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Re: Geotechnical Investigation
Residential Renovation and Addition
65 Dipsea Road
Stinson Beach, California

Introduction

This letter summarizes the results of our geotechnical investigation for the new residential renovation and addition at 65 Dipsea Road in Stinson Beach, California. A Site Location Map is presented on Figure 1. Our services have been provided in general accordance with our Agreement for Professional Engineering and Testing Services, authorized June 27, 2024.

The purpose of our investigation is to evaluate site geologic conditions and develop geotechnical recommendations for use in project design and construction. The scope of our investigation is described in our proposal letter dated June 19, 2024, and includes review of pertinent geologic mapping and geotechnical background data from nearby sites, subsurface exploration with one exploratory soil boring, laboratory testing of recovered samples, development of geotechnical recommendations and design criteria for the proposed improvements, and preparation of this letter report.

Project Description

Based on review of preliminary plans, we understand the project generally includes removal of a portion of the existing, attached garage, converting it to living space and construction of a new residential addition on the northwest corner of the house. The new addition will create a new accessory dwelling unit. We understand the new addition will utilize shallow spread footings and may use raised wood floors. A new front entry deck will be located between the garage and new addition. A Site Plan showing the extents of the existing improvements is presented on Figure 2.

Existing Site Conditions

The site consists of a developed parcel bounded to the south by Seadrift Lagoon that is separated from Bolinas Lagoon by Dipsea Road and Seadrift Road, to the north by Dipsea Road and further by Bolinas Lagoon, and to the east and west by existing single-family residential developments. The site is developed with an existing single-family residence in the central portion of the property with a short driveway at the northwest corner of the parcel that provides access to the attached garage. The residence is located approximately 10-feet north of the isolated waterfront with a small deck and dock along the rear property line. Site elevations range from about +10- to +11-feet. The site slopes downward from the residence and driveway to the northeast of the property which contains protruding pipes (likely septic leach lines) and small shrubs. The existing residence appears to be performing well and does not show signs of significant distress or damage.

Regional Geology

The project site is located in the Coast Ranges geomorphic province of California, which is typified by generally northwest-trending ridges and intervening valleys formed as a result of movement along a group of northwest-trending fault systems, including the San Andreas Fault. Bedrock geology within Marin County is dominated by sedimentary, igneous, and metamorphic rocks of the Jurassic-Cretaceous age Franciscan Complex. Sandstone and shale comprise the majority of Franciscan rock types, while less common rocks include chert, serpentinite, basalt, greenstone, and exotic low- to high-grade metamorphic rocks, including phyllite, schist, and eclogite.

Regional geologic mapping (Clark and Brabb, 1997) indicates the site is underlain by Quaternary (geologically young) beach sand, which is typically composed of loose, moderately- to well-sorted, fine to coarse-grained sand. Mapping also indicates the main trace of the San Andreas Fault is located approximately 4,000-feet west of the site. A regional geologic map is shown on Figure 3.

Seismicity

The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated, or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust, but typically comprised of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as bay mud.

1. Active Faults in the Region - The project site is located within a seismically active region that includes the Central and Northern Coast Mountain Ranges. Several active faults are present in the area including the San Andreas, San Gregorio, Hayward, and Rodgers Creek, among others. An “active” fault is defined as one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no evidence of recent rupture. The California Department of Conservation, Division of Mines and Geology has mapped various active and inactive faults in the region (CDMG, 1972 and 2000). These faults are shown in relation to the project site on the attached Active Fault Map, Figure 4.

2. Historic Fault Activity - Numerous earthquakes have occurred in the region within historic times. The approximate locations of earthquakes which occurred between 1830 and 2021 are shown on the Historic Earthquake Map, Figure 5.

3. Probability of Future Earthquakes – The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San

Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (2003, 2008, 2013, 2016) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (aka UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 indicate the highest probability of an M>6.7 earthquake on any of the active faults in the San Francisco Bay region by 2045 is assigned to the Hayward/Rodgers Creek Fault System, located approximately 27.2-kilometers northeast of the site, at 33%. The San Andreas Fault, located about 1.2-kilometers southwest of the site, is assigned a probability of 22%. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

Subsurface Exploration, Laboratory Testing, and Subsurface Conditions

Subsurface exploration for the project included excavation of one soil boring using manual hand auger equipment with 3.25-inch auger bucket on July 10th, 2024. The approximate location is shown on Figure 2 and extended to a maximum depth of about 5-feet below the ground surface. Materials encountered were examined and logged samples collected at select intervals for laboratory testing. Brief descriptions of the terms and methodology used in classifying earth materials are provided on the Soil Classification Chart, Figure A-1. Exploratory boring log is shown on Figure A-2.

Laboratory testing of select samples included determination of moisture content, dry density, and fines content (percentage of particles passing the No. 200 sieve), in general accordance with applicable ASTM standards. Laboratory test results are shown on the boring log, Figure A-2. Our subsurface exploration and laboratory testing is discussed in greater detail in Appendix A.

Our boring, excavated along the west side of the existing residence, confirms the regional geologic mapping and included loose to medium dense, fine to medium grained sands to the full depth explored. Fines content slightly increased with depth. While the hole remained open during exploration, relatively clean sands are prone to caving in open excavations.

Groundwater

Previous investigations in the area encountered groundwater at depths generally between about 3- and 10-feet of the ground surface. Groundwater was not encountered in our boring. Based on experience with previous projects in the area, groundwater should generally be anticipated to coincide with water levels in the adjacent lagoon and Bolinas Bay (Pacific Ocean) and could be shallower during the winter months and following periods of heavy rainfall.

Review of Previous Investigations

We have reviewed the results of previous investigations performed by Miller Pacific for nearby projects in the Seadrift development. Exploratory CPT and boring logs from previous reports are attached for reference in Appendix B, and each report we reviewed is briefly summarized below:

- Miller Pacific Engineering Group (2014), "Geologic and Geotechnical Evaluation, 177 Dipsea Road, Stinson Beach, California", Project No. 1904.001, dated January 13, 2014.

  This project was located about 0.6-miles northwest of the site. Subsurface exploration included 2 soil borings drilled to maximum depths between about 11- and 40-feet below the ground surface. Each boring encountered loose, poorly-graded dune sand to the maximum depths explored, and groundwater was measured at elevations of about +4 and +7-feet immediately following exploration.

- Miller Pacific Engineering Group (2016), "Geotechnical Investigation, 263/265 Seadrift Road (APN 195-340-28 and 195-340-29), Stinson Beach, California", Project No. 2277.001, dated April 12, 2016.

  This project was located about 0.6-miles west of the site, on the opposite (south) side of the Seadrift Lagoon. Exploration included performance of 2 Cone Penetration tests and 2 shallow borings. CPTs indicate dune sands (along with lesser, interbedded silty to gravelly alluvial soils) extend to depths of at least 70-feet, and locally approaching 100-feet. Liquefaction analysis indicated the underlying sands are liquefiable and that post-seismic (liquefaction-induced) settlements of up to 8-inches and lateral displacement may be expected following a significant earthquake.

Geologic Hazards

The principal geologic hazards which could potentially affect the project site are strong seismic shaking, liquefaction, seismically-induced settlement, tsunami inundation, and corrosive soils. Other geologic hazards, such as fault rupture, expansive soils, and others, are not considered significant at the site. More detailed discussion of each geologic hazard considered, their anticipated impacts, and recommended mitigation measures are discussed below.

Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning (APEFZ) Act[1], the California Division of Mines and Geology (CDMG, now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required[2]. As shown on Figure 6, the site is mapped approximately 1.2-kilometers northeast of the Alquist-Priolo Special Studies Zone for the San Andreas Fault. While there is no significant field evidence or documented history of surface rupture in the immediate site vicinity beyond the limits of the Alquist-Priolo Zone, there will be some generally un-quantifiable potential for secondary faulting to occur throughout the Seadrift subdivision and the Bolinas Lagoon area.

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[1] California Department of Conservation, Division of Mines and Geology (1972), Special Publication 42, "Alquist-Priolo Special Studies Zone Act," (Revised 1988).

[2] California Department of Conservation, Division of Mines and Geology (2000), "Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region", DMG CD 2000-004.

Evaluation:	*Risk of damage due to fault rupture is considered low.*
Recommendations:	*For typical residential projects, the risk of damage due to fault surface rupture cannot be adequately mitigated beyond implementation of minimum setbacks from active faults. Given the existing development, thickness of soil layer, scope of the proposed improvements, and setback from currently-known and active traces, we judge additional measures are generally infeasible. However, the Owner should acknowledge a low risk of fault surface rupture may exist at the site.*

Seismic Shaking

The site will likely experience seismic ground shaking from future earthquakes in the 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. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods. For residential developments, deterministic methods are typically used.

Deterministic seismic hazard analysis predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations (Abrahamson, Silva & Kamai; Boore, Stewart, Seyhan & Atkinson; Campbell & Borzognia; and Chiou & Youngs, (2014)) were utilized to provide approximate estimates of median peak site accelerations. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake, probable median accelerations and plus one standard deviation (+1σ), peak ground accelerations (PGA) for earthquakes on faults near the site are shown in Table A.

TABLE A
ESTIMATED PEAK GROUND ACCELERATION
FOR PRINCIPAL ACTIVE FAULTS
65 Dipsea Road
Stinson Beach, California

Fault	Moment Magnitude(1)	Estimated Distance (km)(1)	Median Peak Ground Acceleration (g)(1,2)	Median PGA +1 Std Dev (g)
San Andreas	8.0	1.2	0.52	0.86
San Gregorio	7.4	2.1	0.49	0.81
Hayward	7.6	27.2	0.21	0.35

(1) Values estimated using Google Earth KML Files showing Quaternary Faults & Folds in the US obtained from USGS website July 2, 2024.

(2) Values determined using Vs_{30} = 260 m/s (Soil Profile Type S_D) for stiff soil subsurface conditions.

The calculated bedrock accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations. Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements, such as light fixtures, shelves, cornices, etc., to fall, presenting a hazard to building occupants and contents. The first building code regulations relating to earthquake resistance appeared in 1933. As knowledge has increased, codes have been updated; however, the field of earthquake-resistant structural engineering is very complex and relatively new. Compliance with provisions of the California Building Code (CBC) should result in structures that do not collapse in an earthquake. Damage may still occur, and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their close proximity, the San Andreas and San Gregorio Faults present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation:	*Risk of damage due to seismic shaking is considered significant.*
Recommendations:	*New structures and foundations should be designed in accordance with the most recent version of the California Building Code (2022). Recommended seismic design coefficients are presented in the Conclusions and Recommendations section of this report.*

Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. This phenomenon can occur where there are saturated, loose, granular (sandy) deposits subjected to seismic shaking. Liquefaction-related phenomena include settlement, flow failure, and lateral spreading. Lateral spreading typically occurs where stiff soils are underlain by soft, liquefiable deposits which flow towards an adjacent free face, such as a slope or channel bank. Regional mapping (ABAG, 2024) indicates that the site lies in a zone of “very high” liquefaction susceptibility as shown on Figure 7.

The results of our previous work in the area indicate liquefiable soils underlying the site are likely on the order of 70- to 100-feet thick, and significant post-seismic settlements of up to 8-inches may be expected.

Evaluation:	*Risk of damage due to liquefaction is considered significant.*
Recommendations:	*Reduction in the risk of damage due to liquefaction and related settlements typically requires significant ground-improvement, deep foundation systems, or design of extremely stiff/rigid shallow foundations. Such measures are generally expensive and may not be practical considering the scope of the proposed improvements. Therefore, it may be necessary to accept the higher risk of damage and maintain/repair any damages that may occur as a result of liquefaction.*

Seismically-Induced Ground Settlement

Seismic ground shaking can induce settlement of unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Since loose, granular soils are anticipated at the site, we judge the likelihood of seismically-induced settlement is moderate.

Evaluation:	*Risk of damage due to seismically-induced settlement is considered significant.*
Recommendations:	*Reduction in the risk of damage due to seismically-induced settlement typically requires significant ground-improvement, deep foundation systems, or design of extremely stiff/rigid shallow foundations. Such measures are generally expensive and may not be practical considering the scope of the proposed improvements. Therefore, it may be necessary to accept the higher risk of damage and maintain/repair any damages that may occur as a result of seismic densification.*

Lateral Spreading

Lateral spreading refers to a specific type of liquefaction-induced ground failure characterized primarily by horizontal displacement of surficial soil layers as a consequence of liquefaction of a subsurface granular layer. Lateral spreads generally move down gentle slopes or slip toward a free face such as an incised river or creek channel. Given the proximity to the Pacific Ocean, Bolinas Lagoon, and Seadrift Lagoon, and loose saturated sands to depths of 70-feet or more, we estimate a permanent lateral spread displacement of several feet at the building site. Therefore, we judge the risk of damage due to lateral spreading is high during strong seismic shaking.

Evaluation:	*Risk of damage due to lateral spreading is considered significant.*
Recommendations:	*Reduction in the risk of damage due to lateral spreading typically requires significant ground-improvement or deep foundation systems. Such measures are generally expensive and may not be practical considering the scope of the proposed improvements. Therefore, it may be necessary to accept the higher risk of damage and maintain/repair any damages that may occur as a result of seismic densification.*

Expansive Soils

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, uneven floors, and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. As noted above, the site is underlain by sandy soils, and we therefore judge the risk of damage due to expansive soils is low.

Evaluation:	*Risk of damage due to expansive soils is considered low.*
Recommendations:	*No special engineering measures are required.*

Erosion

Sandy soils on moderately steep slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity.

Construction of the proposed improvements may require grading and changes to existing surface drainage patterns which, if not properly addressed during design and construction, could lead to concentrated surface water flows and increased erosion. Considering the disturbance to existing vegetation and drainage patterns that may result from the proposed improvements, we judge the risk of damage to improvements due to erosion is moderate.

Evaluation: *Risk of damage due to erosion is considered moderate.*

Recommendations: *For new improvements at the site, careful attention should be paid to finished grades and the project Civil Engineer should design the site drainage system to collect surface water into a storm drain system and discharge water at appropriate locations. Surface water runoff from the site should be collected and conveyed to an appropriate discharge location. Re-establishment of vegetation on disturbed areas will minimize erosion. Erosion control measures during and after construction should be in accordance with a prepared Storm Water Pollution Prevention Plan and should conform to the most recent version of the California Stormwater Quality Association (CASQA) Construction Best Management Practice Handbook or superseding local guidelines.*

Seiche and Tsunami

Seiche and tsunamis are short duration earthquake-generated water waves in large, enclosed bodies of water and the open ocean, respectively. The extent and severity of a tsunami would be dependent upon ground motions and fault offset from active faults. Possible adverse effects could include inundation by floodwaters and debris impact to structures and improvements.

The site is located approximately 1,100-feet from the Pacific Ocean and 0.6-miles from the San Andreas Fault and lies within a mapped tsunami inundation zone as shown on Figure 8. Therefore, we judge the risk of tsunami inundation at the site is moderate to high.

Evaluation: *Risk of damage due to tsunami is considered significant.*

Mitigation: *The Stinson Beach area is well developed with single family residences and small commercial buildings. The majority of these structures are located adjacent to or in close proximity to the Pacific Ocean. It is impractical to design structures to withstand the adverse effects of a severe tsunami. By opting to develop land in close proximity to the open ocean the owner accepts the risk of a possible tsunami during a seismic event on any of several local and distant fault sources. In the event a tsunami occurs, follow the procedures set forth in the current Stinson Beach evacuation plan.*

Flooding
The primary adverse impact from flooding is water damage to structures. As shown on Figure 9, the site is located within a FEMA 100-year flood zone. Additionally, given the low site surface elevations, the proximity of the tidally-influenced Bolinas Bay (Pacific Ocean), and predicted sea level rise (between 1- and 3-feet over 100-years), we judge the risk of large-scale flooding at the site is moderate.

Evaluation:	*Risk of damage due to flooding is considered moderate.*
Mitigation:	*Given the scope of the planned improvements, special engineering measures to reduce the risk of flooding are likely neither warranted nor cost-effective. The potential for inundation with floodwater should be considered during design and specification of new improvements and materials.*

Soil Corrosion
Corrosive soil can damage buried metallic structures and underground utilities, deteriorate rebar reinforcement, and cause spalling of concrete. Soils high in soluble sulfates and chlorides, as well as acidic soils and soils of low electrical resistivity, tend to have high corrosive potential. The site is located adjacent to Bolinas Bay (Pacific Ocean), and as such we anticipate the risk of damage to improvements due to corrosive soils is high.

Evaluation:	*Risk of damage due to corrosive soils is considered high.*
Recommendations:	*Designers of new improvements should consider the potential for corrosion during design and specification of improvements and materials. At a minimum, reinforcing steel should be epoxy coated, provided with sufficient cover, and Type II or V cement should be used. Metallic piping and conduit should generally be avoided.*

Conclusions and Recommendations

Based on our site reconnaissance and review of available reference material, we judge that construction of the proposed improvements is feasible from a geotechnical standpoint. It should be acknowledged that, given the site location/geologic setting and the scope of the planned improvements, providing effective mitigation for the risks of fault surface rupture, liquefaction, seismically-induced settlements, lateral spreading, and tsunami/flood inundation is not practical, and the Owner (and future owners) should acknowledge those risks.

Therefore, we judge that primary geotechnical considerations for the project include providing uniform foundation support and adequate design to resist seismic shaking. Shoring and dewatering of excavations for subsurface improvements will also require careful consideration. Recommendations and design criteria for geotechnical project components are presented in the following sections.

Seismic Design
Minimum mitigation of ground shaking includes seismic design of the structures in conformance with the provisions of the most recent version (2022) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics.

Due to the presence of sandy soil layers beneath the building site that are prone to liquefaction, we judge the site should be classified as “Site Class F” per the 2022 California Building Code. However, per section 20.3.1 of the ASCE 7-16, an equivalent linear site-specific response analysis (i.e., SHAKE, DeepSoil, etc.) is not required if the proposed structure has a fundamental period less than 0.5 seconds. We anticipate the proposed structure will have a fundamental period less than 0.5-seconds; therefore, based on observed sampler blow counts and examination of soil samples, we recommend the project be designed on the basis of “Site Class D”.

Additionally, because the S_1 value is greater than 0.20 g a site-specific ground motion analysis should be performed per the procedures outlined in ASCE 7-16. However, per ASCE 7-16 Section 11.4.8, “a site-specific analysis is not required for structures located on sites classified as “Site Class D” if the seismic response coefficient, C_s, is determined by Equation 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or Equation 12.8-4 for $T > T_L$.” If the project structural engineer finds the building will have a period greater than 0.5-seconds, we should provide a site-specific analysis per ASCE 7-16 Section 11.4.8. The proposed improvements should be designed utilizing the seismic design criteria presented in Table B.

TABLE B
2022 CBC FACTORS
65 Dipsea Road
Stinson Beach, California

Factor Name	Coefficient	2022 CBC Site Specific Value
Site Class[1]	$S_{A,B,C,D,E,F}$	S_D
Site Coefficient	F_a	1.00
Spectral Acc. (short)	S_S	2.39 g
Spectral Acc. (1-sec)	S_1	1.00 g
Spectral Response (short)	SM_S	2.39 g
Design Spectral Response (short)	SD_S	1.59 g
MCE_G[2] PGA adjusted for Site Class	PGA_M	1.12 g

Notes:
1. Site Class D Description: Stiff Soil profile with shear wave velocities between 600 and 1,200 ft/sec, standard blow counts between 15 and 50 blows per foot, and undrained shear strength between 1,000 and 2,000 psf.
2. Maximum Considered Earthquake Geometric Mean

Site Preparation and Grading

Minor grading, consisting of a combination of shallow excavations and placement of thin fills, is anticipated for the project. All site preparation and grading should be performed in accordance with the following recommendations.

Surface Preparation – Clear all foundations, trees, brush, roots, over-sized debris, and organic material from areas to be graded. Trees that will be removed (in structural areas) must also include removal of stumps and roots larger than two inches in diameter. Excavated areas (i.e., excavations for stump removal) should be restored with properly moisture conditioned and compacted fill as described in the following sections. Any loose soil or rock at subgrade will need to be excavated to expose firm natural soils or bedrock. Debris, rocks larger than six inches and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas.

Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of about twelve inches, moisture conditioned to within 2% of the optimum moisture content, and compacted to a minimum of 90% relative compaction (ASTM D-1557). Relative compaction should be increased to a minimum of 95% where new asphalt pavements are planned. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop." If soft, wet, or otherwise unsuitable materials are encountered at the subgrade elevation during construction, we will provide supplemental recommendations/field directives to address the specific condition.

Excavations – Site excavations for planned improvements will generally encounter loose to medium-dense, effectively cohesionless sands. Based on our subsurface exploration, we judge the majority of site excavations can be reasonably performed with “traditional” grading equipment, such as medium-size dozers, excavators, and backhoes. Excavations are likely to yield sandy soils which may be suitable for re-use as fill.

All excavations in excess of 5-feet deep will need to be sloped or braced in accordance with Cal/OSHA regulations. Based on our exploration, onsite soils will be prone to raveling and collapse in open excavations and should be considered “Type C”. The project Contractor should anticipate caving conditions and, where needed, provide a shoring system capable of providing immediate support to the walls of the excavation to reduce the risk of caving and collapse. Groundwater should be anticipated in excavations and dewatering may be required to maintain dry working conditions. However, given the proximity to Pacific Ocean, high infiltration rates could exceed pump capacities, and several pumps may be required. Dewatering wells are unlikely to be effective given the proximity of the ocean.

If excavations extend within the “zone of influence” (ZOI) of any adjacent foundations, defined as the region below a 2:1 line projected down from the base of the nearest adjacent foundation element, the adjacent foundations should be underpinned or provided with temporary shoring to avoid damage due to loss of adjacent support. In the event underpinning is required, we should be consulted to provide supplemental recommendations.

Fill Materials – All native or imported fill material shall consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 15, and (3), have a maximum particle size of 6 inches. On-site soils appear to meet this criteria and are suitable for reuse. Any imported fill material needs to be tested to determine its suitability for use as fill material.

Fill Placement and Compaction – Fill materials should be conditioned to near the optimum moisture content, placed in loose horizontal lifts not exceeding 8-inches in thickness, and be compacted to a minimum of 90% relative compaction. Where asphalt pavements or other vehicle-loaded areas are planned, compaction should be increased to 95% minimum. Compaction may be reduced to 85% minimum in landscape areas where no new structures are planned.

Foundation Design

As noted above, significant post-seismic settlements could affect the site. Therefore, in order to accommodate differential post-seismic settlements, we recommend a rigid shallow foundation system. Suitable shallow foundation systems include a thick, heavily-reinforced mat slab, a “waffle” slab, or a post-tensioned slab. Each of the shallow foundation systems should be designed to span areas of non-uniform support up to 15-feet in diameter and minimize the effects of post-seismic differential settlements. If a shallow foundation system is chosen, total post-seismic settlements of up to about 8-inches and differential settlements on the order of about 4- to 6-inches across the site should be expected. Following an earthquake, the structures could be re-leveled via jet- or compaction-grouting, underpinning with deep foundation elements, or other means. Design criteria for shallow foundation systems are presented below in Table C.

TABLE C
SHALLOW FOUNDATION DESIGN CRITERIA
65 Dipsea Road
Stinson Beach, California

Shallow Continuous Spread Footings	
Minimum width:[1]	18 inches
Minimum depth:[2]	18 inches
Allowable bearing capacity:[3]	
Native Sandy Soils:	2,000 psf
Base friction coefficient:	0.30
Lateral passive resistance:[4]	
Native Sandy Soils:	350 pcf
Rigid Mat or Post-Tensioned Slab:	
Modulus of subgrade reaction, k_s	150 pci
Minimum thickness at edge of slab:[5]	12 inches
Maximum unsupported interior span:[5]	15 feet
Maximum unsupported edge (corner) cantilever:[6]	7 feet
Edge moisture variation (e_m) – Center Lift	15 feet
Edge moisture variation (e_m) – Edge Lift	7 feet
Differential soil movement (y_m) – Center Lift	2.0 inch
Differential soil movement (y_m) – Edge Lift	2.0 inch

Notes:
(1) Size foundations to maintain uniform bearing pressures, i.e., size footing widths to design loads instead of uniform foundation widths.
(2) Footings may need to be deeper if the Structural Engineer determines additional rigidity is required to evenly spread column loads.
(3) Dead plus live loads. May increase by 1/3 for total design loads, including wind and seismic.
(4) Equivalent fluid pressure. Ignore upper 6-inches unless confined by asphalt or concrete.
(5) Actual thickness, load distribution, and unsupported spans must be determined by Structural Engineer to reduce deformations to acceptable levels.
(6) Assumes rigid slab behavior with idealized fixed end conditions.

Site and Foundation Drainage

The site is relatively flat and there is a possibility that 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. We recommend that landscaped areas adjoining new structures be sloped downward at least 0.25 feet for 5 feet (5%) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, these surfaces should be sloped at least 0.10 feet in the first 5 feet (2%). Roof gutter downspouts may discharge onto the pavements but should not discharge onto 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 into and existing storm drain system, if one exists, or should be discharged at a suitable location unlikely to result in significant erosion.

Concrete Slabs-on-Grade

Although not expected, any interior concrete slabs should be reinforced with steel bars (not wire mesh) and should be designed by the Structural Engineer. Contraction joints should be incorporated in the concrete slab in both directions, no greater than 10 feet on center, and reinforcing bars should extend continuously through the control joints.

To improve interior moisture conditions, a 4-inch minimum layer of clean, free draining, 3/4-inch angular gravel or crushed base rock should be placed beneath the interior concrete slabs to form a capillary moisture break. The base rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15-mils or thicker, should be placed over the drain rock. The vapor barrier should meet the Class A requirements outlined in ASTM E 1745 and be installed per ASTM 1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth or other adverse conditions.

Exterior concrete slabs should be at least 4-inches thick and reinforced as described above for interior slabs. Exterior slabs should be underlain with 4-inches or more of Caltrans Class 2 Aggregate Base compacted to at least 92 percent relative compaction. Some movement should be expected for exterior concrete slabs as the underlying soils react to seasonal moisture changes. For improved performance, the exterior slabs can be thickened and reinforced as described above for interior slabs and/or underlain with a thicker aggregate base layer.

Utility Trench Excavation and Backfill

Excavations for utilities will typically encounter loose sandy soils. Trench excavations having a depth of five feet or more and will be entered by workers must be sloped, braced, or shored in accordance with current Cal/OSHA regulations. For the purpose of shoring design, onsite soils should be considered “Type C”, and will be prone to raveling and collapse in open excavations. All excavations where collapse of excavation sidewall, slope or bottom could result in injury or death of workers should be evaluated by the contractor’s safety officer and designated competent person prior to entering in accordance with current Cal/OSHA regulations.

Bedding materials for utility pipes should be well graded sand with 90 to 100% of particles passing the No. 4 sieve and no more than 5% finer than the No. 200 sieve. The Contractor shall provide the minimum bedding beneath the pipe in accordance with the manufacturer’s recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils, moisture conditioned to within 2% of the optimum moisture content, placed in thin lifts and compacted to a minimum of 90% relative compaction. Backfill for trenches within pavement areas should consist of non-expansive granular fill. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits. Where utility lines cross under or through perimeter footings, they should be sealed to reduce moisture intrusion into the areas under the slabs and/or footings.

Supplemental Services

We are happy to provide consultation and respond to any geotechnical questions or issues that arise during project planning and design. We should be present intermittently during construction to verify that actual conditions encountered are consistent with our criteria and recommendations, and to revise or otherwise alter our recommendations as needed.

We trust that this letter presents the information required at this time. Should there be any questions or concerns regarding our investigation, please do not hesitate to contact us.

Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

NATHAN G. KLEMIN
GE 3168
EXP. 03/31/25

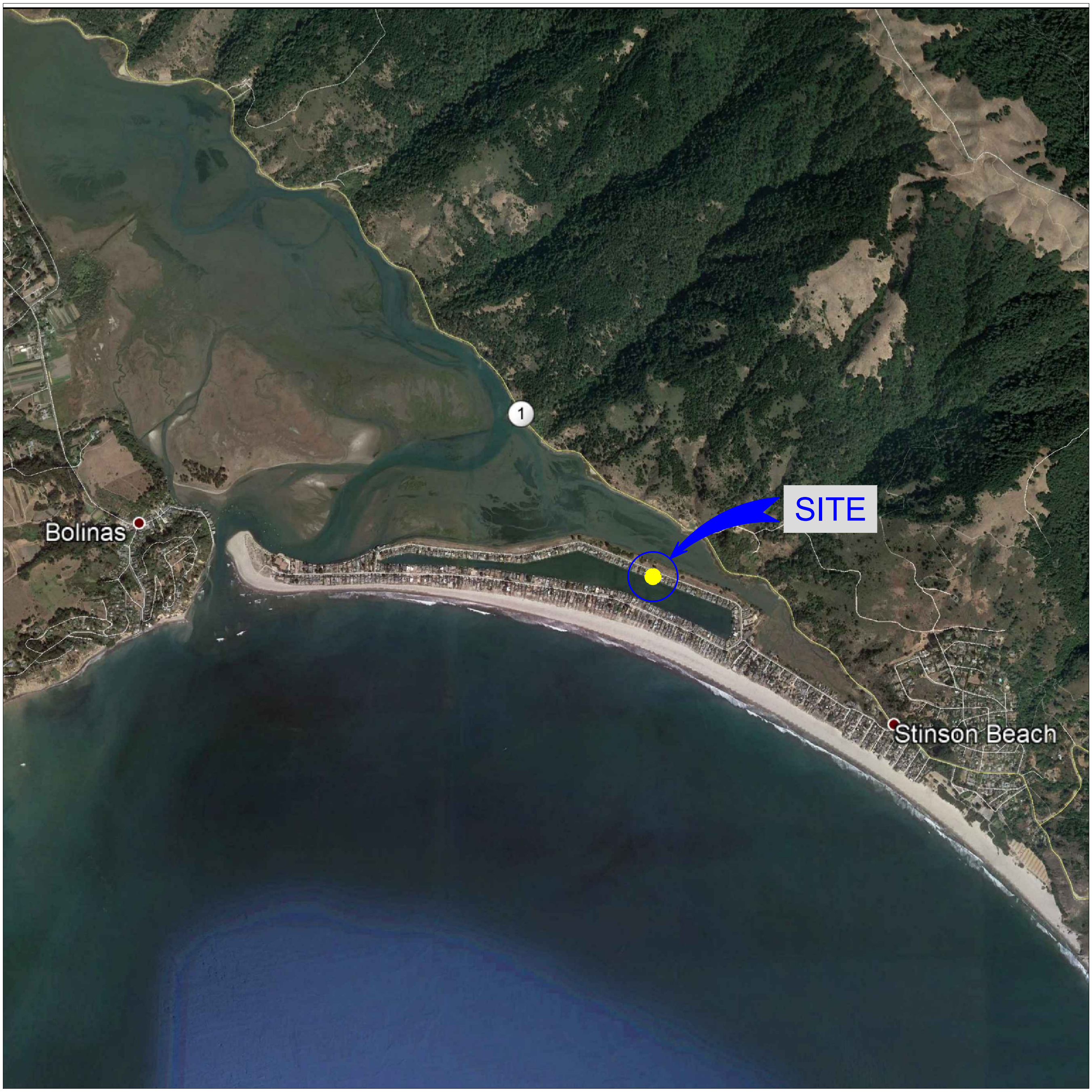
Nathan Klemin
Geotechnical Engineer No. 3168
(Expires 3/31/25)

REVIEWED BY:

SCOTT A. STEPHENS
GE 2398
EXP. 06/30/25

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/25)

Attachments: Figures 1 through 9
Appendix A – Subsurface Exploration and Laboratory Testing
Appendix B – Logs of Previous Subsurface Exploration



SITE COORDINATES
LAT. 37.907138°
LON. -122.657756°

SITE LOCATION

N.T.S.

REFERENCE: Google Earth, 2024

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FILENAME: 3600.001 Figures.dwg

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SITE LOCATION MAP

Proposed Residential Addition
65 Dipsea Road
Stinson Beach, California
Project No. 3600.001
Date: 7/8/2024

Drawn CMS
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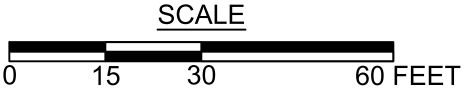
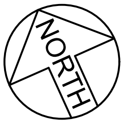
1
FIGURE

SITE PLAN





● Approximate boring location completed by MPEG, 2024

REFERENCE: Google Earth, 2024

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SITE PLAN

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2
FIGURE

REGIONAL GEOLOGIC MAP

NORTH

LEGEND

Qs - BEACH SANDS (QUATERNARY)
Typically well sorted sands and gravels deposited by water

KJf - FRANCISCAN MELANGE (CRETACEOUS-JURASSIC)
Tectonic mixture of resistant rock types, including sandstone, chert, serpentinite, basalt, and high-grade metamorphic rocks embedded in a sheared shale matrix.

REFERENCE: Clark, J.C. and Brabb, E.E., “Geology of Point Reyes National Seashore and Vicinity, Marin County, California: A Digital Database,” U.S. Geological Survey, Open-File Report OF-97-456, Map Scale 1:48,000, 1997.

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REGIONAL GEOLOGIC MAP

Proposed Residential Addition
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Project No. 3600.001 Date: 7/8/2024

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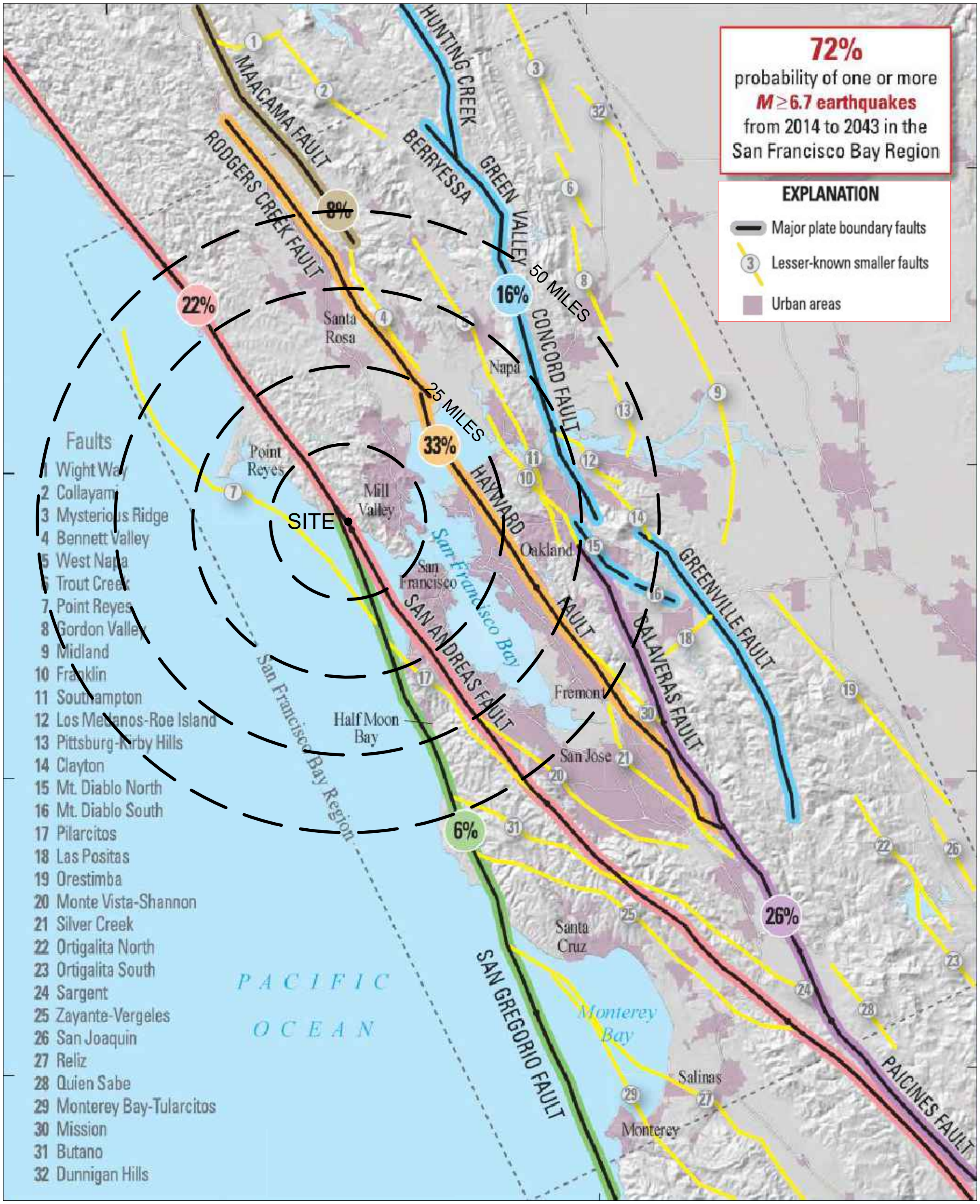
3
FIGURE



SITE COORDINATES
LAT. 37.907138°
LON. -122.657756°

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

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Project No. 3600.001 Date: 7/8/2024

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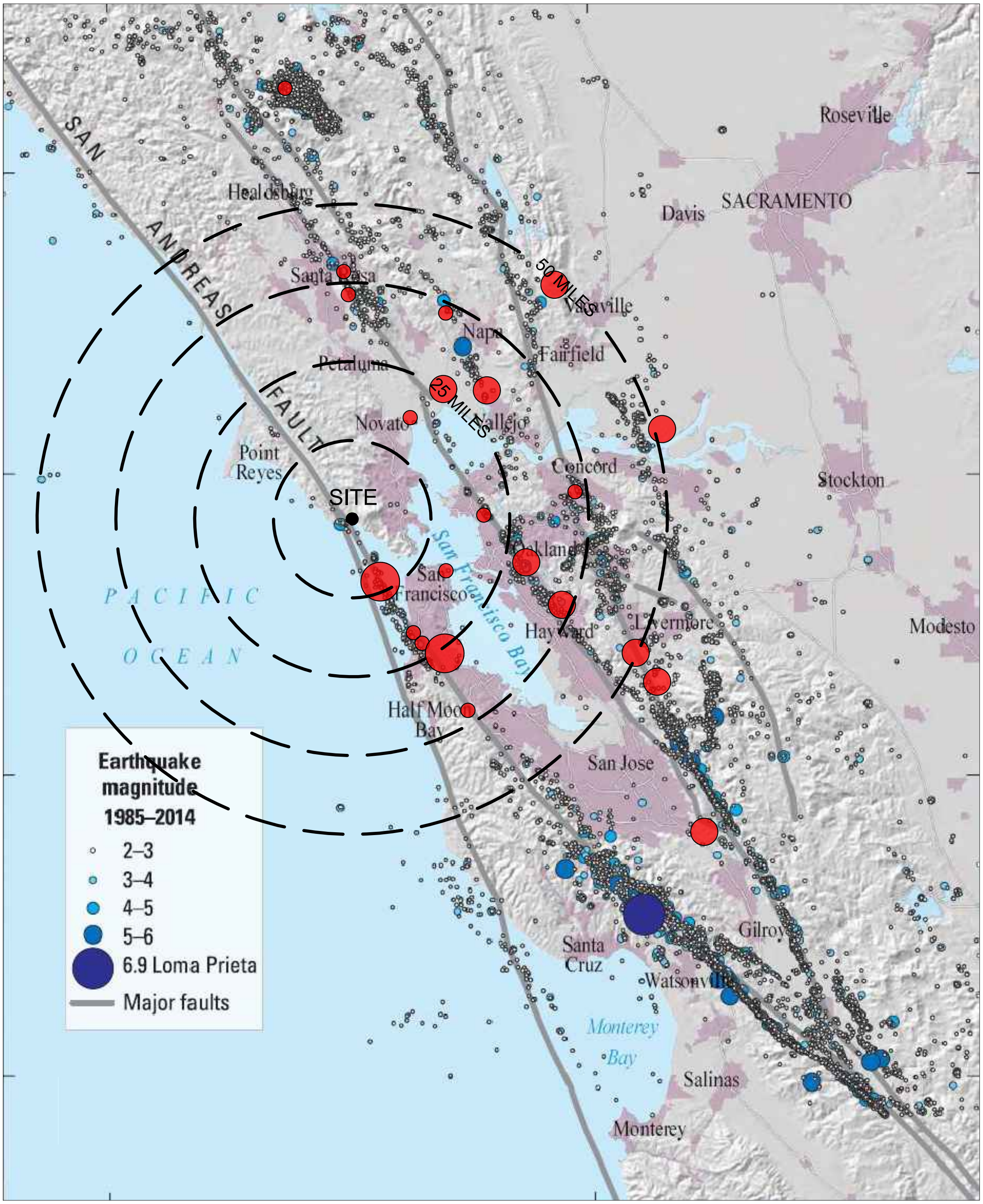
4
FIGURE



SITE COORDINATES
LAT. 37.907138°
LON. -122.657756°

SCALE
0 12.5 25 50 MILES

NORTH

LEGEND & DATA SOURCE:

● ● ● See legend above. 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).

● ● ● Large circles indicate earthquakes M>7.0, medium circles indicate 6.0<M<7.0 and small circles indicate 5.0<M<6.0 .U.S. Geological Survey, Earthquake Catalog Search, https://earthquake.usgs.gov/earthquakes/search/. Earthquakes between 1830 and 2021.

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HISTORIC EARTHQUAKE MAP

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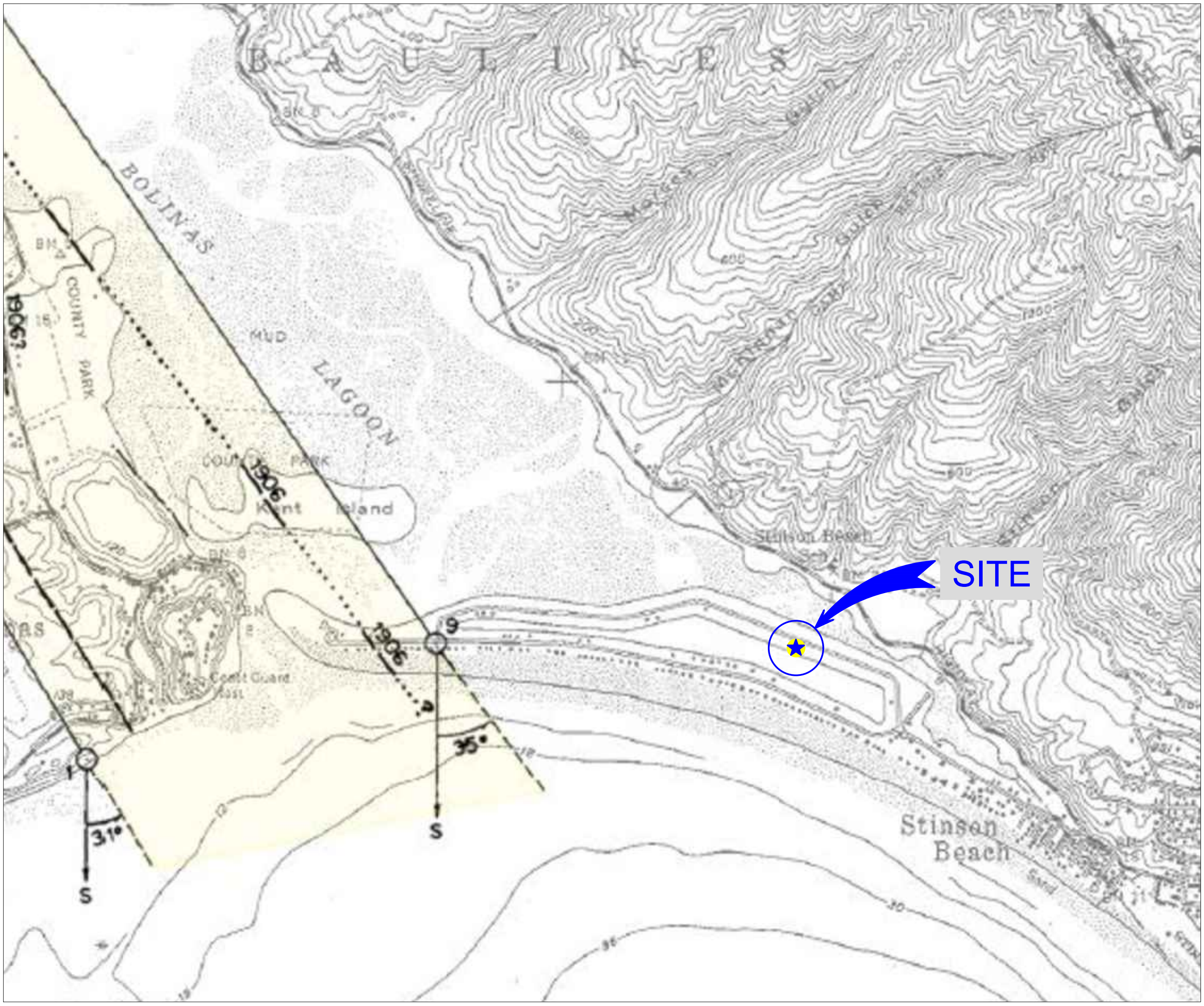
5
FIGURE



MAP EXPLANATION

Potentially Active Faults

Faults considered to have been active during Quaternary time; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Aerial photo lineaments (not field checked); based on youthful geomorphic and other features believed to be the results of Quaternary faulting.

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect consecutively numbered turning points so as to define one or more special studies zone segments.

Seaward projection of zone boundary.

NORTH

REFERENCE: California Department of Conservation, Division of Mines and Geology, "State of California Special Studies Zones - Bolinas Quadrangle", effective July 1, 1974.

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ALQUIST PRIOLO EARTHQUAKE FAULT ZONE MAP

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Project No. 3600.001 Date: 7/8/2024

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6
FIGURE

REFERENCE: ABAG Geographic Information System.

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LIQUEFACTION SUSCEPTIBILITY MAP

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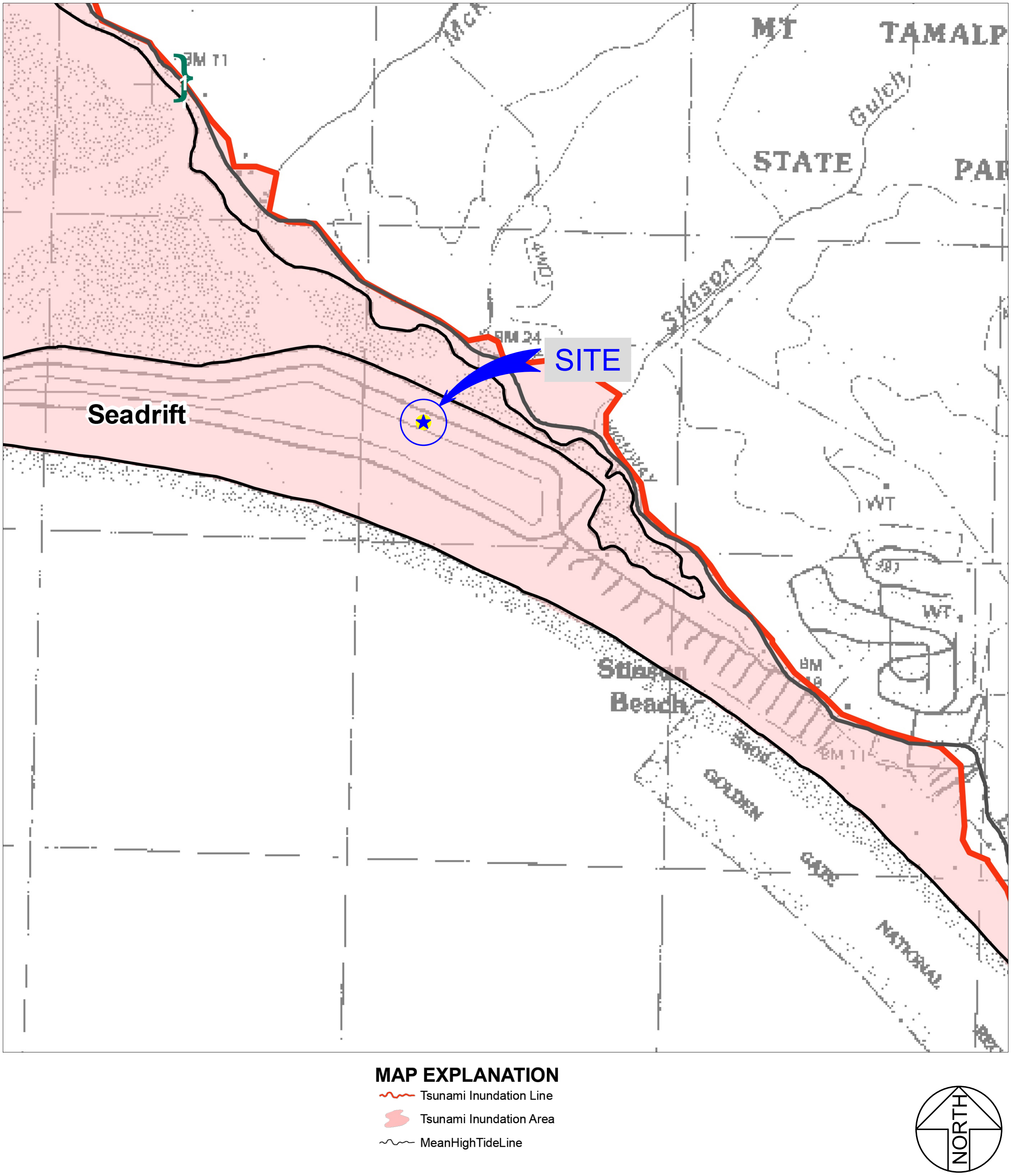
7
FIGURE



REFERENCE: California Emergency Management Agency (CalEMA)(2009), "Tsunami Inundation Map for Emergency Planning, State of California - County of Marin, Bolinas Quadrangle", Map Scale 1:24,000, dated July 1, 2009.

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TSUNAMI INUNDATION MAP

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Project No. 3600.001
Date: 7/8/2024

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8
FIGURE

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

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

REFERENCE: ABAG Geographic Information System.

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FEMA FLOOD MAP

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Project No. 3600.001 Date: 7/8/2024

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9
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

1.0 Subsurface Exploration

Subsurface exploration for the project included one shallow soil boring at the locations shown on Figure 2 on July 10, 2024. The boring was excavated to a depth of about 5-feet below the ground surface using manual hand auger equipment and sampling equipment. Relatively “undisturbed” samples were collected by driving a 2.5-inch inside diameter, 6-inch-long stainless steel “Modified California” sampler liner. The soils encountered were logged and identified in the field 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 the boring log is presented on Figure A-2.

The samples obtained during our exploration were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

2.0 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; and
- Amount of Material in Soils Finer than No. 200 (75-µm) Sieve, ASTM D 1140.

The results of our laboratory testing are shown on the exploratory Boring Log. The exploratory boring logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the boring 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 conditions.

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

UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress
DS (2.0)	DRAINED DIRECT SHEAR (NORMAL PRESSURE, ksf)

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

Proposed Residential Addition
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Project No. 3600.001 Date: 7/18/2024

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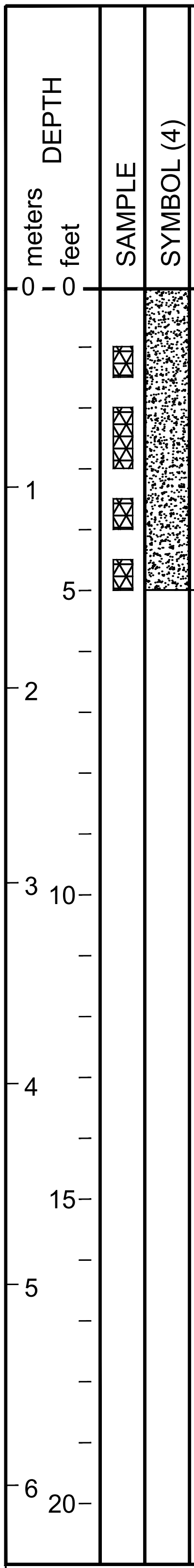
A-1
FIGURE

BORING 1

EQUIPMENT: 3.75-inch Bucket Manual Hand Auger
DATE: 7/10/24
ELEVATION: 11 - feet*
*REFERENCE: Google Earth, 2024



	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
SAND (SP) Light brown, dry, loose, up to 5% fines. [Alluvium]						
Grades to moist.		105	3.2%		P200: 3.2%	
		101	5.1%		P200: 4.8%	
		103	4.2%			
Grades moist to wet and light gray.			13.6%			
Bottom of boring at 5.0 feet. No groundwater encountered.						

▽ Water level encountered during drilling
▼ Water level measured after drilling

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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SOIL CLASSIFICATION CHART

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Project No. 3600.001 Date: 7/18/2024

Drawn CMS
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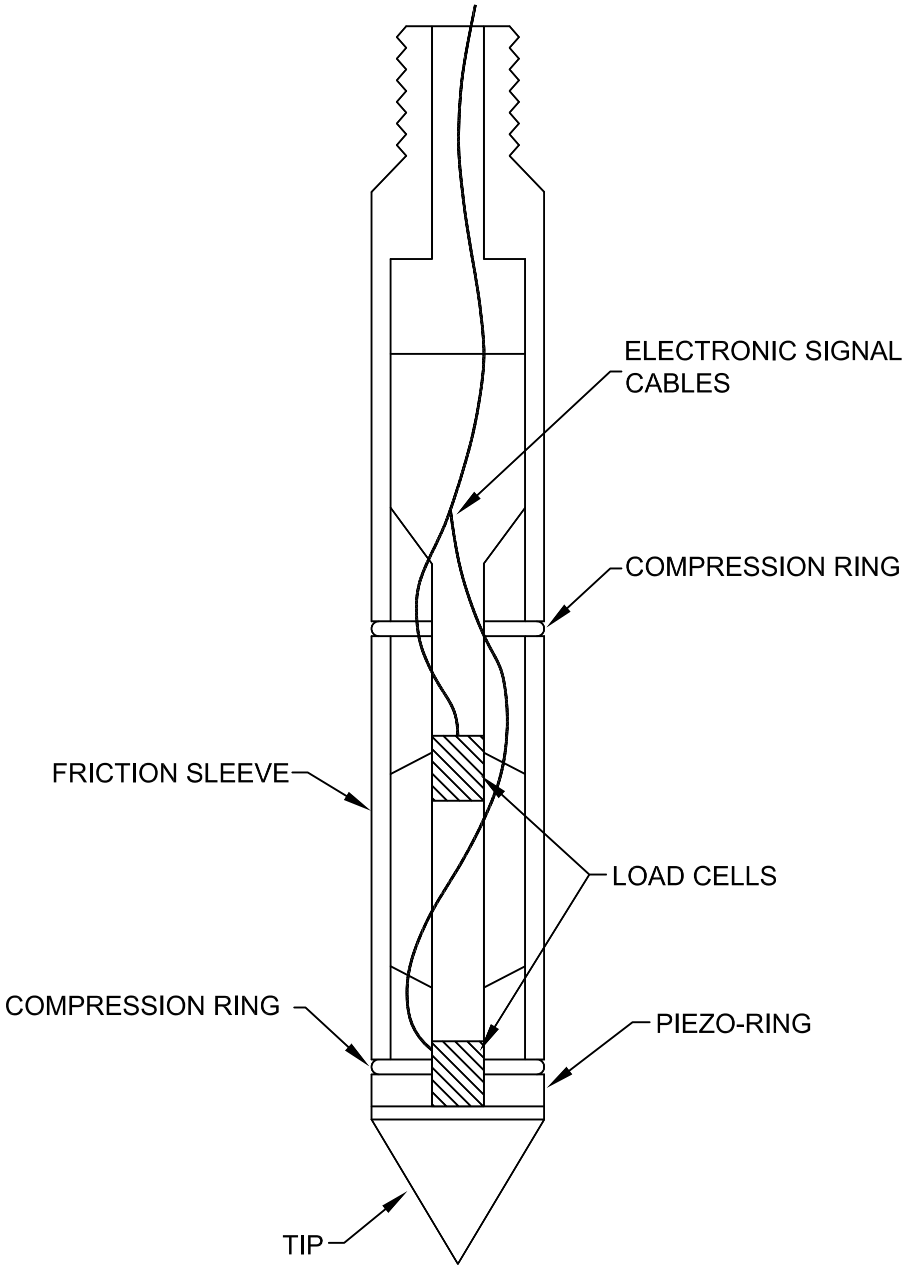
A-2
FIGURE

APPENDIX B
LOGS FROM PREVIOUS SUBSURFACE EXPLORATION



CONE PENETROMETER

(NO SCALE)

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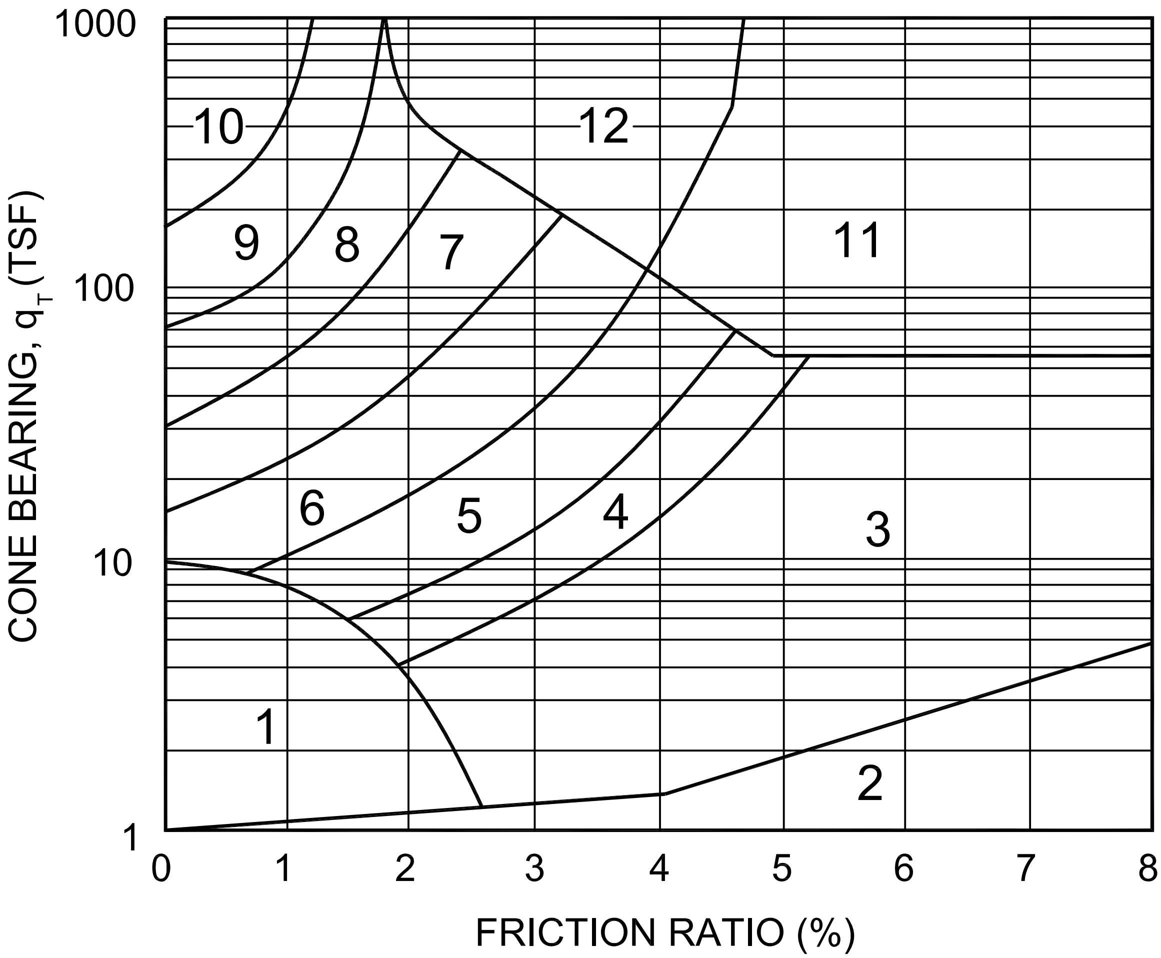
CONE PENETROMETER DIAGRAM

263-265 Seadrift Road
Stinson Beach, California

Project No. 2277.001 Date: 4/5/16

Drawn NGK
Checked

A-1
FIGURE



Zone:	Qc/N	Soil Behavior Type:
1)	2	Sensitive Fine Grained
2)	1	Organic Material
3)	1	Clay
4)	1.5	Silty Clay to Clay
5)	2	Clayey Silt to Silty Clay
6)	2.5	Sandy Silt to Clayey Silt
7)	3	Silty Sand to Sandy Silt
8)	4	Sand to Silty Sand
9)	5	Sand
10)	6	Gravelly Sand to Sand
11)	1	Very Stiff Fine Grained (*)
12)	2	Sand to Clayey Sand (*)

(*) Overconsolidated or Cemented

Reference: Robertson, P.K. (1986), "In-Situ Testing and Its Application to Geotechnical Engineering," Canadian Geotechnical Journal, Vol. 23; No. 23; No. 4, pp. 573-594

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CPT SOIL INTERPRETATION CHART

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Stinson Beach, California

Project No. 2277.001 Date: 4/5/16

Drawn NGK
Checked

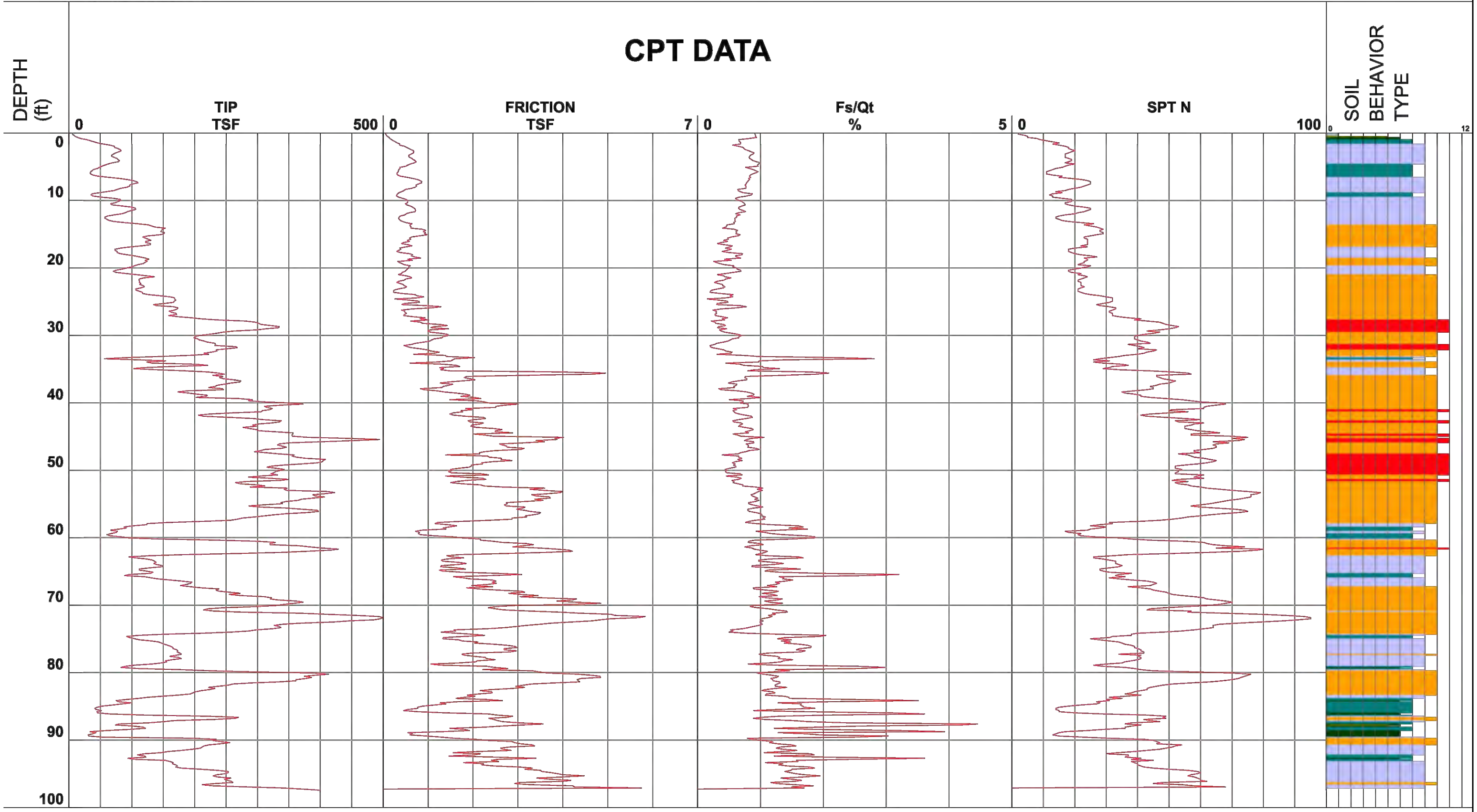
A-2
FIGURE

Miller Pacific Engineering Group

Project	263 Seadrift	Operator	BH	Filename	SDF(623).cpt
Job Number	2277.001	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	3/21/2016 8:57:57 AM	Maximum Depth	97.44 ft
EST GW Depth During Test	9.30 ft				

Net Area Ratio .8

CPT DATA



- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained (*)
- ■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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CPT-1 PLOT

263-265 Seadrift Road
Stinson Beach, California

Project No. 2277.001 Date: 4/5/16

Drawn NGK
Checked

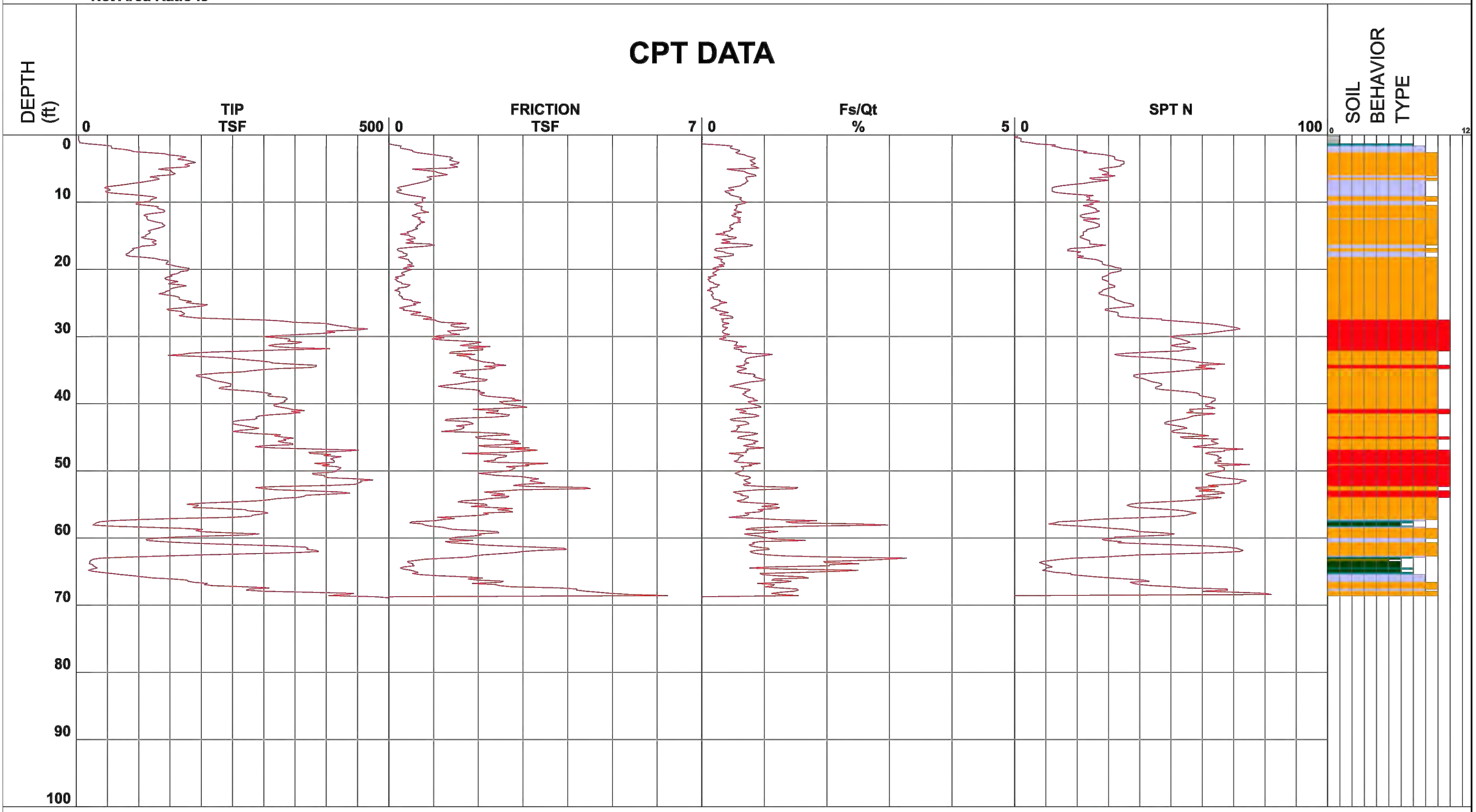
A-3
FIGURE

Miller Pacific Engineering Group

Project	263 Seadrift	Operator	BH	Filename	SDF(625).cpt
Job Number	2277.001	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	3/21/2016 11:15:54 AM	Maximum Depth	68.90 ft
EST GW Depth During Test		10.50 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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CPT-2 PLOT

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Project No. 2277.001 Date: 4/5/16

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A-4
FIGURE

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

AL	ATTERBERG LIMITS TEST
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

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 TYPE

■	MODIFIED CALIFORNIA
▯	STANDARD PENETRATION TEST
▨	THIN-WALLED / FIXED PISTON
⊠	HAND SAMPLER
⊠	ROCK CORE
X	DISTURBED OR BULK SAMPLE

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

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.

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SOIL CLASSIFICATION CHART

263-265 Seadrift Road
Stinson Beach, California

Project No. 2277.001 Date: 4-6-16

Drawn MFJ
Checked

A-5
FIGURE

BORING 1

EQUIPMENT: 3.25 inch manual bucket auger

DATE: 3-24-16
ELEVATION: 9-Feet*
*REFERENCE: Topo by Oberkamper, 2016

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	
						0	0		▨	SAND (SP Light brown, dry, loose, fine-grained [AEOLIAN DUNE SAND] Grades moist, medium dense at 1.0-feet.
				5.1	101		1	■	▨	
										Boring terminated at 1.5-feet. No groundwater encountered during exploration.
							2			
							3			
						1				
							4			
							5			
							6			
						2				
							7			
							8			
							9			
						3				
							10			

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

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BORING LOG

263-265 Seadrift Road
Stinson Beach, California

Project No. 2277.001 Date: 4-6-16

Drawn MFJ
Checked

A-6
FIGURE

BORING 2

EQUIPMENT: 3.25 inch manual bucket auger

DATE: 3-24-16
ELEVATION: 14-Feet*
*REFERENCE: Topo by Oberkamper, 2016

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	
						0	0		▨	SAND (SP Light brown, dry, loose, fine-grained [AEOLIAN DUNE SAND] Grades moist, medium dense at 1.0-feet.
				5.5	102		1	■	▨	
										Boring terminated at 1.5-feet. No groundwater encountered during exploration.
							2			
							3			
						1				
							4			
							5			
							6			
						2				
							7			
							8			
							9			
						3	10			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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Project No. 2277.001 Date: 4-6-16

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A-7
FIGURE

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
▯	STANDARD PENETRATION TEST
▨	THIN-WALLED / FIXED PISTON
▩	HAND SAMPLER
☒	ROCK CORE
X	DISTURBED OR BULK SAMPLE

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

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.

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SOIL CLASSIFICATION

177 Dipsea Road
Stinson Beach, California

Project No. 1904.01 Date: 3/15/13

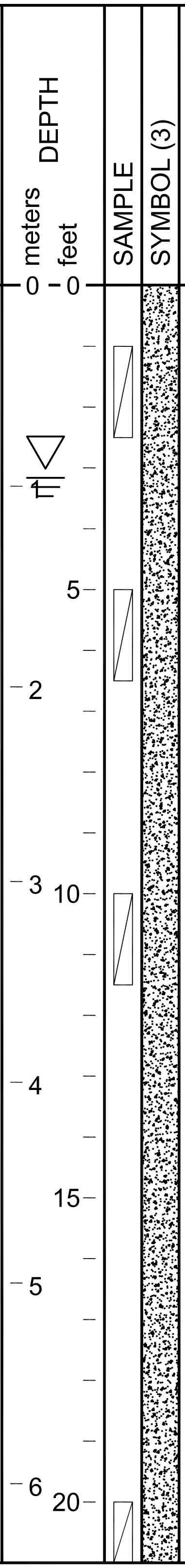
Drawn JTO
Checked

A-1
FIGURE

BORING 1

EQUIPMENT: Track-Mounted SIMCO 4000TR
with 6-inch Hollow Stem Auger
DATE: 3/8/13
ELEVATION: 10.5-Feet (+/- MSL)*
*REFERENCE: Nelson Engineering,
September 19, 2011.

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	
						SAND (SP) Light gray to tan, slightly moist, loose, poorly graded, fine to very fine grained.
			11			
						Grades gray, wet, occasional medium grained sand and subrounded to rounded gravels to 1/2-inch, occasional shell fragments at 3 feet.
			4			Grades ~50% fine grained ~50% medium grained .sand
			11			



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

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BORING LOG

177 Dipsea Road
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Project No. 1904.01 Date: 3/15/13

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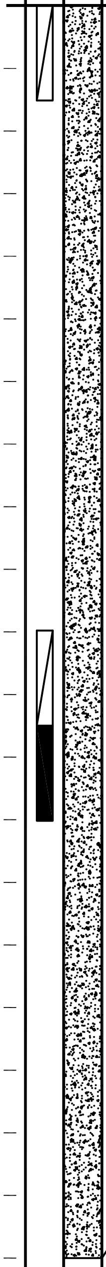
A-2
FIGURE

FILE: 1904.01 BL.dwg

BORING 1
(CONTINUED)



OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters feet	SAMPLE	SYMBOL (3)	
						20			SAND (SP) Light gray to tan, wet, loose, poorly graded, fine to medium grained, occasiona subrounded to rounded gravels and shell fragments.
	P200 3.8%		10						
						7			
						25			
						8			
						9			
						30			Grades medium dense at 30 feet.
	P200 7.3%		30	22.1	103				
						10			
			47	23.2	95				
						35			
						11			No recovery or blowcounts below 33 feet due to loose wet sands.
						12			
						40			Bottom of boring at 40 feet. Groundwater measured at 3 feet one hour after drilling.

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

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BORING LOG

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Stinson Beach, California

Project No. 1904.01 Date: 3/15/13

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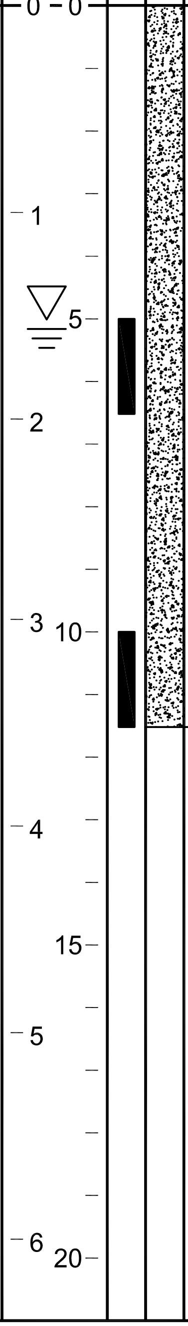
A-3
FIGURE

BORING 2

EQUIPMENT: Track-Mounted SIMCO 4000TR with 6-inch Hollow Stem Auger
DATE: 3/8/13
ELEVATION: 9.5-Feet (+/- MSL)*
*REFERENCE: Nelson Engineering, September 19, 2011.



OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH feet	DESCRIPTION
						0	SAND (SP) Light gray to tan, slightly moist, loose, poorly graded, fine to very fine grained.
						3	Grades gray, wet, occasional medium grained sand and subrounded to rounded gravels to 1/2-inch, occasional shell fragments at 3 feet.
			9	19.6	94	5	Grades ~50% fine grained ~50% medium grained .sand
	P200 1.1%		11			10	
						11.5	Bottom of boring at 11.5 feet. Groundwater measured at 5 feet immediately after drilling.

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

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Project No. 1904.01 Date: 3/15/13

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A-4
FIGURE

BORING 3

EQUIPMENT: Track-Mounted SIMCO 4000TR
with 6-inch Hollow Stem Auger
DATE: 3/8/13
ELEVATION: 11.0-Feet (+/- MSL)*
*REFERENCE: Nelson Engineering,
September 19, 2011.

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	
						0	0		▨	SAND (SP) Light gray to tan, slightly moist, loose, poorly graded, fine to very fine grained.
						1			▨	Grades gray, wet, occasional medium grained sand and subrounded to rounded gravels to 1/2-inch, occasional shell fragments at 3 feet.
			13	6.1 6.3	92 90		5	■	▨	
						2				Bottom of boring at 5.5 feet. No groundwater encountered during drilling.
						3	10			
						4				
							15			
						5				
						6	20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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Project No. 1904.01 Date: 3/15/13

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A-5
FIGURE