Appendix E:
Geology and Soils Supporting Information

THIS PAGE INTENTIONALLY LEFT BLANK

E.1 - Preliminary Geotech Report

THIS PAGE INTENTIONALLY LEFT BLANK

PRELIMINARY GEOTECHNICAL INVESTIGATION
SCANNELL PROPERTIES
PARR BOULEVARD DEVELOPMENT
RICHMOND, CALIFORNIA

April 17, 2018

Project 2042.002

Prepared For:
Scannell Properties
3569 Mt. Diablo Blvd, Suite 220
Lafayette, California 94549

Attn: Mr. Todd Berryhill

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of radon, asbestos, toxic mold and other biological pollutants, and other hazardous materials. The document is for the sole use of the client and consultants on this project. Use by third parties or others is expressly prohibited without written permission. If the project changes, or more than two years have passed since issuance of this report, the findings and recommendations must be reviewed by the undersigned.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY:

Phaidra Campbell
Professional Engineer No. 81850
(Expires 3/31/20)

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

MILLER PACIFIC
ENGINEERING GROUP

PRELIMINARY GEOTECHNICAL INVESTIGATION
SCANNELL PROPERTIES
PARR BOULEVARD DEVELOPMENT
RICHMOND, CALIFORNIA

TABLE OF CONTENTS

MILLER PACIFIC
ENGINEERING GROUP

FIGURES

APPENDIX A – SUBSURFACE EXPLORATION

PRELIMINARY GEOTECHNICAL INVESTIGATION
SCANNELL PROPERTIES
PARR BOULEVARD DEVELOPMENT
RICHMOND, CALIFORNIA

I.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed commercial development for Scannell Properties, located within an undeveloped lot on the northeast corner of the intersection of Parr Boulevard and Richmond Parkway in Richmond, California. The project site location is shown on Figure 1. The purpose of our services is to investigate subsurface conditions, evaluate geologic hazards, and develop geotechnical design criteria and recommendations for use in project design and construction. This report is intended for the sole use of Scannell Properties and the design team for this project and site. No other use is authorized without the express written consent of Miller Pacific Engineering Group.

Our services are provided in accordance with the terms of our Agreement for Professional Engineering and Testing Services dated October 26, 2017. The scope of our services is outlined in our proposal letter dated October 26, 2017, and includes the following:

- Review the subsurface conditions from our previous geotechnical investigation for this site and review the proposed grading and development plans.
- Performed geotechnical analyses for various development options to estimate site settlements and required foundation systems for the different options.
- The previous geotechnical investigation and report for this site included:
- Review of available geologic mapping and geotechnical background information from City and County files as well as our in-house library;
- Exploration of subsurface conditions with two auger borings;
- Evaluation of potential geologic hazards and respective mitigation measures;
- Development of foundation options and corresponding geotechnical design criteria;
- Development of recommendations and design criteria for site preparation and grading;
- Analysis of anticipated total and differential site settlements;
- Development of recommendations for underground utility trench excavation and backfill;
- Development of alternative pavement sections; and
- Other geotechnical items relevant to the proposed development.

The scope of our geotechnical services does not include any investigation or evaluation of the potential for contaminated soils or hazardous materials to be present at the project site.

2.0 PROJECT DESCRIPTION

The project consists of the development of an approximately 30-acre property located northeast of the intersection of Richmond Parkway and Parr Boulevard in Richmond, California. The project site

is relatively level. Several existing structures exist on the site and review of historic aerial photos indicate that many other structures had been located on the property and have been demolished and removed.

The proposed project is an industrial/commercial development which includes two structures of roughly 119,000 square feet and 206,000 square feet with associated driveways, parking, and site access improvements. The proposed buildings are planned to be rectangular shaped with the long dimension in the north-south direction. Anticipated loading conditions for the floor and columns of the proposed building have not yet been defined by the Structural Engineer. Once defined, we will update our evaluation based on these loading conditions. The finished floor will also be several feet above the surrounding grade. Fill placement to raise grades would add another several hundred pounds per square foot of new fill pressure to the underlying soils. Asphalt-concrete parking, heavy truck traffic drive aisles and loading docks are planned around the new structure. New underground utilities, walkways and landscape improvements are anticipated. The proposed improvements are shown on Figure 2.

The design team for the currently proposed development includes Scannell Properties (Owner) and HPA Architecture (Architects). Other team members are not known at this time.

3.0 SITE CONDITIONS

3.1 Regional Geology

The site is located within the Coast Range Geomorphic Province of California. The regional bedrock geology consists of complexly folded, faulted, sheared, and altered sedimentary, igneous, and metamorphic rock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex.

Northwest-southeast trending mountain ridges and intervening valleys that were formed from tectonic activity between the North American Plate and the Pacific Plate characterize the regional topography. Extensive faulting during the Pliocene Age (1.8-7 million years ago) formed the uneven depression that is now the San Francisco Bay. More recent tectonic activity is concentrated along the San Andreas Fault zone, a complex group of generally parallel faults.

Regional geologic mapping indicates that the project site is underlain by alluvial basin deposits. These alluvial deposits typically consist of layers of silty clay and clay. The northwest edge of the site is also mapped near a contact with Bay Mud. Bay Mud typically is composed of highly compressible, very soft, high plasticity clay and silty clay, and commonly includes seams and lenses of fine sands and organic materials such as peat. A regional geologic map is on Figure 3.

3.2 Seismicity

3.2.1 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 both east and west of the site including the San Andreas, Contra Costa, Hayward, Rodgers Creek, and San Gregorio Faults. 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 sign of recent rupture. The California Department of Conservation, Division of Mines and Geology and the U.S. Geological Survey has mapped various active and inactive faults in the region (CDMG, 1972, 2000 and USGS 2016). These faults, are shown in relation to the project site on the attached Active Fault Map, Figure 4. The Hayward Fault and Contra Costa Shear Zone are the nearest known active faults to the site, located about 1.5 and 20 kilometers east of the site, respectively.

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that at least 23 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 150 kilometers (96 miles) of the site area between 1900 and 2015. The six most significant historic earthquakes to affect the project site are summarized in Table A.

TABLE A
SIGNIFICANT EARTHQUAKE ACTIVITY
Scannell Properties
Parr Blvd. and Richmond Parkway
Richmond, California

Epicenter (Latitude, Longitude)	Historic Richter Magnitude	Year
38.822, -122.841	5.0	2016
38.216, -122.312	6.0	2014
38.379, -122.413	5.0	2000
37.737, -121.740	5.0	1980
38.296, -122.755	5.7	1969
37.750, -122.550	7.7	1906

Reference: USGS (2018)

3.2.3 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"[1,2,3] 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.

The 2003 study (UCERF) specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region of northern California. The 2008 study (UCERF2) applied many of the analyses used in the 2003 study to the entire state of California and updated some of the analytical methods and models. The most recent 2013 study (UCERF3) further expanded the database of faults considered and allowed for consideration of multi-fault ruptures, among other improvements. As a result, the apparent over-prediction of moderate (M6.5-7.0) earthquakes generated by the UCERF2 model has been removed, and the UCERF3 model suggests an approximate 43% increase in the rate of all M>5.0 earthquakes statewide.

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 San Andreas Fault, located approximately 26 kilometers southwest of the site, at 33%. The nearest known active fault, the Hayward Fault, is assigned a 33% probability of an M>6.7 earthquake by 2045. 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.

---

[1] United States Geological Survey (2003), "Summary of Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2032," The 2003 Working Group on California Earthquake Probabilities, 2003.
[2] United States Geological Survey (2008), "The Uniform California Earthquake Rupture Forecast, Version 2," The 2007 Working Group on California Earthquake Probabilities, Open File Report 2007-1437, 2008.
[3] Field, E.H. et al (2015), "Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3)", Bulletin of the Seismological Society of America, Volume 105, No. 2A, 33pp., April 2015, doi: 10.1785/0120140093

3.3 Surface Conditions

The project site is bounded to the north and west by Richmond Parkway, to the east by General Petroleum, and to the south by Parr Boulevard. Existing site elevations within the proposed development area range from approximately +9 to +12-feet MSL (mean sea level), and generally slope gently down to the north. The project area is relatively flat and covered with vegetation and wild grasses.

3.4 Subsurface Exploration and Laboratory Testing

Subsurface exploration at the project site includes previous exploratory borings and Cone Penetration Tests (CPTs). The two (2) borings were drilled by Miller Pacific with truck-mounted equipment on May 29, 2015. The five (5) CPTs were completed by Miller Pacific on June 1, 2015. The locations of the CPTs and borings are shown on Figure 2. The field exploration and laboratory testing program is discussed in more detail in Appendix A.

Boring 1 was excavated using a truck-mounted drill rig which utilized rotary wash drilling to a maximum explored depth of 76.5 feet. Materials encountered were logged by our field geologist and select samples retained for laboratory testing. A soil classification chart is presented on Figure A-1 and provides a brief explanation of the terms and methods used in identifying and describing the subsurface materials encountered. Exploratory boring logs are presented on Figures A-2 through A-6.

The Cone Penetration Test (CPT) is an exploration technique that provides a continuous profile of data throughout the depth of exploration. It is particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential. The device is illustrated on Figure A-7. The recorded data is transferred to an in-office computer for reduction and analysis. The cone tip bearing and sleeve friction (i.e., friction ratio) indicates the soil type, soil density or strength. Variations in the data profile indicate changes in stratigraphy. This test method has been standardized and is described in detail by the ASTM Standard Test Method D3441 "Deep, Quasi-Static Cone and Friction Cone Penetration Tests of Soil." CPT plots of interpreted subsurface conditions are shown on Figures A-8 through A-12.

Laboratory testing included determination of in-situ dry density, moisture content, Atterberg limits, consolidation, triaxial confined, and unconfined compressive strength. Corrosion testing was performed on a composite soil sample obtained from soils in Borings 1 and 2. Most of the laboratory testing results are presented on the boring logs. The Atterberg limits test results is presented on Figure A-13 and the consolidation test results are presented on Figures A-14 and A-15. The corrosion test results are presented on Figure A-16.

3.5 Subsurface Conditions

Subsurface data generally confirm the regionally mapped geology. The upper 5 feet at the project site is composed of medium stiff, sandy clay. From about 5 to 30 feet, subsurface conditions consist of soft to medium stiff, silty clay. Below 30 feet the clay is slightly stiffer and extends to depths in excess of 125 feet.

Groundwater was not observed in the borings during drilling due to the low permeability clayey soils. However, groundwater monitoring wells in the vicinity have measures groundwater levels at depths between 2- and 7-feet below the ground surface. Groundwater should generally be expected in onsite excavations deeper than about 3 feet below grade and may be shallower during the winter months or following periods of heavy rain.

4.0 GEOLOGIC HAZARDS EVALUATION

4.1 Summary

This section identifies potential geologic hazards at the project site, their significant adverse impacts, and respective mitigation measures. Based on our evaluation, the primary geologic hazards to be considered during project planning and design are strong seismic ground shaking, settlement, expansive soils and flooding. Other geologic hazards are not considered significant at the site. More detailed discussion of geologic hazards and mitigation measures is presented below.

4.2 Fault Surface Rupture

Under the Alquist-Priolo Special Studies Zone Act[4], the California Department of Conservation, Division of Mines and Geology (1972, 2000) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. The project site is not located within an Alquist-Priolo Special Studies Zone and has a relatively thick clay layer over the bedrock. The potential for fault surface rupture is therefore low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.3 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active San Francisco Bay Area. Earthquakes along any of 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

[4] The Alquist-Priolo Earthquake Fault Zoning Act prohibits placing most structures for human occupancy across traces of active faults. These fault zones are shown on maps issued by the Department of Conservation's Division of Mines and Geology.

will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. The design seismic motions also depend on the evaluation method used.

Both deterministic and probabilistic evaluations have been preformed to estimate the strong seismic shaking at the site. Deterministic methods are commonly used for the majority of residential, commercial, and industrial developments. Probabilistic methods are used for "critical" facilities such as hospitals and schools or where "superior" seismic performance is desired.

Deterministic Seismic Hazard Analysis – Deterministic Seismic Hazard Analysis (DSHA) 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 provide approximate estimates of median peak site accelerations (PGA). A summary of the principal active faults affecting the site, their closest distance, earthquake moment magnitude and probable peak ground accelerations associated with each fault are shown in Table B. These acceleration values are for an earthquake originating on the closest portion of the fault to the site.

TABLE B
ESTIMATED PEAK GROUND ACCELERATION
FOR PRINCIPAL ACTIVE FAULTS
Scannell Properties
Parr Blvd. and Richmond Parkway
Richmond, California

Fault	Moment Magnitude for Characteristic Earthquakes(1)	Closest Estimated Distance(1)	Median Peak Ground Acceleration(1,2)
Hayward	7.3	1.5 km	0.48 g
San Andreas	8.0	25.8 km	0.21 g
Rodgers Creek	7.3	22.9 km	0.18 g
San Gregorio	7.4	28.0 km	0.16 g
Contra Costa Shear Zone	6.5	19.4 km	0.16 g

(1) Caltrans ARS Online, V2.3.09 (web-based acceleration response spectra calculator tool), http://dap3.dot.ca.gov/ARS_Online/, accessed April 13, 2018.
(2) Values determined using Vs^{30} = 270 m/s for Site Class "D" in accordance with the 2016 California Building Code.

Probabilistic Seismic Hazard Analysis – Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely

dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the PGA for two separate probabilistic conditions, the 2% chance of exceedance in 50 years (2,475-year statistical return period) and the 10% chance of exceedance in 50 years (475-year statistical return period), utilizing the 2014 Interactive Deaggregation (USGS, 2014). The PGA arising from a probabilistic analysis for a 10% chance of exceedance in 50 years are commonly utilized for residential, commercial, and industrial developments, while the PGA arising from a probabilistic analysis for a 2% chance of exceedance in 50 years is typically used for "critical" facilities such as schools and hospitals. The results of the probabilistic analyses are presented below in Table C.

TABLE C
PROBABILISTIC SEISMIC HAZARD ANALYSES
Scannell Properties
Parr Blvd. and Richmond Parkway
Richmond, California

	Statistical Return Period	Mean Moment Magnitude(1)	Peak Ground Acceleration (g)(1,2)
2% in 50 years	2,475 years	7.1	1.33 g
10% in 50 years	475 years	7.1	0.67 g

(1) USGS 2014 Interactive Deaggregation, https://earthquake.usgs.gov/hazards/interactive/, accessed April 13, 2018.
(2) Values determined using Vs^{30} = 270 m/s for Site Class "D" in accordance with the 2016 California Building Code.

The potential for strong seismic shaking at the project site is high. The Hayward Fault present the highest potential for severe ground shaking. 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 improvements should be designed in accordance with the provisions of the most recent edition (2016) of the California Building Code. Recommended seismic design criteria are presented in Section E of this report.*

4.4 Liquefaction Potential

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. This phenomenon can occur where there are saturated, loose, granular (sandy) deposits subjected to seismic shaking. Liquefaction-related phenomena can include potential settlement, flow failure, and lateral spreading.

Regional mapping (ABAG 2015) indicates that the site lies in a zone of “high” to “very high” liquefaction susceptibility as shown on Figure 5. Based on the results of previous and more recent exploration, subsurface conditions at the project site are dominated by high plasticity clayey soils that are not susceptible to liquefaction. There are a few discontinuous seams and lenses of dense granular materials (typically representative of historic stream channels) at depths below 50 feet which may have a low susceptibility to liquefaction.

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. Considering the thick cap of non-liquefiable soils and the relatively thin and localized sand layers, even if we assume liquefaction does occur in the deep sand layers, our analysis indicates the potential for damaging settlements to occur at the ground surface is low.

Based on the results of our subsurface exploration, laboratory testing, and engineering analyses, it is our professional opinion that the potential for damage to the proposed improvements due to liquefaction during a strong seismic event is low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.5 Seismically-Induced Ground Settlement

Ground shaking can induce settlement of loose granular soils above the water table. Subsurface exploration did not encounter any loose granular deposits above the water table. Therefore, the likelihood of damage to improvements at the site due to seismically-induced ground settlement is low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.6 Lurching, Lateral Spreading and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. Lurching and ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep channel banks. Lateral spreading generally occurs where liquefiable deposits flow towards a “free face”, such as channel banks, during an earthquake. These conditions do not exist at the project site. Therefore, the likelihood of damage to improvements due to lurching, lateral spreading and ground cracking is low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.7 Erosion and Scour

Sandy soils on moderate slopes and clayey soils on steep slopes are susceptible to erosion, particularly when subjected to concentrated water flow. Scour occurs where soil or rock is eroded by flowing water, such as in a stream channel or by wave action and can result in undermining of steep banks or structures such as retaining walls or bridge abutments.

The project site is relatively level and not adjacent to an existing stream channel, therefore not susceptible to significant erosion or scour. The risk of damage to new improvements due to erosion and scour is low.

Evaluation: *Less than significant.*
Mitigation: *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 that discharges water at appropriate locations. Re-establishment of vegetation on disturbed areas will also 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, Stormwater Best Management Practice Handbook.*

4.8 Seiche and Tsunami

Seiche and tsunami are short duration earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The project site is not mapped (ABAG 2015) as lying within a tsunami inundation zone. However, the map does indicate the tsunami inundation zone is located directly across Richmond Parkway, as shown on Figure 6.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.9 Flooding

The most significant adverse impact from flooding is water damage to structures. Site elevations range from +9 to +13 feet mean sea level. The project site is mapped (ABAG 2015) within a 500-

year flood zone and the northwest corner on Richmond Parkway is mapped within a 100-year flood zone (coastal) as shown on Figure 7. Given the site elevations, the risk of damage due to large-scale flooding at the site is low to moderate.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Finished floors for the planned structure should be constructed above the flood elevation or the lower portion of the building could be designed as a floor wall with flood doors. The project Civil Engineer determine the recommended minimum floor elevation and should design the storm drainage system for the maximum credible rainfall event. Careful consideration should be given to design of finished grades and site drainage to minimize the potential for damage due to localized flooding. Additional discussion of expected future settlements and recommendations for site grading and drainage are presented in Section V of this report.*

4.10 Expansive Soil

Moderate and highly plastic silts and clays, when located near the ground surface, can exhibit expansive characteristics (shrink-swell) that can be detrimental to structures and flatwork during periods of fluctuating soil moisture content. Boring logs from the subsurface exploration do indicate the presence of moderate to highly plastic, expansive near-surface soils. Excavation and fill placement is expected during site grading which will change the current conditions. The risk of damage due to expansive soils is moderate.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Building should be designed with foundations to account for expansive soil conditions. For deep foundation the grades beams should account for uplift pressure and pier depths check to confirm adequate uplift resistance. If shallow foundations are used they need to be designed for expansion, differential soil movement. Parking, driveways and sidewalks would need to be designed stronger than typical due to the expansive soil conditions. Alternatively, lime-cement treatment of the soils could be performed to reduce the expansion potential, improve stability and reduce designed pavement sections. Grading recommendation and foundation design criteria are provided in subsequent sections of this report.*

4.11 Settlement/Subsidence

Significant settlement can occur when new loads are placed at sites that are located over soft compressible clays, such as Bay Mud. The amount and rate of settlement is dependent on the magnitude of additional new loads (i.e. new structures and/or new fill), the thickness of compressible

material, and the inherent compressibility properties of the Bay Mud. The project site is underlain by a thick layer of moderately compressible clay.

Differential settlements are also possible due to variations in the thickness or properties of compressible Bay Mud, variations in new long-term loads (fill thickness or foundation loads) and variations in historic use of the land, i.e. old channels or low points through the site that may have required thicker fills, or previous "surcharges", such as old structures or fill mounds. Therefore, the risk of total and differential settlements at the site is moderate to high.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Settlement will occur if as new fill or building loads are applied to the existing ground surface. There are several methods by which post-construction settlements may be reduced or mitigated. Site grading, foundation system and utilities need to be designed to account for anticipated settlements. Recommendations for settlement mitigation and foundation design are presented in Section B and F of this report.*

4.12 Slope Instability/Landsliding

Weak soils and bedrock on moderate to steep slopes can move downslope due to gravity. Slope instability is often initiated or accelerated by soil saturation and groundwater pressure. Slope movement can vary from slow, shallow soil creep to large, sudden debris flows. Landslides can cause significant damage to structures and improvements. The project site is relatively level and planned fill placement is setback about 100 feet from the Petaluma River. Based on our investigation, slope instability is not considered a significant hazard at the site.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.13 Soil Corrosion

Corrosive soil and sea-water can damage buried metallic structures and underground utilities, deteriorate rebar reinforcement, and cause spalling of concrete. Laboratory corrosivity testing of the site soils is presented on Figure A-16. The soils at the project site are moderately corrosive due to low resistivity and high chloride content. Designers of site utilities and structural steel and concrete elements should account for corrosive environments. Considering the presence of brackish-water around the project site, we judge the hazard due to corrosion to be moderate to high.

Evaluation: *Less than significant with mitigation.*
Mitigation: *The project Civil and Structural Engineer should specify materials that are resistant to corrosive soil or provide cathodic corrosion protection. At a minimum, concrete for*

reinforced concrete structures should utilize Type II or Type V Portland cement with a water-cement ration of 0.45 or less and minimum compressive strength of 4000 psi. At least 3-inches of concrete coverage should be provided over reinforcing steel. Underground utilities should be constructed of plastic or PVC pipe; metallic piping should be avoided.

4.14 Radon-222 Gas

Radon-222 is a product of the radioactive decay of uranium-238 and raduim-226, which occur naturally in a variety of rock types, chiefly phosphatic shales, but also in other igneous, metamorphic, and sedimentary rocks. While low levels of radon gas are common, very high levels, which are typically caused by a combination of poor ventilation and high concentrations of uranium and radium in the underlying geologic materials, can be hazardous to human health. The project site is located in Contra Costa County, California, which is mapped in radon gas Zone 2 by the United States Environmental Protection Agency (USEPA, 2014). Zone 2 is classified by the EPA as exhibiting a “moderate” potential for Radon-222 gas with average predicted indoor screening levels between 2 and 4 pCi/L; therefore, the potential for hazardous levels of radon at the project site is low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.15 Volcanic Eruption

Several active volcanoes with the potential for future eruptions exist within northern California, including Mount Shasta, Lassen Peak, and Medicine Lake in extreme northern California, the Mono Lake-Long Valley Caldera complex in east-central California, and the Clear Lake Volcanic Field, located in Lake County approximately 70 miles north of the project site. The most recent volcanic eruption in northern California was at Lassen Peak in 1917, while the most recent eruption at the nearest volcanic center to the project site, the Clear Lake Volcanic Field, was about 10,000 years ago. All of northern California’s volcanic centers are currently listed under “normal” volcanic alert levels by the USGS California Volcano Observatory (USGS, 2018). While the aforementioned volcanic centers are considered “active” by the USGS, the likelihood of damage to the proposed improvements due to volcanic eruption is generally low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.16 Naturally Occurring Asbestos (NOA)

Naturally-occurring asbestos is commonly found in association with serpentinite and associated ultramafic rock types. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of the greater San Francisco Bay Area. The site is underlain by deep alluvial deposits. Therefore, the likelihood of naturally-occurring asbestos at the site is low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on our investigation and previous experience with similar sites and projects, we conclude that the planned projects are feasible from a geologic and geotechnical standpoint. The primary geotechnical issues to be considered during project design are appropriate seismic design, mitigating anticipated total and differential settlements, and providing uniform foundation support for the planned structures.

5.2 Historic and Anticipated Future Site Settlements

The project site is underlain by soft to medium stiff, moderately compressible clays that will consolidate with applications of surface loads resulting in settlement of the ground surface. The rate at which the settlement occurs depends on the thickness of the clay deposit, the distance to a drainage layer, and the vertical permeability of the clay. There are two modes of settlement: primary consolidation and secondary compression. Consolidation settlement often takes decades to complete. Secondary compression is generally a fraction of the primary settlement and occurs over a longer time.

Based on our review of historic aerial photographs, the project site was initially graded and used for farming prior to 1938. The western half of the project site was developed with several structures prior to 1987. The remainder was covered with structures (nursery greenhouses) by 1993. Most of the structures were removed in 2008. Based on the aerial photos and subsurface exploration there does not appear to have been any significant fill placement on the property. Based on laboratory testing data, the subsurface clay materials show they are slightly over-consolidated.

We evaluated the anticipated settlement of the site based on a generic building load of 600 psf. The Structural Engineer will provide more refined anticipated building loads and we will update our settlement evaluation using those loading conditions. However, with the generic building load, we used the program Settle 3D 3.0 by Rocscience and calculated the building would settle roughly 9-

to 12-inches with differential settlement of about 3- to 4-inches. These numbers do not account for placement of new fill to raise grades for an elevated finished floor.

Using a coefficient of vertical consolidation of the Bay Mud of 19 ft^2/year and drainage conditions between single and double, it would take about 100 years to achieve about 95 percent of the primary consolidation of the underlying clay.

We understand that moderate total or differential settlements are not acceptable for the project and therefore a deep foundation system that reduces future anticipated settlements is the likely foundation system. However, future total or differential settlement could be reduced by surcharging the site and installing vertical wick drains to consolidate the underlying clay an accelerated rate and allow for site grades to be raised while minimizing expected post-construction settlements. The rate of settlement is a function of the horizontal permeability of the clay and the distance between vertical wick drains. For wick spacing on the order of 6-feet on center, estimated time to achieve 95% consolidation is about a year. Another option to reduce settlements is to counteract the weight of the building by excavating existing soil and replacing with lightweight fill (i.e. geofoam blocks). More detailed analyses would be needed if one of these options will be pursued.

5.3 Site Preparation and Grading

Site grading will be required to create a level building pad for the new structures and for the associated site improvements. Site preparation and grading should conform to the following recommendations.

5.3.1 Surface Preparation

Clear all grass, brush, roots, and other organic matter from areas where improvements are planned. Existing structures, foundations and old pavements and any construction debris or abandoned utilities encountered should be removed from the site. Utilities may be abandoned in-place provided neat cement grout completely fills all voids in the conduit. Vegetation scrapings should be stockpiled for re-use in landscape areas or removed from the site.

5.3.2 Excavations

Subsurface conditions at the site generally consist of medium stiff, sandy clay and soft to medium stiff clay. We anticipate excavations can be reasonably completed with “traditional” equipment such as backhoes and dozers. Excavations having a depth of 5-feet or more must be excavated and shored in accordance with Cal-OSHA regulations. We recommend that the project Contractor be responsible for site safety, including trench shoring and de-watering. Pursuant to Cal-OSHA classifications, the onsite fill soils are “Type C” and may be prone to “squeezing” and raveling in open excavations. Additionally, groundwater should

be anticipated in excavations deeper than 3 to 5-feet, and the Contractor should anticipate the need for adequate de-watering and shoring in all excavations deeper than 5 feet. Many shoring systems are available, and the Contractor should select an appropriate system that allows for efficient installation to prevent collapse. We recommend that de-watering be accomplished by use of submersible pumps.

5.3.3 Fill Materials

Select soil and rock mixtures generated from on-site excavations may be suitable for re-use as fill provided they can be processed to meet the specifications presented below. Whether imported or derived of onsite materials, all fill material should 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 20, and (3) have a maximum particle size of 4 inches. Any imported fill material needs to be tested to verify its suitability for use as fill material prior to placement.

5.3.4 Fill Placement and Compaction

Prior to fill placement, all subgrades should be scarified a minimum of 8-inches deep, moisture-conditioned slightly above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. New fill shall be placed in layers not exceeding 8-inches and compacted to at least 90 percent relative compaction. The compacted subgrade and fill should be firm and unyielding when proof rolled with heavy compaction equipment.

Within pavement areas, relative compaction should be increased to 95 percent minimum. These areas should also produce a smooth, firm, and unyielding surface when proof-rolled with heavy construction equipment such as loaded water trucks or scrapers. 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.”

5.4 Seismic Design

Minimum mitigation of ground shaking includes seismic design of the structures in conformance with the provisions of the most recent version (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 to the Hayward, San Andreas, San Gregorio and Rodgers Creek Faults, we recommend the CBC coefficients and site values shown in Table D below for use in equations 16-37[(1)] and 16-38 to calculate the design base shear of the new construction. To determine site seismic coefficients, we used the USGS Seismic Design Maps web application, using the latitude and longitude shown on Figure 4.

TABLE D
2016 CBC FACTORS
Scannell Properties
Parr Blvd. and Richmond Parkway
Richmond, California

Factor Name	Coefficient	Site Specific Value
Site Class[1]	$S_{A,B,C,D,E, or F}$	S_D
Site Coefficient	F_a	1.0
Site Coefficient	F_v	1.5
Spectral Acc. (short)	S_s	2.093 g
Spectral Acc. (1-sec)	S_1	0.859 g

(1) Soil Profile Type S_D Description: Stiff soil profile with shear wave velocity between 600 and 1,200 feet per second, Standard Penetration Test N value between 15 and 50, and Undrained Shear Strength between 1,000 and 2,000 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 building damage may still occur during strong ground shaking.

5.5 Foundation Design

Considering the planned grading, subsurface conditions, building loads and potential settlements, deep foundations bearing in the medium stiff clays are recommended. However, if some settlements are acceptable, or if other options to reduce settlements are implemented, then a rigid shallow foundation could be utilized as discussed below.

Shallow Foundations

Where some future total and differential settlements are acceptable, we recommend a rigid shallow foundation system. Suitable shallow foundation systems include a thick, heavily-reinforced mat slab, a “waffle” slab (consisting of continuous, interconnected footings), or a post-tensioned slab. These types of systems should be designed to bridge over 20 ft. diameter areas on non-uniform support to minimize the effects of post-construction differential settlements. Design criteria for shallow foundation systems are presented below in Table E.

TABLE E
SHALLOW FOUNDATION DESIGN CRITERIA
Scannell Properties
Parr Blvd. and Richmond Parkway
Richmond, California

Shallow Continuous Spread Footings	
Minimum width:[1]	12 inches
Minimum depth:[2]	24 inches
Allowable bearing capacity:[3,4]	2,000 psf
Base friction coefficient:	0.30
Lateral passive resistance:[5]	300 pcf
Rigid Mat or Post-Tensioned Slab:	
Modulus of subgrade reaction, k_s	100 pci
Minimum thickness at edge of slab:[6]	18 inches
Maximum unsupported interior span:[7]	20 feet
Maximum unsupported edge (corner) cantilever:[7]	10 feet
Edge moisture variation (e_m) – Center Lift	20 feet
Edge moisture variation (e_m) – Edge Lift	10 feet
Differential soil movement (y_m) – Center Lift	1.5 inch
Differential soil movement (y_m) – Edge Lift	1.5 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) Foundation to bear on compacted fill, placed and compacted in accordance with the recommendations presented in Section V (D) of this report.
(5) Equivalent fluid pressure. Ignore upper 6-inches unless confined by asphalt or concrete.
(6) Actual thickness, load distribution, and unsupported spans must be determined by Structural Engineer to reduce deformations to acceptable levels.
(7) Assumes rigid slab behavior with idealized fixed end conditions.

Deep Foundations

For the heavier structures, a deep foundation system using skin friction in the deeper soils are recommended. Suitable deep foundation options at the site could include driven piles, auger-cast piers or torque-down piles. Traditional drilled piers could be used but would need to be cased or slurry supported due to the high groundwater conditions and possible “squeezing” soils. With adequately embedded foundations into the firm alluvial soils, building settlements should be small.

However, the actual amount of anticipated settlement will be determined once more detailed information about the loading conditions is provided by the Structural Engineer.

Some differential settlement between the building and exterior grade should be expected due to settlement associated with grading required to achieve the exterior finished grades. More detailed discussion of the deep foundation options is presented below.

In areas where future settlement is expected, deep foundations will experience “down drag” effects as the soil settles inducing negative skin friction on the pile. Therefore, deep foundations will need to be designed for downdrag and structural loads.

Driven Piles

Driven piles are precast steel or concrete piles driven with a large pile hammer until a suitable driving resistance and bearing capacity is achieved. The depth to achieve full pile capacity could vary across the building site. Pre-cast piles can be costly to extend or cut-off if needed. Driven piles will also cause significant noise and vibrations.

Auger Cast Piles

Auger Cast Piles (ACP) are installed by rotating a continuously flight hollow shaft auger in to displace the soil to a specified depth. High strength cement grout is pumped under pressure through the hollow shaft as the auger is slowly withdrawn. Reinforcing is installed while the cement grout is still fluid, or in the case of full length single reinforcing bars, through the hollow shaft of the auger prior to the withdrawal and grouting process. The resulting reinforced grout column hardens and forms an auger cast pile.

Torque Down Piles (TDP)

Torque down piles (TDP) are full displacement, concrete-filled steel pipe piles that consist of a large diameter steel shaft with a tapered closed ended conical tip with a helix to aid in installation. TDP are drilled into the ground to depth and the steel shaft displaces the surrounding soil as it advances. Once the design depth is achieved steel reinforcement and concrete fills the steel pipe pile. TDP achieves vertical capacity through both skin friction between the soil and the steel pipe and the end bearing of the closed end tip. Since the TDP is a displacement pile no soil spoils are generated during construction. Additionally, TDP are drilled into the ground, not driven, therefore the construction process is relatively quiet and excess vibrations are not generated.

Vertical capacities depend on the driven depth, diameter and structural capacity of the piles. Seventy (70) kip design loads are fairly typical and much higher design loads can be achieved with deeper and larger diameter piles. Preliminary vertical capacity versus depth for deep foundation is presented on Figure 8. We should coordinate with the Project Structural Engineer to develop specific deep foundations criteria for the preferred deep foundation type. Load testing should be

performed on at least one pile to confirm the anticipated subsurface conditions and verify the design capacity has been achieved.

For preliminary lateral pile analyses, the soil parameters displayed in Table F can be used:

TABLE F
LPILE Soil Input Parameters
Scannell Properties
Parr Blvd. and Richmond Parkway
Richmond, California

Material	Eff. Unit Wt.	Eff. Friction	c	k	ε_{50}
Clay, upper 30 feet	105 pcf	N/A	4.0 psi	80 pci	0.01
Clay, below 30 feet	115 pcf	N/A	7.0 psi	100 pci	0.005

Ground Improvement with Shallow Foundations

Another option for foundations and settlement mitigation is to couple subsurface improvements that increase soil bearing capacity with a shallow foundation system. Drill Displacement Columns (DDC) involve drilling to design depths and injecting Controlled Low Strength Material (CLSM) under pressure, which creates large diameter, well defined, compaction columns and effectively increases soil strength. DDC does not structurally connect to the foundation and therefore requires a subsequent shallow foundation. Other ground-improvement options include deep soil-cement mixing (DSCM), whereby Portland cement is mixed continuously with soil to create overlapping columns of increased strength, or implementation of “geopiers” which are essentially stone columns of dense aggregate created by “vibro-replacement”. These options reinforce soft native soils to increase surficial bearing capacities and reduce settlements.

Another option to reduce total and differential settlement to acceptable levels is to offset the weight of the structures by over-excavating and removing an equivalent weight of soil by creating a deepened crawl space or basement. Alternatively, the building footprint could be over-excavated and backfilled with a low-density material such as Geofoam or lava rock. The depth of the required over-excavation would be dependent on the building weight and amount of soil removal needed to create a net “zero” load after the structure has been constructed.

Over-excavating and replacing with lightweight fill would reduce the total and differential settlement but would not completely eliminate it. The columns and perimeter walls will create zones of higher pressure compared to the open interior areas. This differential loading can cause some differential settlement. The amount of settlement depends on the rigidity of the foundation and ability to spread and distribute the loads.

5.6 Capillary Break and Moisture Barrier

We recommend that concrete slabs have a minimum thickness of five inches and be reinforced with steel reinforcing bars (not welded wire mesh). To improve interior (crawl space) moisture conditions, a 6-inch layer of clean, free draining, 3/4-inch angular gravel should be placed beneath the interior concrete slab to form a capillary moisture break. The 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 and meeting the requirements of ASTM E-1745 Class A, should be placed over the rock layer 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 slab resulting in mold growth or other adverse conditions.

It should be noted that where the gravel capillary break layer is placed beneath floor slabs (especially a below-grade mat slab), there is a chance that water will collect in the gravel layer and become trapped. If this condition occurs, the potential for moisture problems at the surface of the slab will be increased. One method of minimizing the potential for this to occur would be to construct a subdrain trench through and just below the gravel layer so that water collected in this area can escape. The subdrain should extend at least 12 inches below the base of the slab and 6 inches below the bottom of the gravel layer, and would consist of a four-inch diameter, perforated pipe (Schedule 40 PVC) surrounded by gravel. The subdrain would connect to the gravel layer beneath the slab, and the pipe should lead (at a minimum one-percent slope) to a storm drain or another suitable outlet point. The outlet pipe should transition to nonperforated pipe at a point two-feet inside the perimeter footing of the structure. A compacted clayey soil plug or other type of moisture barrier should be placed around the pipe at the point where the outlet pipe leaves the building footprint to prevent seepage from back-flowing into the underslab gravel layer.

The industry standard approach to floor slab moisture control, as discussed above, does not assure that floor slab moisture transmission rates will meet the building use requirements or that indoor humidity levels will be low enough to inhibit mold growth. Building design, construction, and intended use have a significant role in moisture problems and should be carefully evaluated by the owner, designer, and builder in order to meet the project requirements.

5.7 Site Drainage Considerations

Future differential settlements may result in uneven grades and generally poor site surface drainage. Careful consideration should be given to design of new finished grades at the site to ensure positive drainage. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at 5 percent for a distance of at least 5 feet 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. The tight pipe system should discharge at an appropriate location unlikely to result in adverse erosion, preferably into an established municipal storm drain system.

5.8 Underground Utilities

The project site is likely to experience some future settlements and underground utilities may be prone to breakage, particularly during a seismic event. In order to reduce the likelihood of damage, flexible utility conduits and connections should be utilized and fitted with both automatic and manual emergency shutoff valves at key connection points and at service tie-ins. Bedding materials for utility pipes should be non-corrosive sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 15 percent finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6-inches. Utility excavations should be backfilled with select fill meeting the criteria described in Site Grading and compacted to a minimum of 90 percent relative compaction. In pavement areas, relative compaction should be increased to a minimum of 95 percent in the upper 12 inches.

5.9 Exterior Concrete Slabs-on-Grade

Exterior reinforced concrete mat slabs, such as concrete sidewalks or patios, should be a minimum of 4 inches thick and reinforced with steel rebar. The exterior slabs should be constructed on 4 inches or more of Caltrans Class 2 Aggregate Base compacted to at least 92 percent relative compaction. The soil subgrade should be prepared as described in Site Grading and compacted to at least 92 percent relative compaction.

5.10 Asphalt Concrete Pavements

We have calculated pavement sections in accordance with Caltrans procedures for flexible pavement design (2017) using an assumed R-value of 10. We have provided a range of Traffic Indices (TI) from 4, 5, 6, and 10 depending on the expected traffic loads for a twenty-year design life. In general, areas expected to experience loading from heavy vehicles (such as fire lanes, loading dock access roads, trash enclosures, etc.) should be designed using the higher Traffic Index, while parking areas and other lightly-loaded areas can utilize a thinner pavement section based on the lower Traffic Index. Anticipated ESALs for locations expected to experience loading from frequented heavy vehicles were provided by the client and calculated to have a TI of about 9.0; however, Table 613.3B states that the TI should not be less than 10 for a 20-year pavement design life. Therefore, the recommended pavement sections are presented in Table G.

TABLE G
PAVEMENT DESIGN CRITERIA
Scannell Properties
Parr Blvd. and Richmond Parkway
Richmond, California

	T.I.	Asphalt Concrete	Aggregate Base	Untreated Subgrade
Light passenger vehicles/parking	4	2.5 inches	7 inches	95% R.C.
Light truck traffic drive aisles	5	3.0 inches	9 inches	95% R.C.
Moderate truck traffic drive aisles	6	3.5 inches	12 inches	95% R.C.
Heavy truck traffic and fire lanes	7	4.0 inches	15 inches	95% R.C.
Frequented heavy truck traffic w/ 20-yr design life	10	6.0 inches	22 inches	95% R.C.

The subgrade soil could be treated lime-cement to improve the strength and bearing capacity. If the upper 12 inches of the subgrade is lime cement treated the revised pavement sections are listed below.

	T.I.	Asphalt Concrete	Aggregate Base	Lime-Cement Subgrade
Light truck traffic drive aisles	5	2.5 inches	4 inches	95% R.C.
Moderate truck traffic drive aisles	6	3.0 inches	5 inches	95% R.C.
Heavy truck traffic and fire lanes	7	3.5 inches	6 inches	95% R.C.
Frequented heavy truck traffic w/ 20-yr design life	10	5.5 inches	12 inches	95% R.C.

Subgrade preparation for asphalt-paved areas should be performed in accordance with the recommendations shown in Section V (D) of this report. The base rock should consist of compacted Class 2 Aggregate Base (Caltrans, 2017), 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.

6.0 SUPPLEMENTAL SERVICES

Following review and consideration of this report, we should consult with you regarding the "preferred" foundation type for new structures within the residential development area. We will also be available to provide consultation throughout the design process on other geotechnical items.

As project plans near completion, we should review them to ensure that the intent of our

recommendations has been sufficiently incorporated and provide a Geotechnical Plan Review letter to the City of Richmond and/or Contra Costa County, if/as required. During construction, we should be present intermittently to observe and test the geotechnical portions of the work, for the purposes of verifying that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to ensure that the Contractor's work is performed in accordance with the project plans and specifications.

7.0 LIST OF REFERENCES

American Society of Civil Engineers (ASCE) (2010), "Minimum Design Loads for Buildings and Other Structures," (ASCE 7-10).

American Society for Testing and Materials (ASTM) (2009), "2009 Annual book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics," ASTM, Philadelphia.

Association of Bay Area Governments (ABAG) (2015), Liquefaction Susceptibility Maps, FEMA Flood Maps, and Tsunami Inundation Maps, http://gis.abag.ca.gov/website/hazards/, accessed June 10, 2015.

California Building Code (2016), California Building Standards Commission/International Conference of Building Officials, Whittier, California.

California Department of Transportation (2017), Highway Design Manual, Sacramento, California.

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

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.

California Department of Transportation (2018), "Caltrans ARS Online, V2.3.09" (web-based acceleration response spectra calculator tool), http://dap3.dot.ca.gov/ARS_Online/, accessed April 13, 2018.

Google Earth (2018), Historical Photos, accessed April 13, 2018. (ver. 7.3.1.4507).

National Oceanic and Atmospheric Administration (NOAA) (2013), National Geophysical Data Center/(NGDC/WDS) Global Historical Tsunami Database, Boulder, Colorado, USA, http://www.ngdc.noaa.gov/hazard/tsu_db.shtml, accessed April 13, 2018.

United States Environmental Protection Agency (USEPA) (2014), EPA Map of Radon Zones in California, https://www.epa.gov/sites/production/files/2014-08/documents/california.pdf, accessed April 13, 2018.

United States Geological Survey (2003), "Summary of Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2032," The 2003 Working Group on California Earthquake Probabilities, 2003.

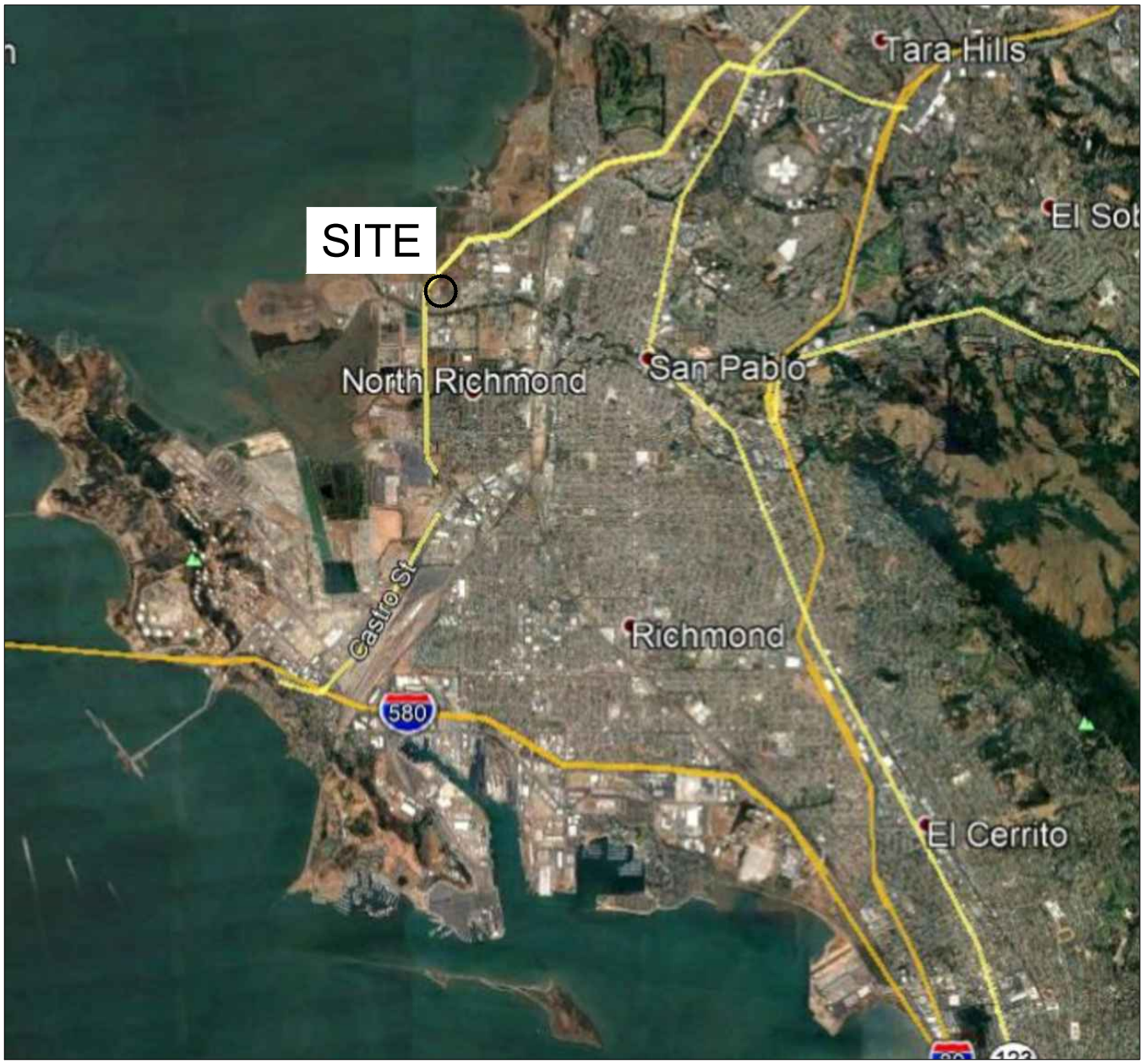
United States Geological Survey (2008), "The Uniform California Earthquake Rupture Forecast, Version 2," The 2007 Working Group on California Earthquake Probabilities, Open File Report 2007-1437, 2008.

United States Geological Survey (2014), "2014 Interactive Deaggregations", https://earthquake.usgs.gov/hazards/interactive/index.php, accessed April 13, 2018.

United States Geological Survey (2016), 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).

United States Geological Survey (2018), Earthquake Hazards Program, Earthquake Map, https://earthquake.usgs.gov/earthquakes/, accessed April 13, 2018.

United States Geological Survey (2018b), Volcano Hazards Program, California Volcano Observatory, http://volcanoes.usgs.gov/observatories/calvo/calvo_activity_update_last10.html, accessed April 13, 2018.



SITE: LATITUDE, 37.9686°
LONGITUDE, -122.3726°

SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth 2018

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

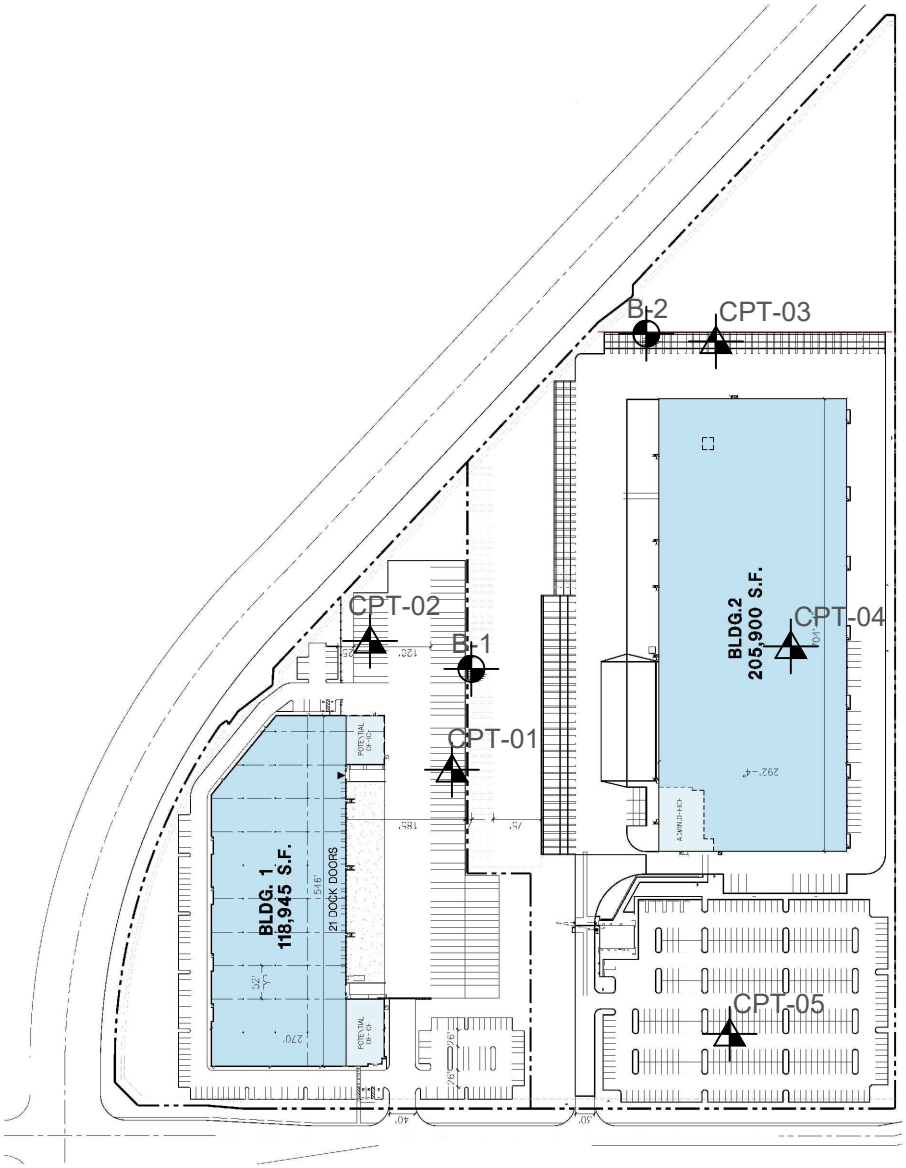
SITE LOCATION MAP

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

1
FIGURE



SITE PLAN

1"=300'

MPEG Boring Location, May 2015

Blackburn Consulting CPT Location, February 2004

SCALE

0 150 300 600 FEET

NORTH

REFERENCE: Site Plan Provided by HPA Architects, March 22, 2018.

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

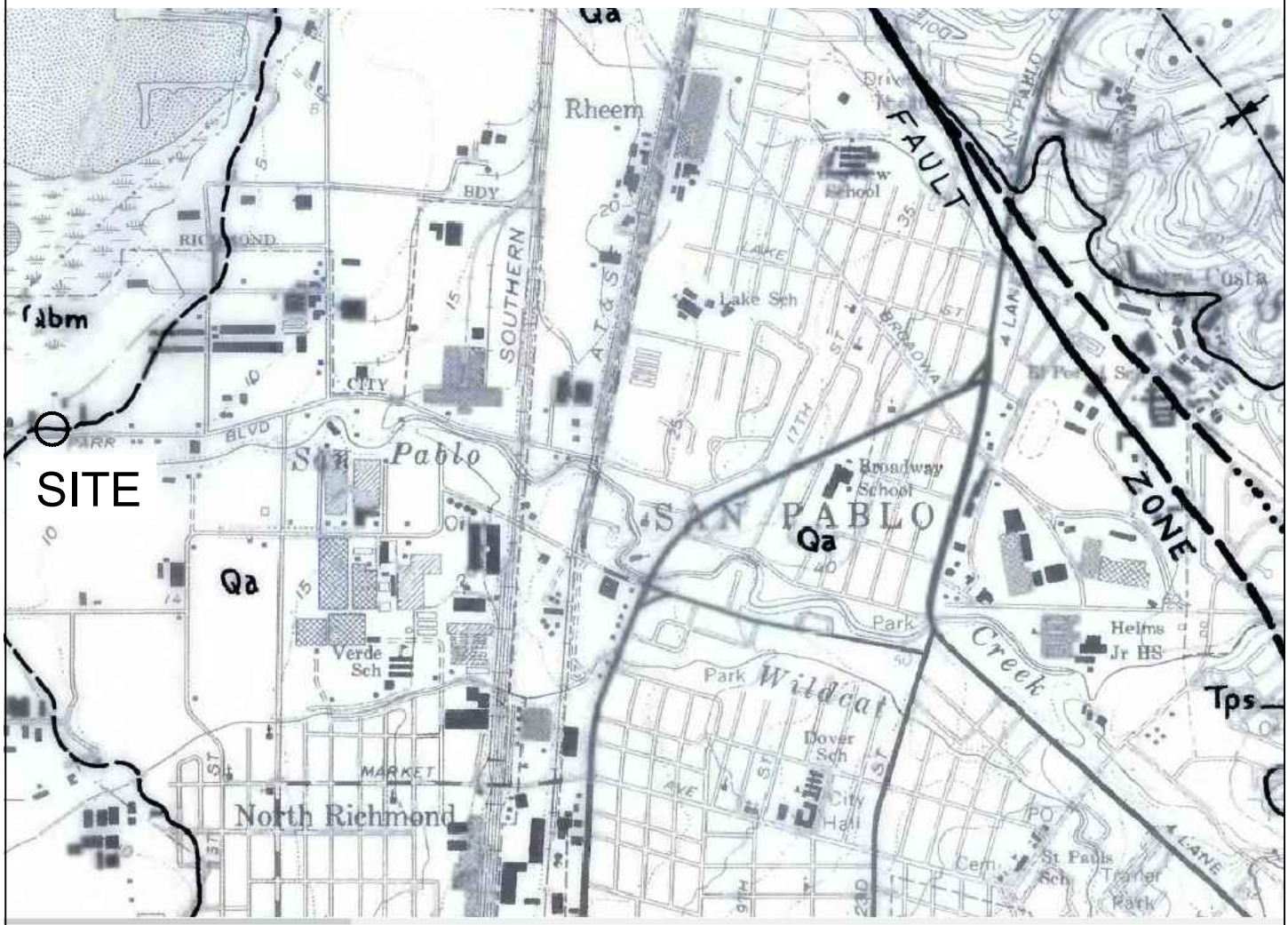
SITE PLAN

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

2
FIGURE



REGIONAL GEOLOGIC MAP

N.T.S

LEGEND:

Qbm	Bay Mud - Highly compressible, highly expansive marine clays and silts
Qa	Alluvium - Poorly to moderately sorted sands, gravels, silts, and clay
— — —	Geologic Contact, Dashed where appropriate
━━━	Fault

NORTH

REFERENCE: Dibblee, Thomas W. (1980), "Preliminary Geologic Map of the Richmon Quadrangle, Alameda and Contra Costa Countles, California." Department of the Interior, USGS, Open File Report 80-1100, Scale 1:24,000.

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

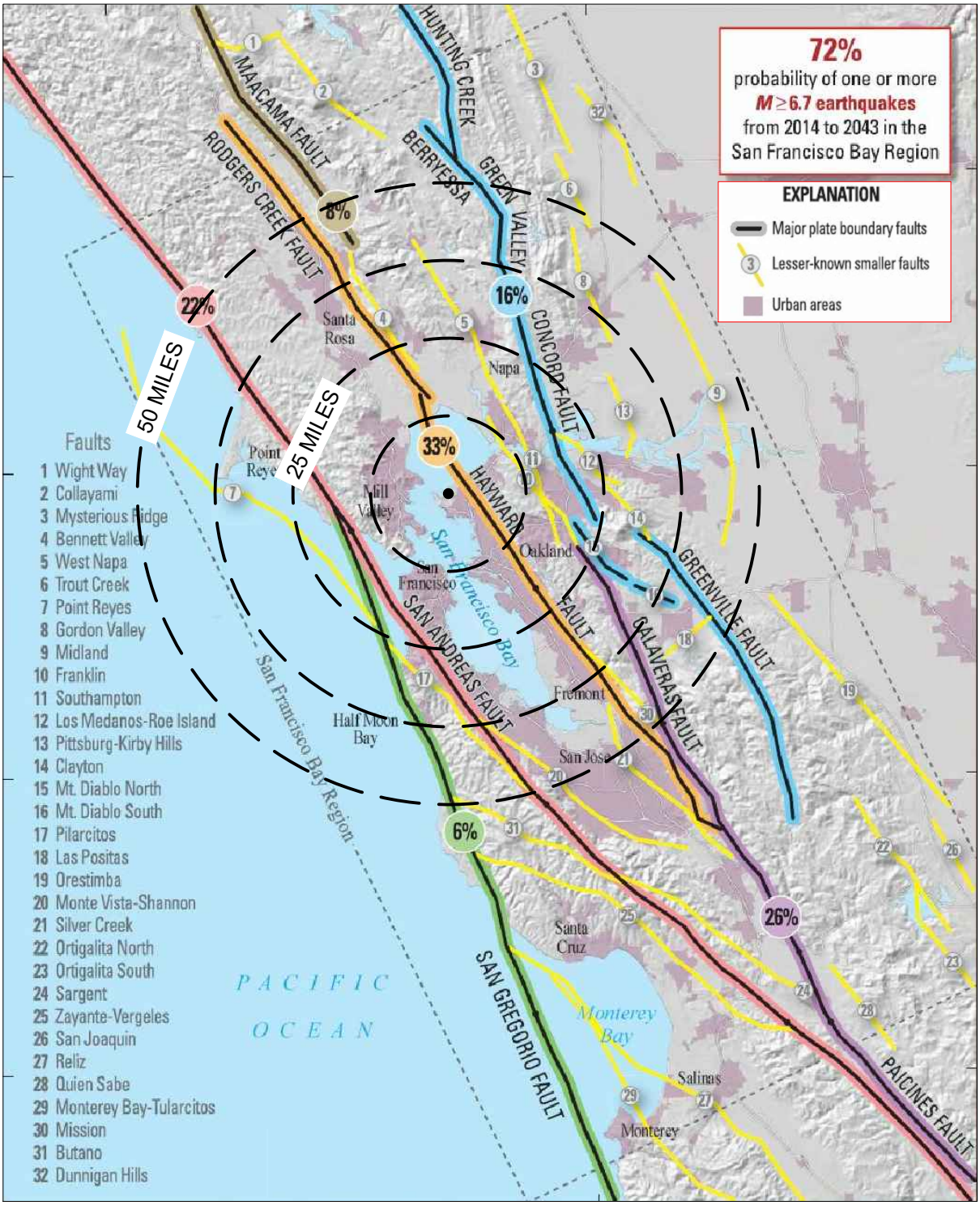
REGIONAL GEOLOGIC MAP

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

3
FIGURE



SCALE

0 12.5 25 50 MILES

NORTH

REFERENCE: 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).

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

ACTIVE FAULT MAP

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

4
FIGURE

N.T.S

SUSCEPTIBILITY LEVEL:

- Very High
- High
- Moderate
- Low
- Very Low
- Major Road
- Local Road

NORTH

REFERENCE: ABAG Geographic Information System.

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

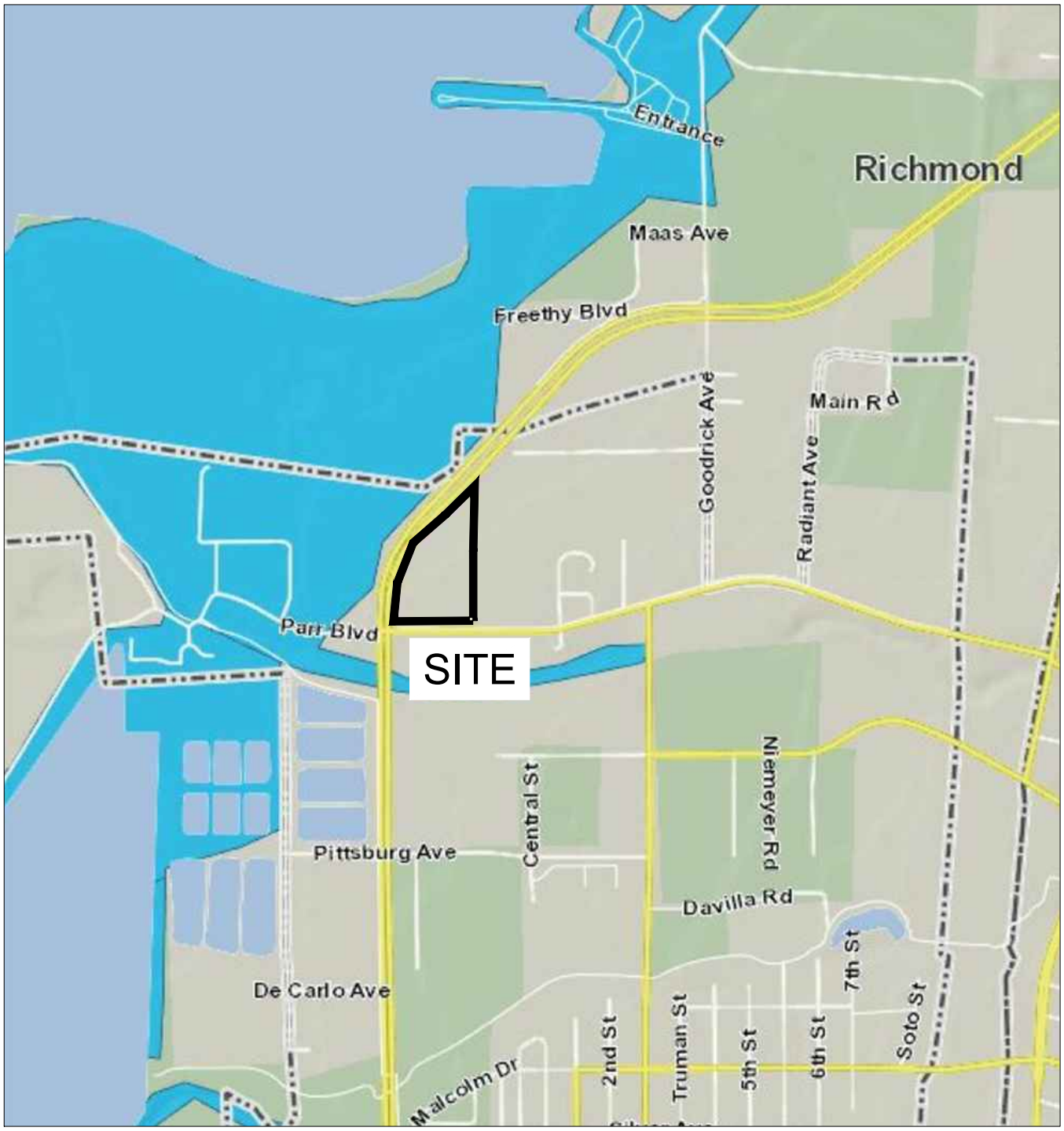
LIQUEFACTION SUSCEPTIBILITY MAP

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

5
FIGURE



N.T.S

TSUNAMI INUNDATION:

- Tsunami Inundation Area
- Urbanized Area

NORTH

REFERENCE: ABAG Geographic Information System.

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

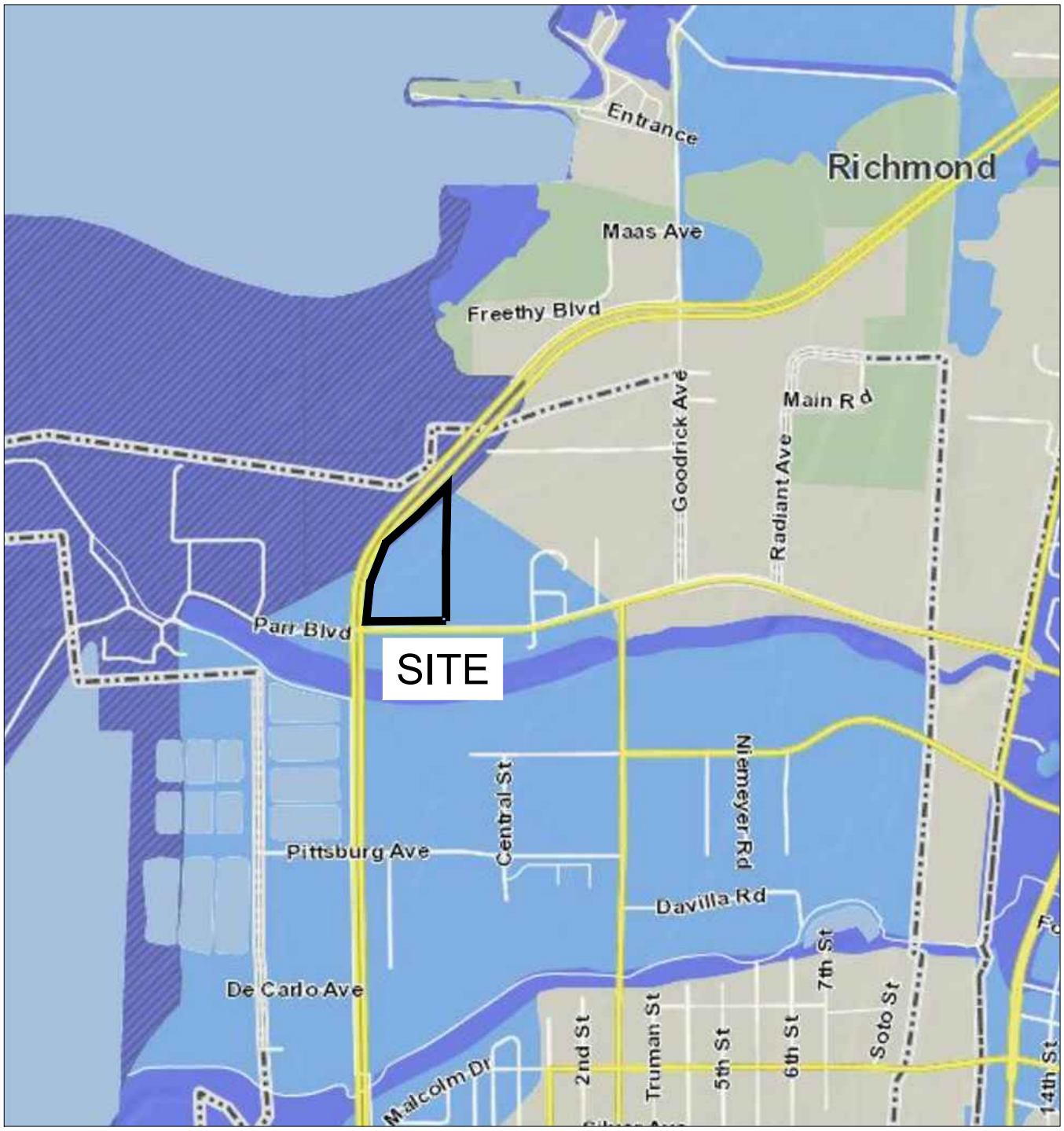
TSUNAMI INUNDATION MAP

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

6
FIGURE



N.T.S

FLOOD HAZARD AREA:

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

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

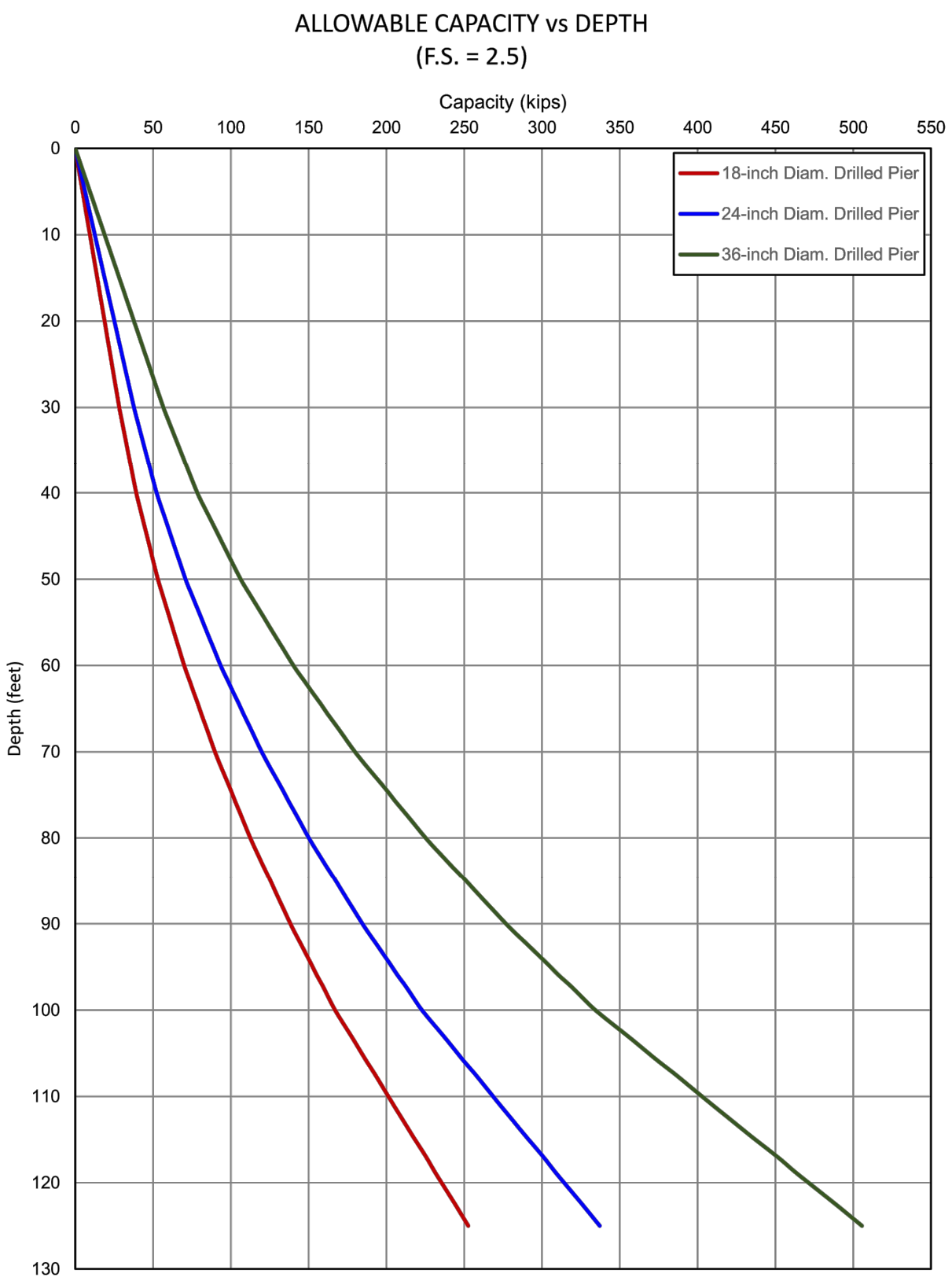
FEMA DIGITAL FLOOD INSURANCE RATE MAP (DFIRM)

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

7
FIGURE



MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 2042.002 Figures.dwg

ALLOWABLE PILE CAPACITY VERSUS DEPTH

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 4/13/2018

Drawn PDC
Checked SAS

8
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

1.0 Subsurface Exploration

Exploratory Borings

We explored subsurface conditions at the site by drilling two exploratory borings utilizing a truck mounted CME 75 drilling rig. Boring 1 was excavated using a rotary wash system and Boring 2 was excavated with 4-inch solid flight augers. Both borings were drilled on May 29, 2015. The approximate boring locations are shown on Figure 2. The borings were drilled to a maximum depth of 76.5- and 21.5-feet, respectively, below the ground surface.

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. The boring logs are presented on Figures A-2 through A-6.

We obtained “undisturbed” samples using a 3-inch diameter, split-barrel modified California sampler with 2.5 by 6-inch brass tube liners, with a 2-inch diameter, split-barrel Standard Penetration Test (SPT) sampler or with a 2.5 by 18-inch brass piston tube. The sampler was driven with a 140-pound hammer falling 30 inches, and the tubes were pushed into the soil with a hydraulic pressure system. The number of blows required to drive the samplers or push the piston 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving or as the pressure needed to push the piston tube. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

Cone Penetration Tests (CPTs)

CPTs provide a continuous profile of data throughout the depth of exploration. It is particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential. Five CPT’s were performed as part of a previous investigation. The CPT is a cylindrical probe, 35 mm in diameter, which is pushed into the ground at a constant rate of 2 cm/sec. The device is illustrated on Figure A-7. It is instrumented to obtain continuous measurements of cone bearing (tip resistance) and sleeve friction. Electronic signals from the instrument are continuously recorded by an on-board computer at the surface.

The recorded data is analyzed to interpret soil type, the cone bearing alone indicates soil density or strength, and the pore pressure indicates the presence of clay. Variations in the data profile indicate changes in stratigraphy. This test method has been standardized and is described in detail by the ASTM Standard Test Method D3441 "Deep, Quasi-Static Cone and Friction Cone Penetration Tests of Soil." The interpretation of CPT data is illustrated on Figures A-8 through A-13.

It should be noted that the boring and CPT logs description of soils encountered reflect conditions only at the location of the exploration at the time they were advanced. 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.

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;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;
- Atterberg Limits, ASTM D 4318;
- pH in soil, EPA 9040; Resistivity in Soil, SM 2510; and Anions in soil (sulfate and chloride), EPA 300.
- Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils; ASTM D2850 - 03a (2007)
- One-Dimensional Consolidation Properties of Soils Using Incremental Loading, ASTM D2435 / D2435M

The moisture content, dry density, triaxial and unconfined compressive strength determinations are shown on the exploratory boring logs. Results of Atterberg Limits testing are presented on Figure A-14. Consolidation test results are presented on Figures A-15 and A-16. Results from corrosion testing is shown Figure A-17.

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

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

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 BL.dwg

SOIL CLASSIFICATION CHART

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 6/5/15

Drawn NGK
Checked

A-1
FIGURE

BORING 1

EQUIPMENT: Truck Mounted CME 75 using Rotary Wash Drilling
DATE: 5/29/15
ELEVATION: 10 - feet*
*REFERENCE: Google Earth, 2015

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			Sandy CLAY (CL) Dark gray, dry to moist, soft to medium stiff, low to medium plasticity clay, ~25-30% fine sand. [Alluvium]
						1				
			500 psi				5			Piston sampler used from 5.0 to 5.5 feet, unable push sample, no sample collected.
	PI=39 LL=61	900 UC	7	29.3	93	2				CLAY (CH) Dark gray, moist, soft to medium stiff, medium to high plasticity clay. [Alluvium]
						3	10			
						4				
							15			
		600 TXUU	600 psi	46.1	74	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)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 BL.dwg

BORING LOG

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 6/5/15

Drawn NGK
Checked

A-2
FIGURE

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)	BORING 1 (CONTINUED)
						20			CLAY (CL/CH) Dark gray, moist, soft to medium stiff, medium to high plasticity clay. [Alluvium]
						7			
						25			Grades stiff.
		700 UC	1000 psi	75.3	57	8			
						9			
						30			
						10			
						35			
		1200 TXUU	1500 psi	37.6	82.2	11			
						12			
						40			

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

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 BL.dwg

BORING LOG

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 6/5/15

Drawn NGK
Checked

A-3
FIGURE

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)	BORING 1 (CONTINUED)
						40			CLAY (CL/CH) Dark gray, moist, stiff, medium to high plasticity clay. [Alluvium]
						13			
						45			
		1075 UC	13	38.2	85	14	■		
						15			
						50			
						16			
						55			
		1450 UC	14	35.8	86	17	■		
						18			
						60			

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

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 BL.dwg

BORING LOG

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 6/5/15

Drawn NGK
Checked

A-4
FIGURE

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)	BORING 1 (CONTINUED)
						60			CLAY (CL/CH) Dark gray, moist, stiff, medium to high plasticity clay. [Alluvium]
						19			
						65			
		500 UC	18	39.0	83	20	■		
						21			
						70			
						22			
						75			Grades stiff to very stiff.
		650 UC	25	38.5	83	23	■		
									Bottom of boring at 76.5 feet. No groundwater encountered during drilling.
						24			
						80			

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

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 BL.dwg

BORING LOG

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 6/5/15

Drawn NGK
Checked

A-5
FIGURE

BORING 2

EQUIPMENT: Truck Mounted CME 75 with 4-inch solid flight augers
DATE: 5/29/15
ELEVATION: 9 - feet*
*REFERENCE: Google Earth, 2015

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			Sandy CLAY (CL) Dark gray, dry to moist, soft to medium stiff, low to medium plasticity clay, ~25-30% fine sand. [Alluvium]
						1				CLAY (CH) Dark gray, moist, soft to medium stiff, medium to high plasticity clay. [Alluvium]
		750 UC	9	33.1	90		5	■		
						2				
						3	10	■		Trace gravel.
		775 UC	10	29.9	93					
						4				
							15	■		Sandy CLAY (CL/CH) Dark gray, moist, very soft to soft, medium plasticity clay, ~20-25% fine sand. [Alluvium]
		450 UC	3	52.6	72	5				
		475 UC	6	76.2	56	6	20	■		Bottom of boring at 21.5 feet. No groundwater observed during 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

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 BL.dwg

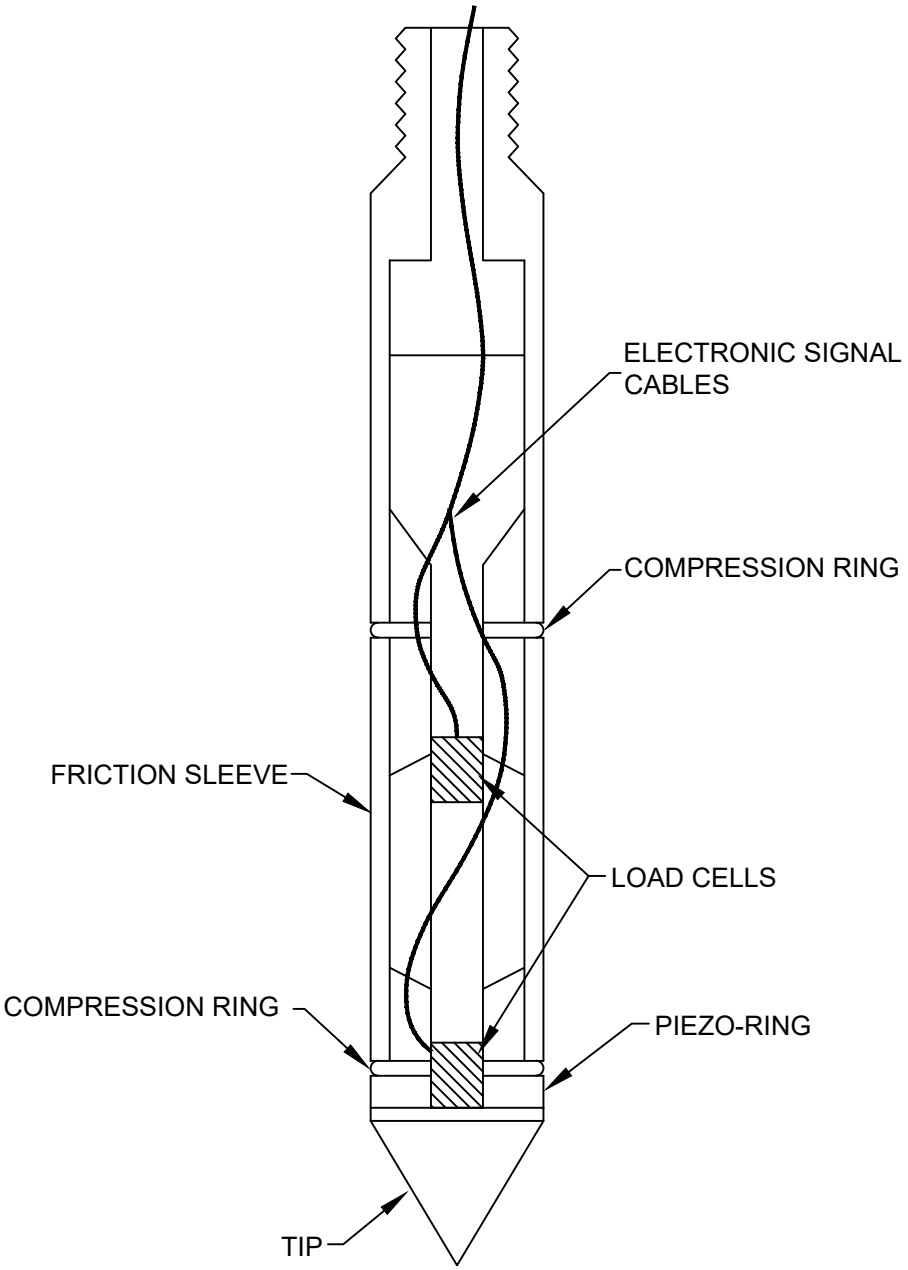
BORING LOG

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 6/5/15

Drawn NGK
Checked

A-6
FIGURE



CONE PENETROMETER

(NO SCALE)

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 CPT.dwg

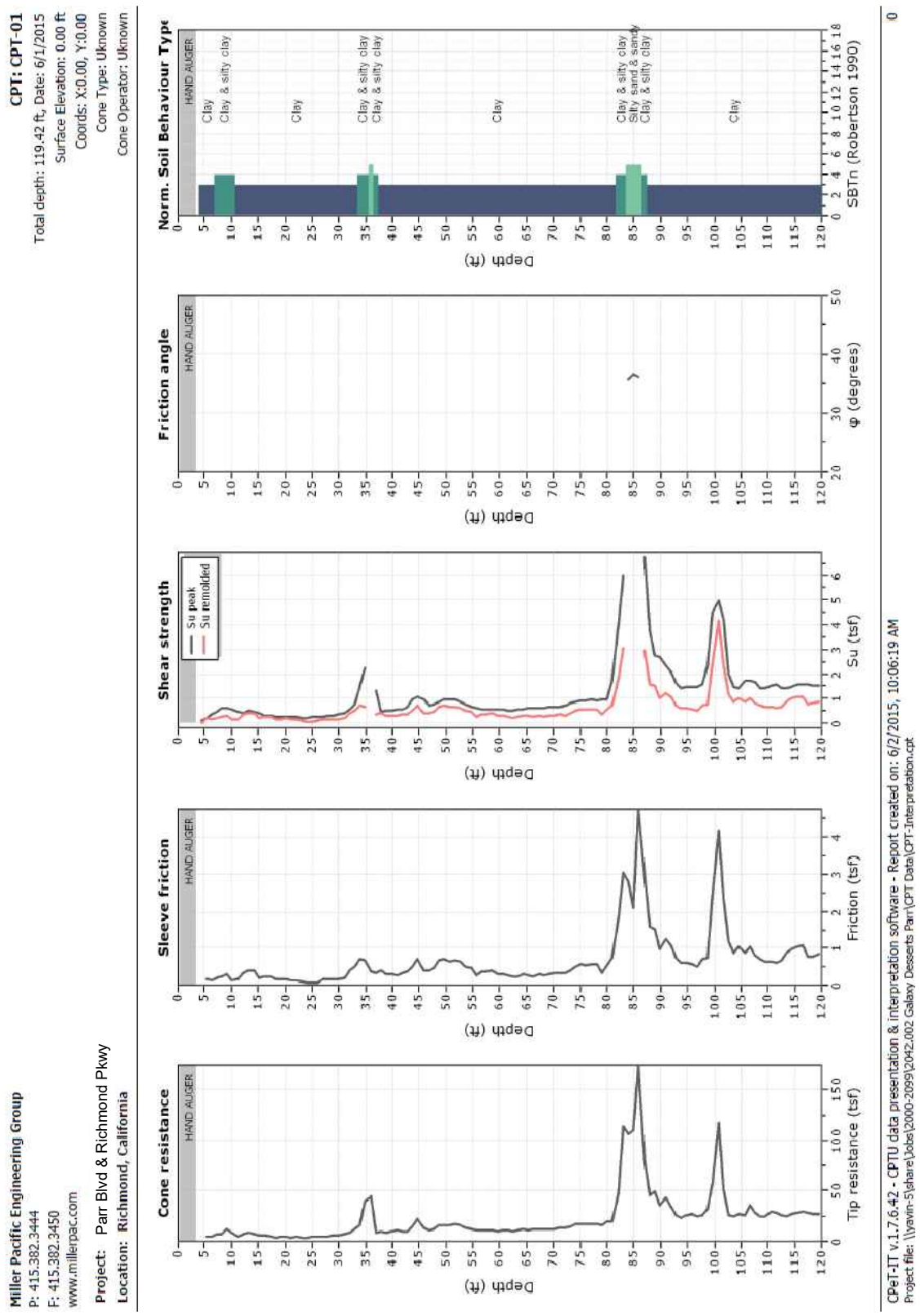
CONE PENETROMETER

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-7
FIGURE



Miller Pacific
ENGINEERING GROUP

FILE: 2042.002 CPT.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

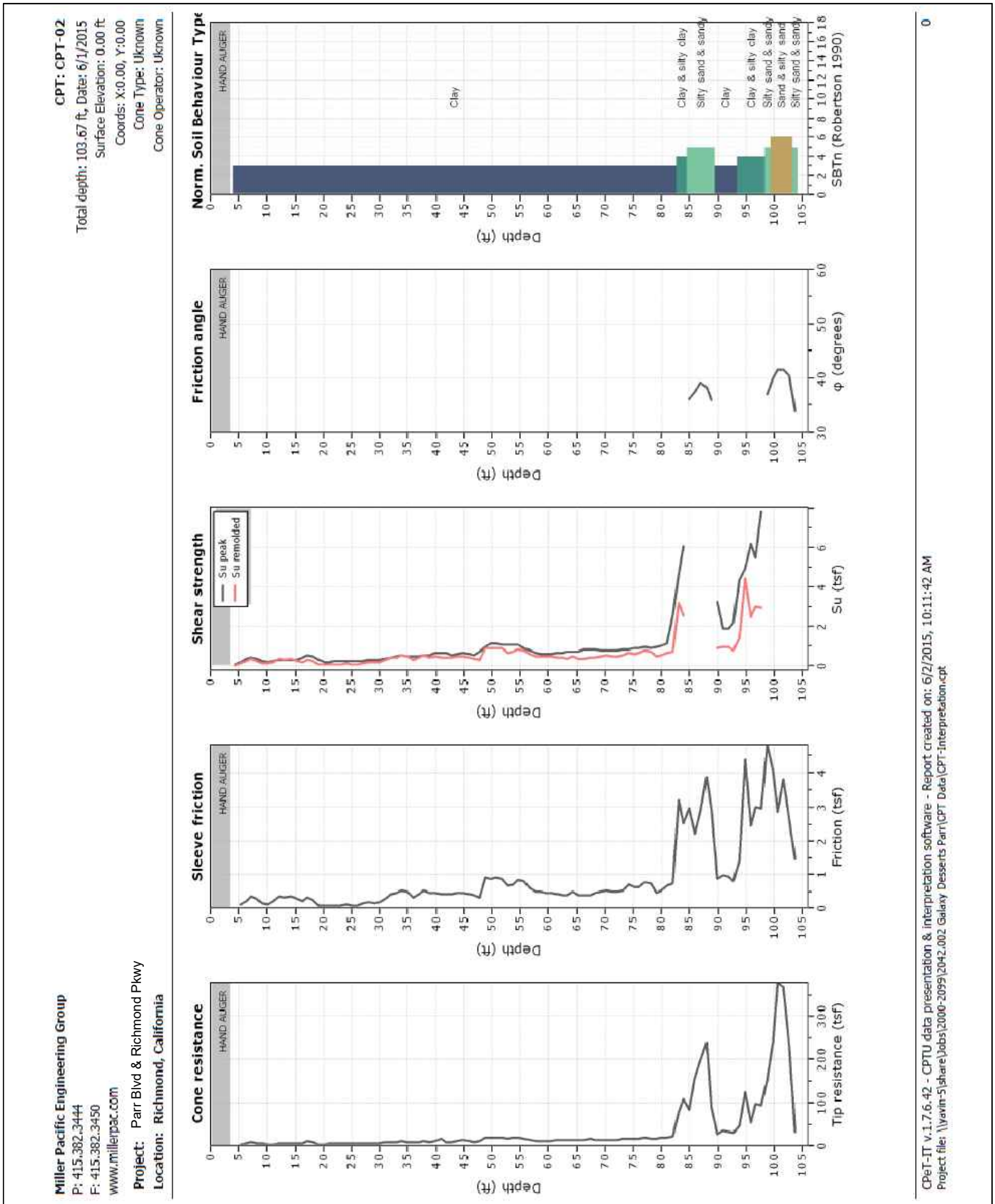
CONE PENETRATION TEST 1

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-8
FIGURE



Miller Pacific
ENGINEERING GROUP

FILE: 2042.002 CPT.dwg

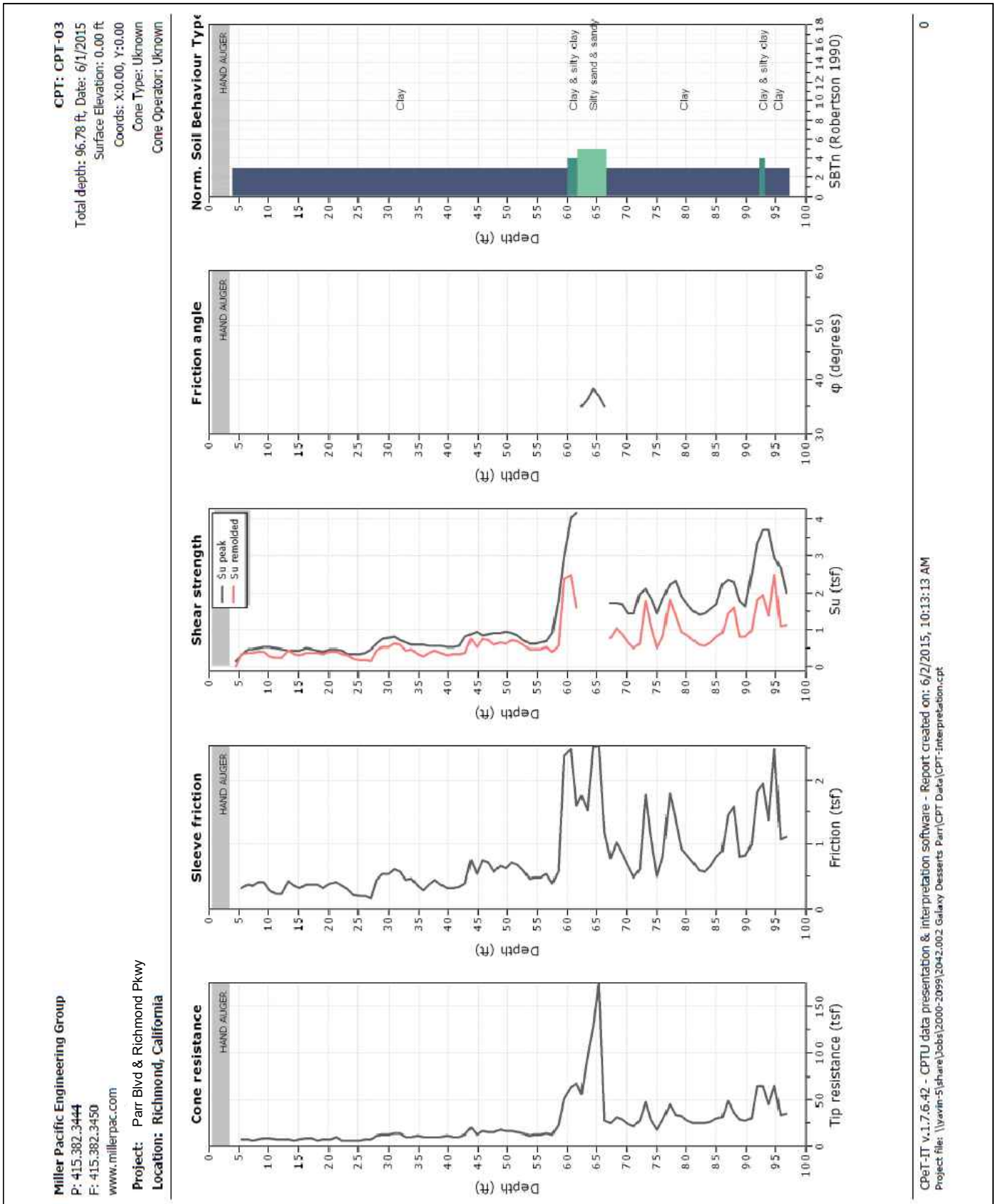
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

CONE PENETRATION TEST 2

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California
Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-9
FIGURE



Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 CPT.dwg

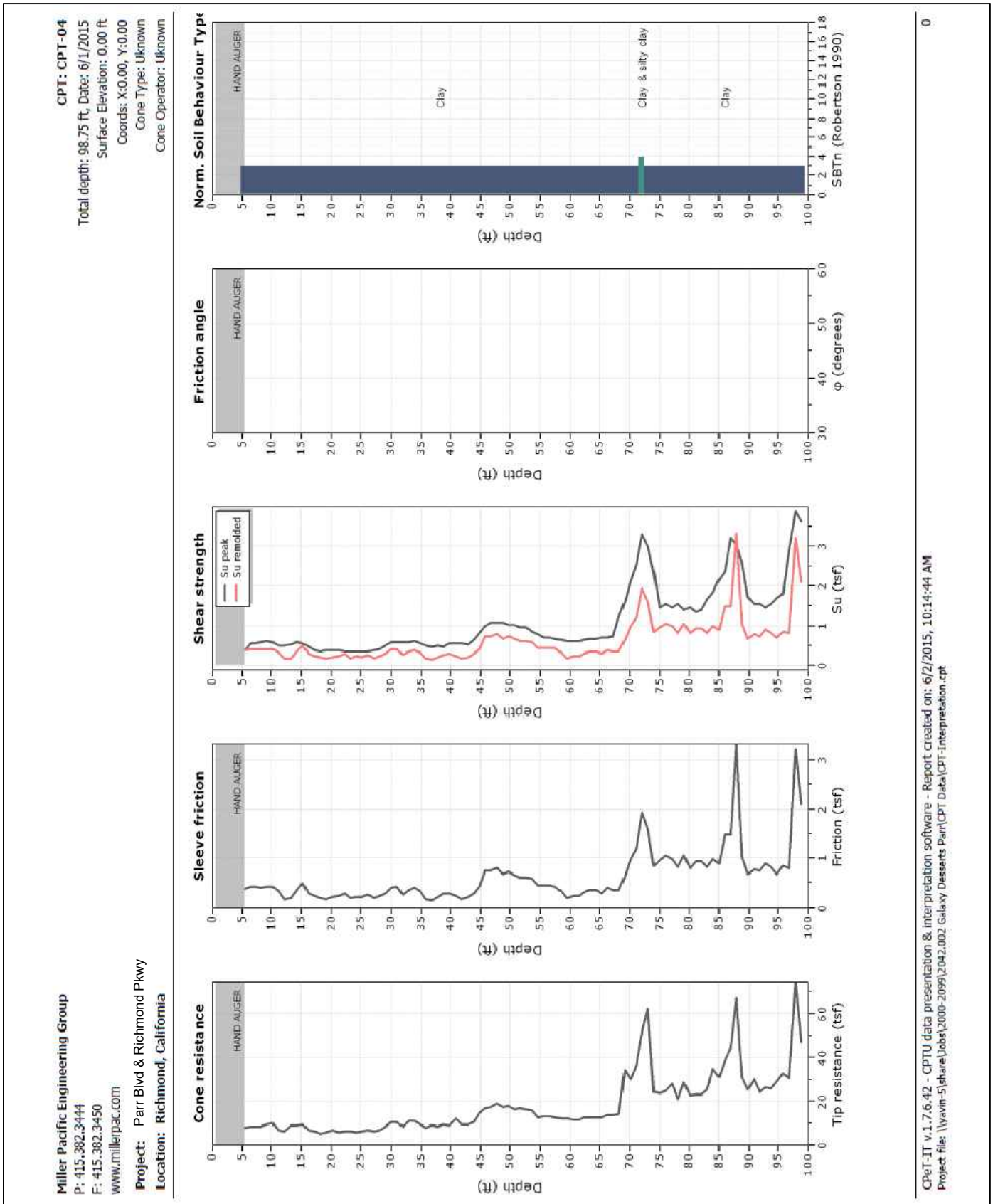
CONE PENETRATION TEST 3

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-10
FIGURE



Miller Pacific
ENGINEERING GROUP

FILE: 2042.002 CPT.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

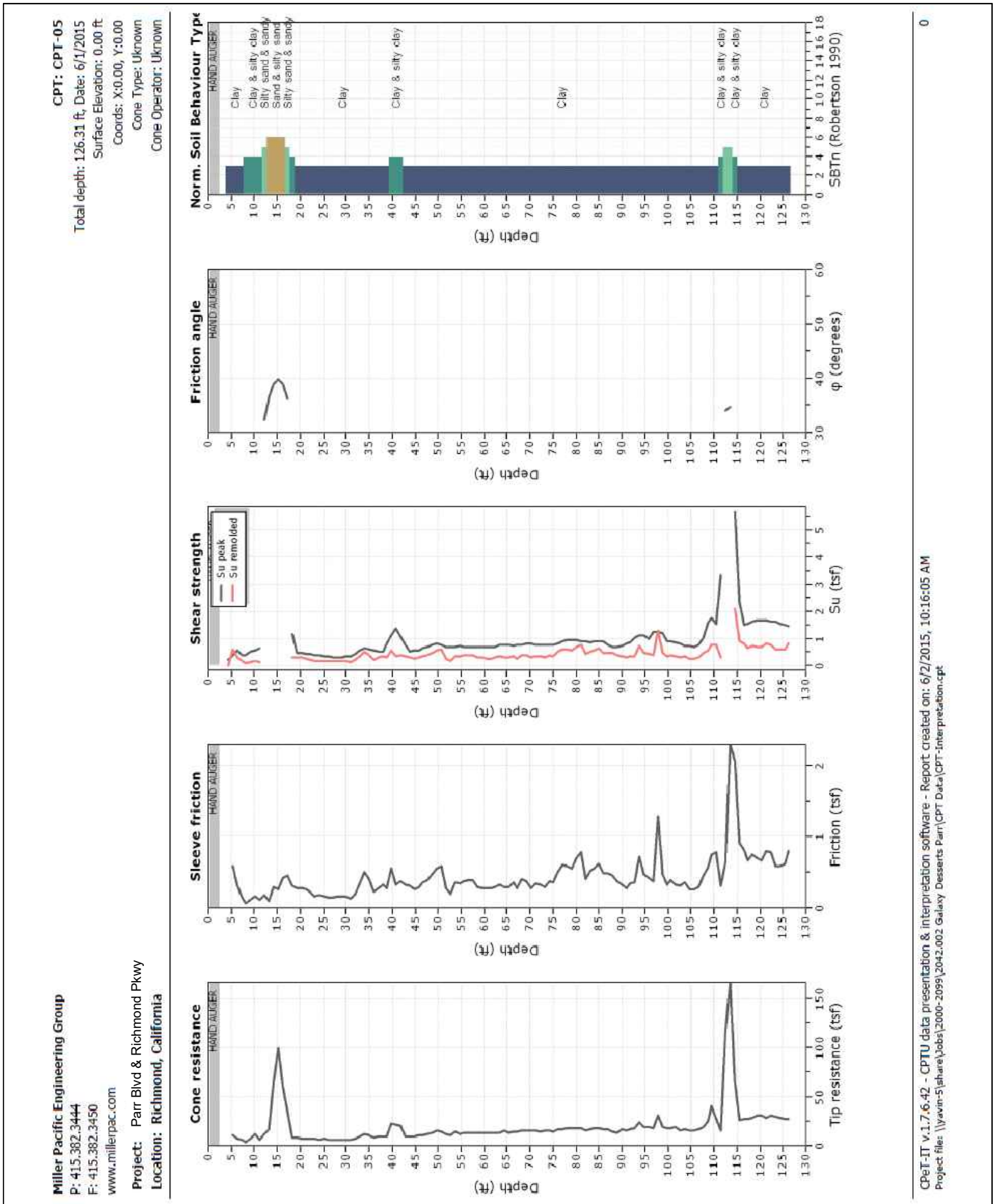
CONE PENETRATION TEST 4

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-11
FIGURE



Miller Pacific
ENGINEERING GROUP

FILE: 2042.002 CPT.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

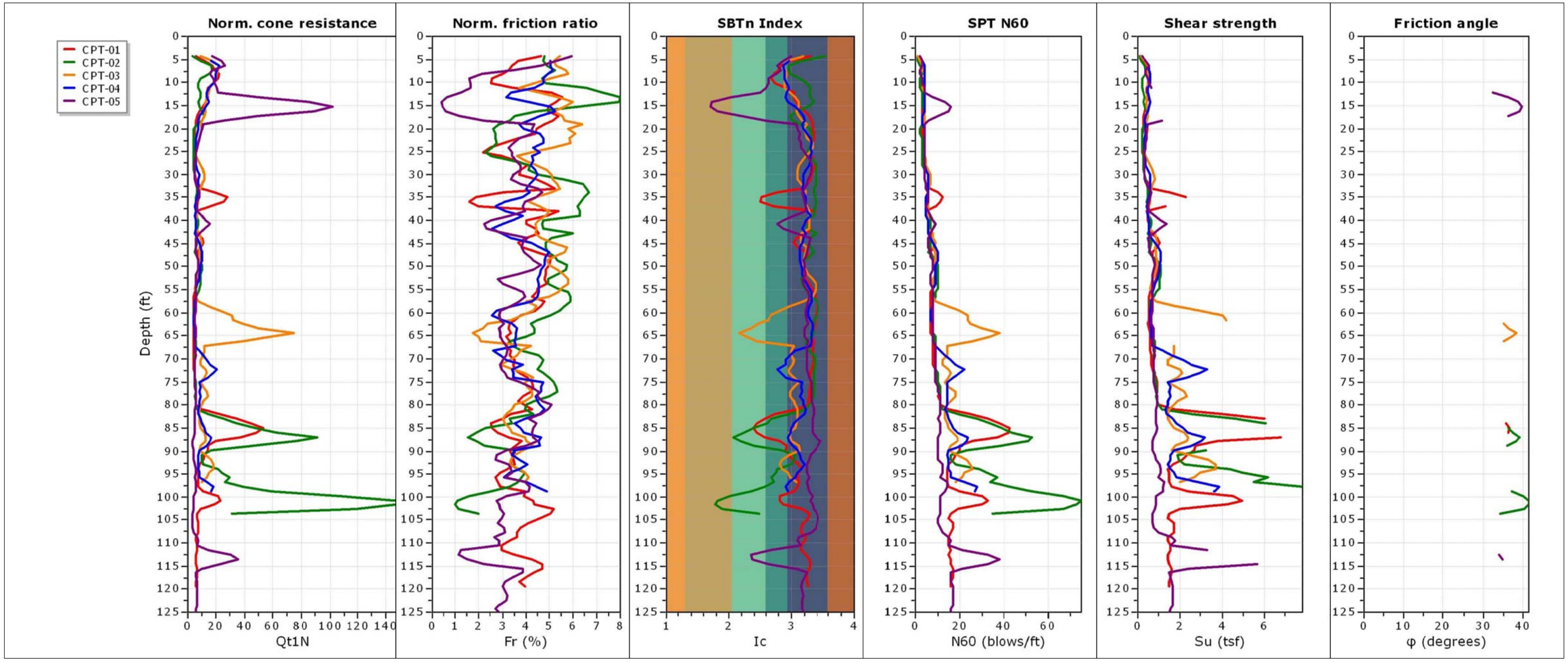
CONE PENETRATION TEST 5

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California

Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-12
FIGURE



SBTn LEGEND

- Gravely SAND to SAND
- Clean SAND to Silty SAND
- Silty SAND to Sandy SILT
- Clayey SILT to Silty CLAY
- CLAY to Silty CLAY
- Organic Material

Miller Pacific
ENGINEERING GROUP

FILE: 2042.002CPT.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

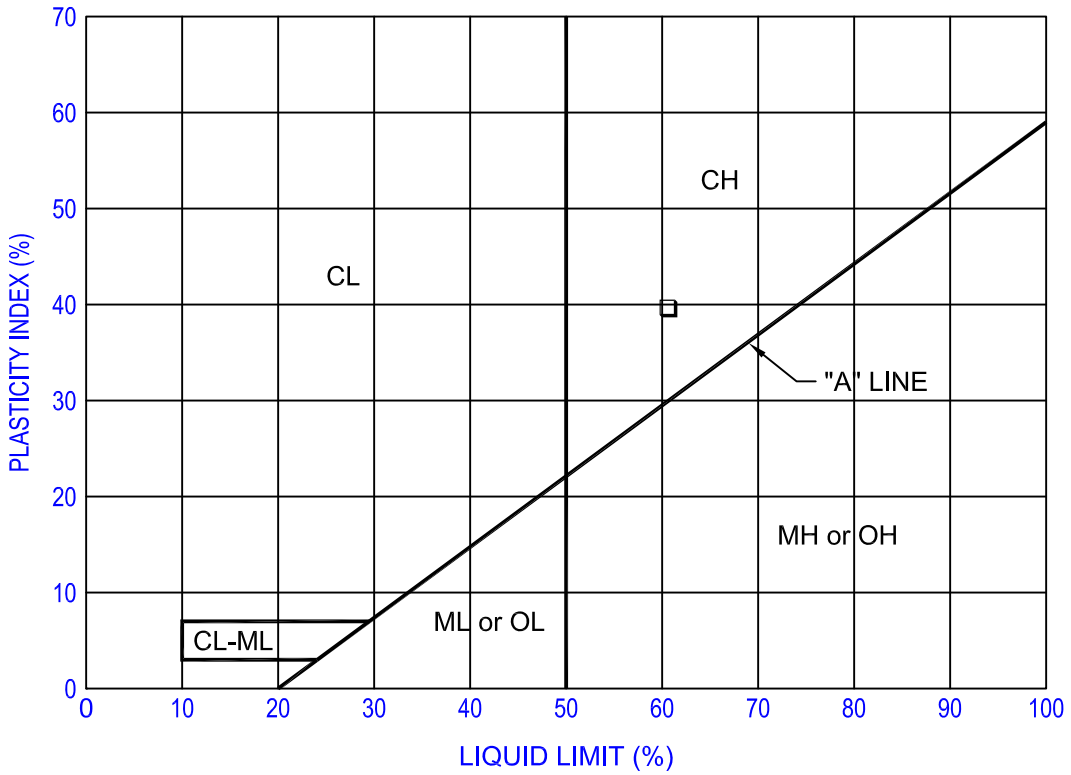
CPT DATA SUMMARY

Scannell Properties
Parr Blvd. and Richmond Pkwy
Richmond, California

Project No. 2042.002 Date: 6/05/15

Drawn BSP
Checked

A-13
FIGURE



SYMBOL	SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)
❑	BORING 1 6.5 Feet	CLAY (CH) dark gray	61	22	39

REFERENCE: Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D 4318

Miller Pacific
ENGINEERING GROUP

FILE: 2042.002 PI.dwg

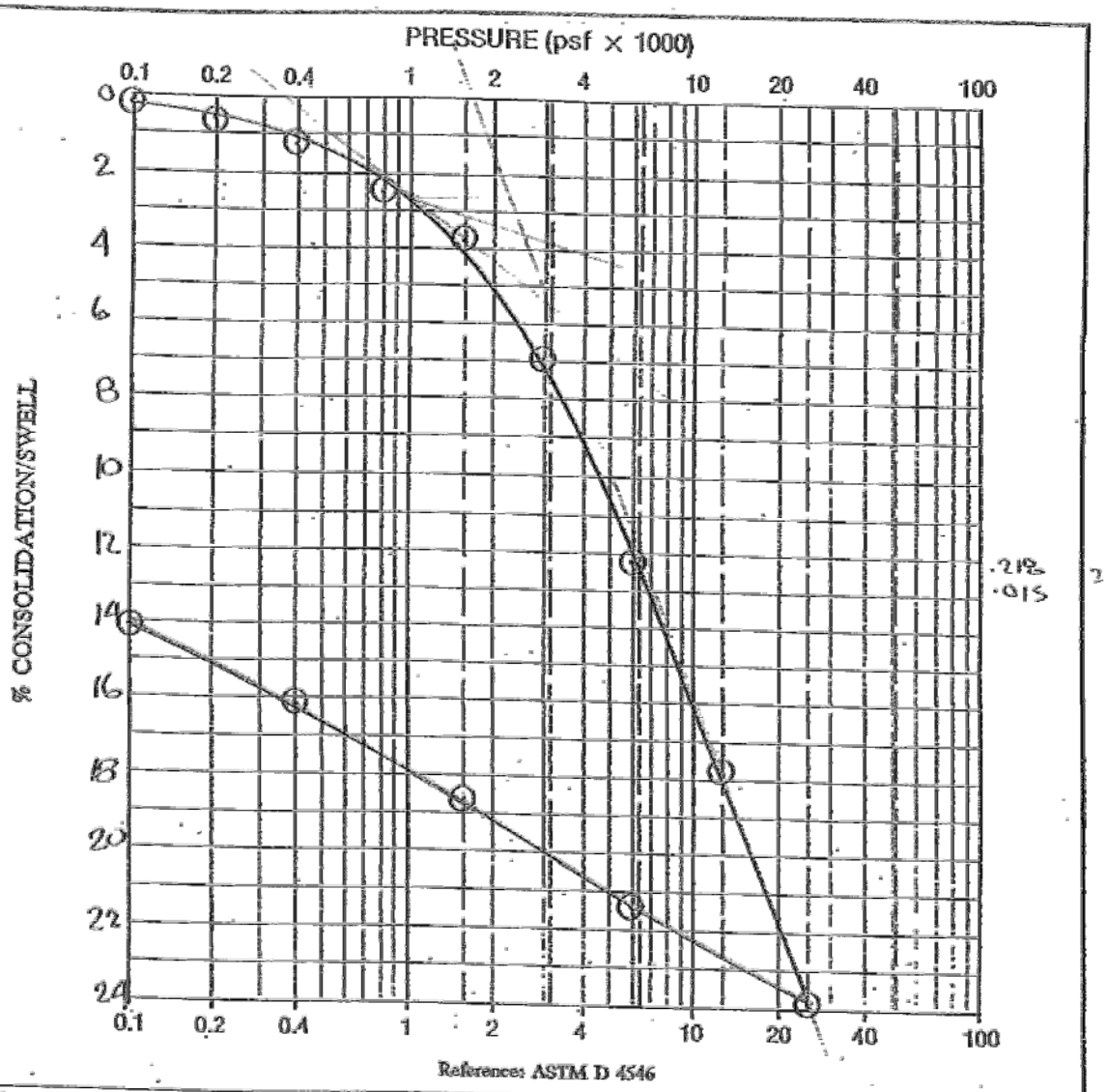
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

PLASTICITY INDEX

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California
Project No. 2042.002 Date: 6/16/15

Drawn NGK
Checked

A-14
FIGURE



Type of Specimen UNDISTURBED		Condition		Before Test		After Test	
Diameter (in.) 2.43	Height (in.) 0.80	Water Content	w_o	45.9 %	w_f	35.1 %	
Overburden Press., P_o	psf	Void Ratio	e_o	1.309	e_f	0.985	
Preconsol. Press., P_c (Casa.) 2600 psf		Saturation	S_o	99 %	S_f	100 %	
Compression Index, C_c 0.203		Dry Density	γ_d	76.0 pcf	γ_d	88.4 pcf	
LL	PL	PI			Gs 2.81		
Classification DK GRAY CLAY (CH)			Source B1 @ 15'				

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 118.1.18
Date: 6/17/15
Appr: [illegible]

CONSOLIDATION/SWELL TEST REPORT
MILLER / GALAXY DESSENTS
#2042.002

PLATE

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

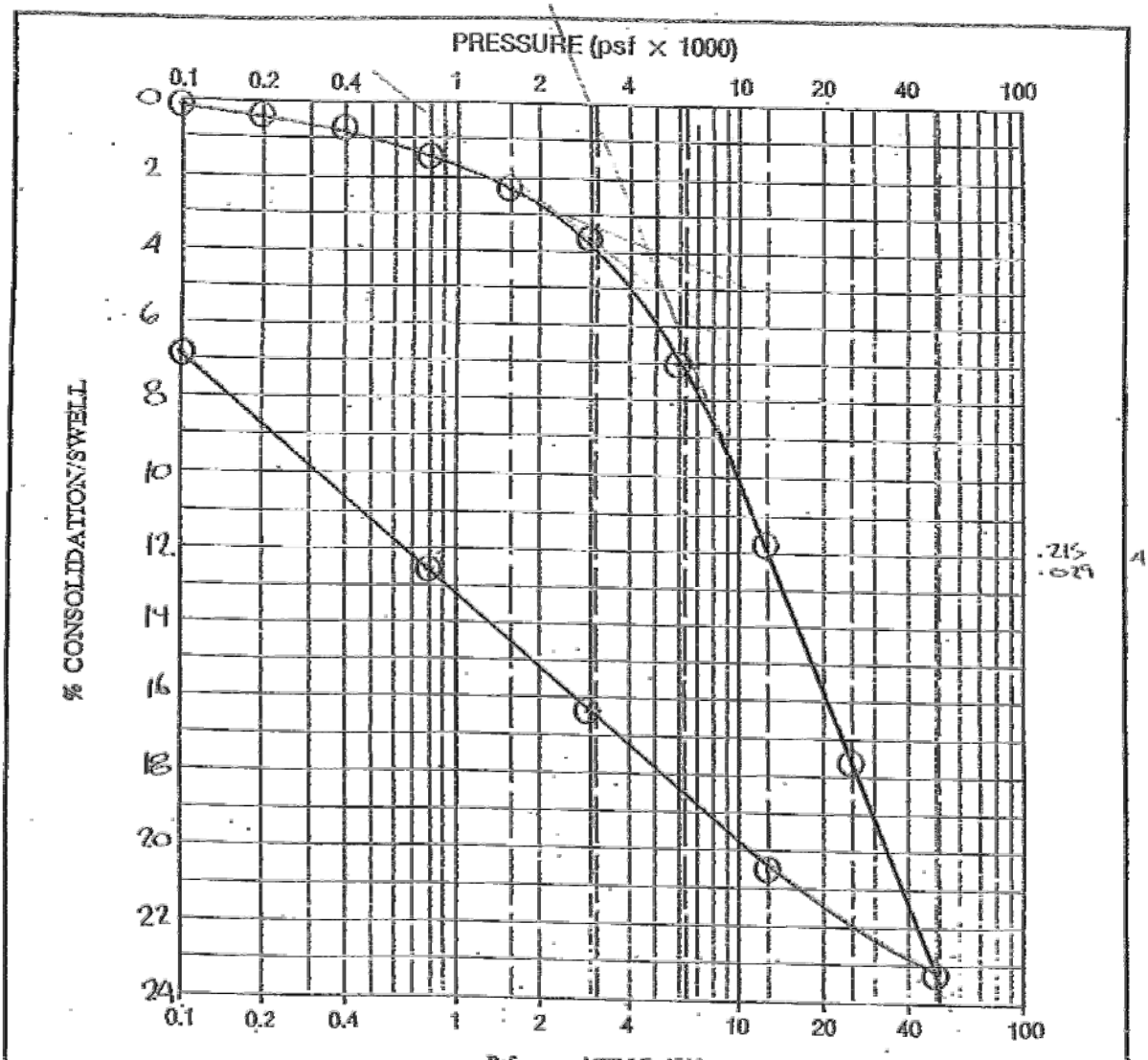
FILE: 2042.002 Consol.dwg

CONSOLIDATION TEST RESULTS - B1 AT 15-FEET

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California
Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-15
FIGURE



Reference: ASTM D 4546

Type of Specimen UNDISTURBED		Condition		Before Test		After Test	
Diameter (in.) 2.43	Height (in.) 0.80	Water Content	w_o	40.6 %	w_f	36.0 %	
Overburden Press., P_o	psf	Void Ratio	e_o	1.075	e_f	1.016	
Preconsol. Press., P_c (Casa) 4600 psf		Saturation	S_o	98 %	S_f	100 %	
Compression Index, C_c 0.186		Dry Density	γ_d	81.0 pcf	γ_d	87.1 pcf	
LL.	PL.	PI		Gs 2.81			
Classification DK GRAY CLAY (CH)		Source B1 @ 35'					

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 18.1.18
Date: 6/17/15
Appr: [illegible]

CONSOLIDATION/SWELL TEST REPORT
MILLER/GALAXY DESERTS
#2042.002

PLATE

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 2042.002 Consol.dwg

CONSOLIDATION TEST RESULTS - B1 AT 35-FEET

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California
Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-16
FIGURE

E T S

Environmental Technical Services

975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
e-mail: *entech@pacbell.net*

-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support

Serving people and the environment so that both benefit.

COMPANY: Miller Pacific Engineering Group, 504 Redwood Blvd., Suite 220, Novato, CA 94947		ANALYST(S)	SUPERVISOR
ATTN: Scott Stephens	DATE of	D. Salinas	D. Jacobson
JOB & SITE: Galaxy Desserts, Parr Blvd., Richmond, Calif.	DATE RECEIVED / COMPLETION	S. Santos	LAB DIRECTOR
PROJ.NUM.: 2042.002	6/10/2015 / 6/18/2015		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL MIN. RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
06411-1	GD1-PB/R	Native Soil (B1+B2 Comp @ 0'-20')	**7.22**	**154**	**[6490]**	**840**	**2,565**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
Method	Detection	Limits --->	---	0.1	0.1	1	0.1

COMMENTS

Resistivity is under 500 ohm-cm which is very poor (assign 10 pts, all specs), although soil reaction (i.e., pH) is mildly alkaline (assign Ø pts); both sulfate & chloride are greatly elevated (SO4 @ >200 ppm, assign Ø-2 pts, Cl @ >1,000 pts, assign 3 pts). The standard CalTrans times to perforation for 18 ga galvanized in this soil are <12 yrs, and ~25yrs for 12 ga; and the calculated average pitting rate (following Uhlig) is ≈0.64 mm/yr, thus pitting to a 2 mm depth is only ≈3 yrs. Chloride is so high that it would constitute a serious problem for concrete reinforcement and exposed steel as well. The calculated time to failure due to corrosion of rebar in standard concrete at standard depth is just 24 yrs. Sulfate is high enough that there would be at least mild adverse impact to cement, concrete, grout or mortar. Lime or mild cement treatment would not be of any benefit in this case. Considerable, and very likely, impractical steel upgrading (i.e., increased gauge and more resistant steels) would be needed. Cathodic protection of wrapped assets probably would be impractical as well requiring a huge amount of DC current. Other options are increased or special engineering fill, use of a polymer coating, or use of bonded plastic coated steels, or use of plastic, fiberglass or hardened concrete assets, etc. Total point value for this soil would be in the range of 13-15 pts, depending on the specifications. Standard concrete mixes would not be acceptable in this soil based on these results, i.e., upgraded concrete would be needed (i.e., ASTM Type II, at least, and possibly ASTM Type 5, as well as heavier rebar). It is probably the case that this soil has been influenced by brackish water since Na and Cl levels are elevated in such a way as to suggest that this has happened.

\\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

Miller Pacific
ENGINEERING GROUP

FILE: 2042.002 Corrosion.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

SOIL CORROSION TEST RESULTS

Scannell Properties
Parr Blvd. and Richmond Pkwy.
Richmond, California
Project No. 2042.002 Date: 7/22/15

Drawn EDT
Checked

A-17
FIGURE

E.2 - Paleontological Records Search

THIS PAGE INTENTIONALLY LEFT BLANK

Kenneth L. Finger, Ph.D.
Consulting Paleontologist

18208 Judy St., Castro Valley, CA 94546-2306 510.305.1080 klfpaleo@comcast.net

September 17, 2019

Dana DePietro
FirstCarbon Solutions
1350 Treat Boulevard, Suite 380
Walnut Creek, CA 94597

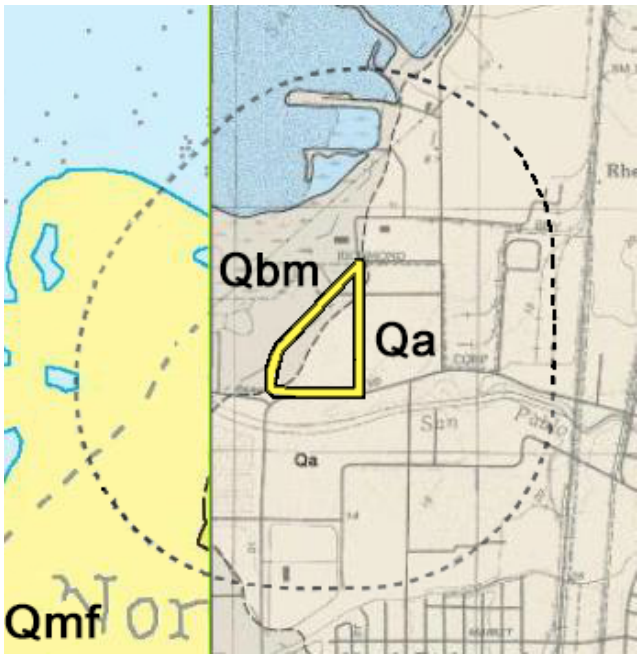
Re: Paleontological Records Search for Records Search for Scannell Properties Warehouse Project (2648.0014), North Richmond, Contra Costa County

Dear Dr. DePietro:

As per your request, I have performed a paleontological records search for the Scannell Properties Warehouse project in North Richmond. The project site is located on the east side of Richmond Parkway and north side of Parr Boulevard. Its PRS location is SW¼, SW¼, Sec. 36, T2N, R5W, Richmond quadrangle (USGS 7.5-series topographic map). Google Earth imagery shows that the project site is on flat terrain that appears to have been disturbed by industrial and possibly agricultural use.

<u>Geologic Units</u>

The adjacent geologic map shows a marked discontinuity because it of combines adjoining parts of those by Blake et al. (2000) and Dibblee and Minch (2005). The surface of the area of the project site (yellow outline at center) consists of both bay mud (Qbm) and alluvium (Qa). The surrounding half-mile search area (dashed outline) also has artificial fill overlying marine and marsh deposits (Qmf). All of these units are Holocene and therefore too young to be fossiliferous.



<u>UCMP Records Search</u>

A records search was not performed because Holocene units have no paleontological potential or sensitivity. In addition, the absence Pleistocene or older deposits in the search area suggests that any in the subsurface of the project site would be a depths below all earth-disturbing construction activities.

Remarks and Recommendations

No further paleontological mitigation measures are recommended because potentially fossiliferous deposits are not surficially mapped on or near the project site.

Sincerely,

References Cited

Blake, M.C., Graymer, R.W., Jones, D.L., and Soule, A., 2000, Geologic map and map database of parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California, U.S. Geological Survey Miscellaneous Field Studies Map MF-2337, scale 1:75,000.

Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Richmond quadrangle, Contra Costa & Alameda Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-147, scale 1:24,000.