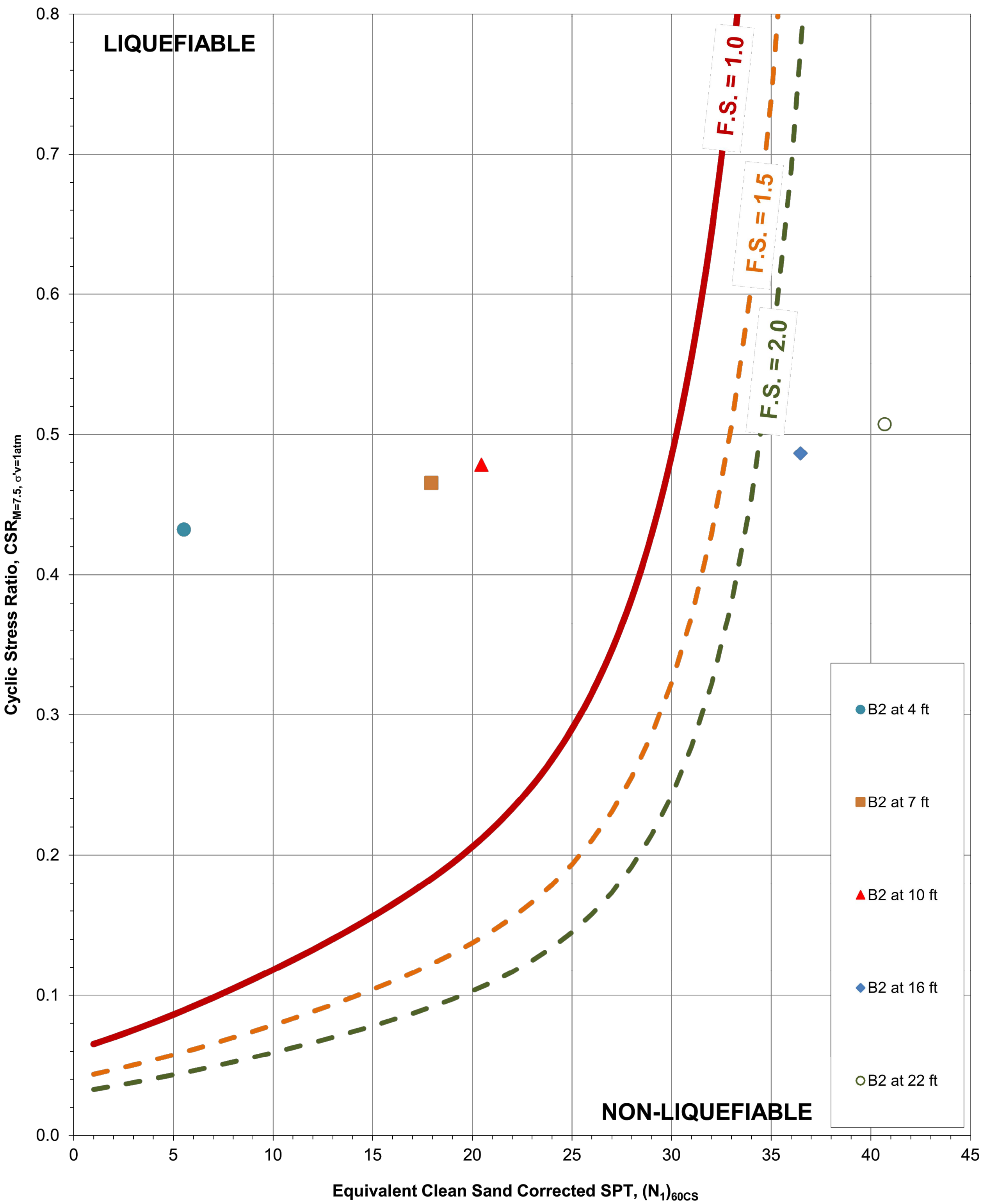
5
FIGURE

Liquefaction Analysis
(Idriss, I.M.& Boulanger, R.W., 2008 & 2010)



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Flood Hazard Area:

	Zone V- 100yr.		Zone X - 500yr.
	Zone A - 100yr.		Urbanized Area

No Scale

Zone V: This code identifies an area inundated by 1% annual chance flooding with velocity hazard (wave action).
Zone A: This code identifies an area inundated by 1% annual chance flooding.

Zone X 500yr: This code identifies an area inundated by .02% annual chance flooding and area inundated by 1% annual chance of flooding with average depth of less than 1 foot of with drainage areas less than 1 square mile or an area protected by levees form 1% annual chance flooding.

NORTH

Map Reference: ABAG Geographic Information System.

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FEMA DIGITAL FLOOD INSURANCE RATE MAP (DFIRM)

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7
FIGURE

APPENDIX A

SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. Soil and Rock Classification Systems

We explored subsurface conditions at the site with 2 exploratory borings drilled on August 30, 2017. Borings were excavated to depths of 31.5-feet using a portable hydraulic drill rig equipped with 6-inch hollow-stem augers. The soils encountered were logged and identified by our field geologist in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figure A-1, Soil Classification Chart and Figure A-2 Rock Classification Chart. Exploratory boring logs are shown on Figures A-3 through A-6.

B. Laboratory Testing

We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;
- Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D 4318; and
- Amount of Material in Soils Finer than No. 200 (75-µm) Sieve, ASTM D 1140.

The moisture content, dry density, unconfined compressive strength and percentage of particles finer than the no. 200 sieve test results are shown on the Boring Logs, Figures A-3 through A-6. Plasticity index test results are shown on Figure A-7.

The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the excavation at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate, and changes in surface and subsurface drainage.

MAJOR DIVISIONS		SYMBOL	DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	Well-graded gravels or 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
		GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
	SAND with fines	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <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, gravely clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, muck, and other highly organic soils
ROCK			Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

MODIFIED CALIFORNIA

HAND SAMPLER

STANDARD PENETRATION TEST

ROCK CORE

THIN-WALLED / FIXED PISTON

X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

TV	FIELD TORVANE (UNDRAINED SHEAR)
UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

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SOIL CLASSIFICATION CHART

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A-1
FIGURE

FRACTURING AND BEDDING

Fracture Classification	Spacing	Bedding Classification
Crushed	less than 3/4 inch	Laminated
Intensely fractured	3/4 to 2-1/2 inches	Very thinly bedded
Closely fractured	2-1/2 to 8 inches	Thinly bedded
Moderately fractured	8 to 24 inches	Medium bedded
Widely fractured	2 to 6 feet	Thickly bedded
Very widely fractured	greater than 6 feet	Very thickly bedded

HARDNESS

Low	Carved or gouged with a knife
Moderate	Easily scratched with a knife, friable
Hard	Difficult to scratch, knife scratch leaves dust trace
Very hard	Rock scratches metal

STRENGTH

Friable	Crumbles by rubbing with fingers
Weak	Crumbles under light hammer blows
Moderate	Indentations <1/8 inch with moderate blow with pick end of rock hammer
Strong	Withstands few heavy hammer blows, yields large fragments
Very strong	Withstands many heavy hammer blows, yields dust, small fragments

WEATHERING

Complete	Minerals decomposed to soil, but fabric and structure preserved
High	Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates
Moderate	Fracture surfaces coated with weathering minerals, moderate or localized discoloration
Slight	A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation
Fresh	Rock unaffected by weathering, no change with depth, rings under hammer impact

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

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ROCK CLASSIFICATION CHART

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A-2
FIGURE

BORING 1

EQUIPMENT: Portable Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 8/30/17
ELEVATION: 9 - feet*
*REFERENCE: Google Earth, 2017

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			SAND with Clay (SP-SC) Medium yellow brown mottled black and white, wet, very loose, fine to medium grained sand, ~5-10% low plasticity clay. [Alluvium]						
1										
	5									
2				Sandy SILT (MH) Dark gray black, saturated, medium stiff, high plasticity silt, ~10-15% fine grained sand. [Alluvium]						
3	10									
4										
	15									
5										
6	20	X X								LL:92 PI:23

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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A-3
FIGURE

BORING 1 (CONTINUED)

DEPTH meters / feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
20 – 7			**Sandy SILT (MH)** Dark gray black, saturated, medium stiff, high plasticity silt, ~10-15% fine grained sand. [Alluvium]						
25 – 8 – 9 30 – 10 – 35 – 11 – 12 40			Boring terminated at 24.0 feet. Groundwater measured at 3.0 feet upon completion of exploration.						

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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A-4
FIGURE

BORING 2

EQUIPMENT: Portable Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 8/30/17
ELEVATION: 6 - feet*
*REFERENCE: Google Earth, 2017

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			SAND with Clay (SP-SC) Medium brown mottled black and white, wet, very loose, fine to medium grained sand, ~5-10% low plasticity clay. [Alluvium]	2		24.0			
1				Grades dark brown and black, saturated.	3		16.3	P200 7.2%		
	5									
2				Grades medium gray and brown, loose to medium dense.	10		14.4	P200 6.7%		
3	10			Grades medium dense, ~10-15% low plasticity clay.	11		15.2	P200 11.9%		
4										
	15									
5					25		13.0	P200 11.3%		
6	20									

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-5
FIGURE

BORING 2 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			SAND with Clay (SP-SC) Medium gray and brown, saturated, medium dense, fine to medium grained sand, ~5-10% low plasticity clay. [Alluvium]	29		11.4		P200 6.4%	
7										
	25									
8				SANDSTONE Yellow brown and gray, intensely fractured, low hardness, weak, highly weathered. [Bedrock]						
9	30				69		17.0			
				Boring terminated at 31.5 feet. Groundwater measured at 1.0 feet upon completion of exploration.						
10										
	35									
11										
12	40									

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-6
FIGURE

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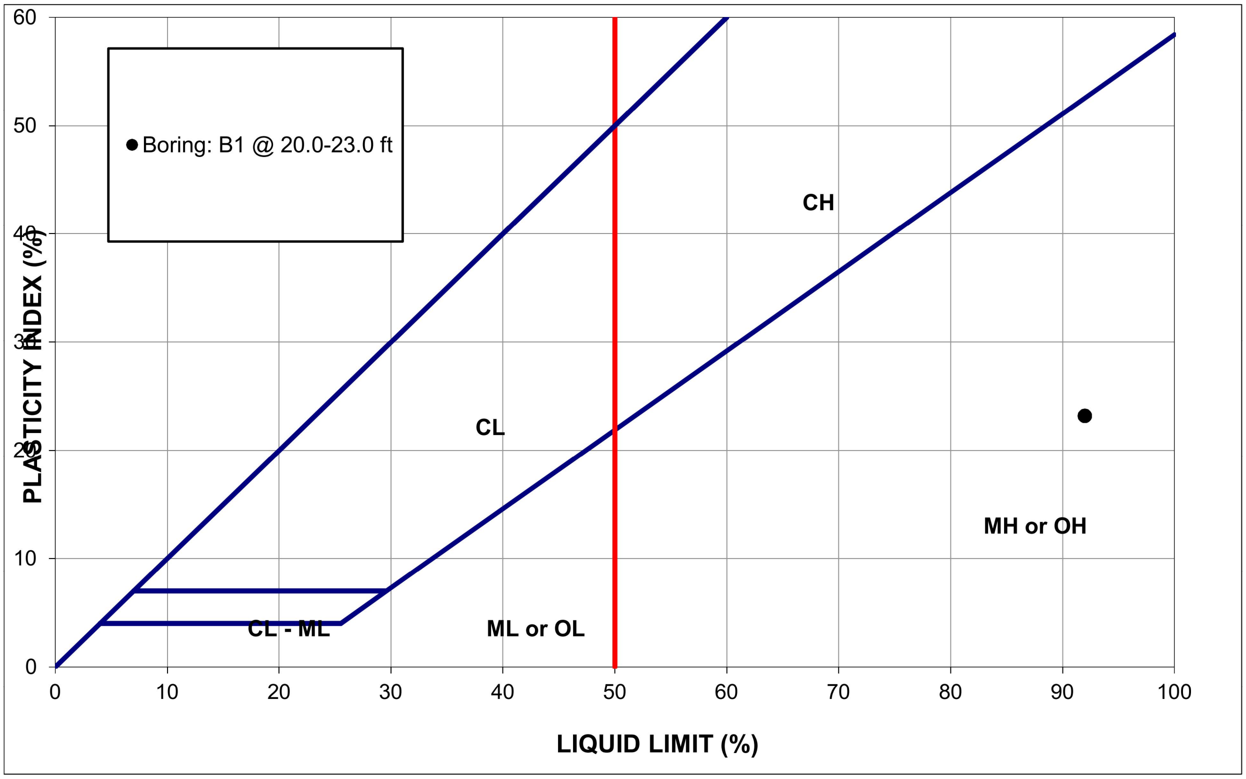
ATTERBERG LIMITS TEST (ASTM D 4318)



Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Boring: B1 @ 20.0-23.0 ft	Sandy SILT (MH) dark gray black	92	69	23

PI = 0-3: Non-Plastic
PI = 3-15: Slightly Plastic
PI = 15-30: Medium Plasticity
PI = >30: High Plasticity

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PLASTICITY INDEX RESULTS

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A-7
FIGURE