ROLLO & RIDLEY
GEOTECHNICAL ENGINEERS & SCIENTISTS

GEOTECHNICAL INVESTIGATION
703 THIRD STREET
San Rafael, California

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PLANNING

703 Third Street Associates, LLC
San Rafael, California

July 2, 2018
Project No. 1570.1

ROLLO & RIDLEY
GEOTECHNICAL ENGINEERS & SCIENTISTS

July 2, 2018
Project No. 1570.1

703 Third Street Associates, LLC
c/o Seagate Properties, Inc.
Attn: Dennis P. Fisco
980 Fifth Avenue
San Rafael, California 94901

Subject: Geotechnical Investigation
703 Third Street
San Rafael, California

Dear Mr. Fisco:

Our geotechnical investigation report for the proposed development at 703 Third Street in San Rafael, California is attached. The services described in the report are outlined in our proposal dated March 20, 2018 and executed professional service agreement dated May 1, 2018. This cover letter omits detailed findings and conclusions; therefore, anyone relying on the report should read it in its entirety. Our conclusions and recommendations apply only to the project described in the report.

The rectangular-shaped lot is bound by Third Street to the north, Tamalpais Avenue to the east, Lincoln Avenue to the west and a private property to the south. The gently sloping property is approximately 140 feet by 196.5 feet in plan and comprised of several one- to two- story commercial buildings and surface parking lots. We understand the buildings and associated asphalt parking lots will be demolished and removed to make space for the proposed development.

We understand the project includes the construction of a six-story structure at grade (no basement levels) which will encompass the majority of the lot. The structure will include commercial space along Third Street with car-lift systems (stacker parking) garage over the remainder of the first floor accessed from Tamalpais and Lincoln Avenues. Floors 2 through 6 will include residential units and roof decks as the structure steps in at the perimeter and includes an interior atrium feature.

During our subsurface investigation, we encountered approximately 5 to 6-½ feet of fill consisting of loose to medium dense sand with gravel and clayey sand. Below the sandy fill we encountered alluvium deposits and residual bedrock consisting of medium dense to very dense clayey sand or clayey sand with gravel and very stiff to hard sandy clay to sandy clay with gravel. The alluvium/residual bedrock is underlain by Franciscan Complex shale and sandstone to the maximum depth explored during this investigation. Unstabilized groundwater readings were recorded at depths of approximately 24- feet during our field investigation, but groundwater has been encountered at depths of 9- to 19- feet by others in the vicinity.

On the basis of our field investigation and engineering analysis, we judge the proposed development may be supported on either: 1) a shallow foundation consisting of a reinforced concrete mat provided the mat bear a minimum of 18 inches into competent alluvial soils (below the fill at a depth of approximately 8 feet) or 2) a new reinforced concrete grid foundation or mat supported on drilled displacement columns (DDCs) which extend through the fill and into the competent native soil (alluvium). Detailed design recommendations are contained within this report.

The recommendations contained in the report are based on limited subsurface exploration. Variations between expected and actual soil conditions may be found in localized areas during construction. Consequently it may be necessary to perform additional field explorations after demolition of the existing structures and prior to foundation construction depending on the foundation system selected. These additional studies will allow us to verify the areas of the site where access was not obtainable and make supplemental recommendations to this report if deemed necessary.

We appreciate the opportunity of being of service to you on this project and look forward to working with you during construction.

Best regards,
ROLLO & RIDLEY, INC.

Chris Y. Tan

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ROLLO & RIDLEY
GEOTECHNICAL ENGINEERS & SCIENTISTS

GEOTECHNICAL INVESTIGATION
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San Rafael, California

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ROLLO & RIDLEY

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GEOTECHNICAL INVESTIGATION
703 THIRD STREET
San Rafael, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rollo & Ridley, Inc. for the proposed development at 703 Third Street in San Rafael, California.

The rectangular-shaped lot is bound by Third Street to the north, Tamalpais Avenue to the east, Lincoln Avenue to the west and a private property to the south, as shown on the Site Location Map, Figure 1. The gently sloping property is approximately 140 feet by 196.5 feet in plan and comprised of several one- to two- story commercial buildings and surface parking lots. We understand the buildings and associated asphalt parking lots will be demolished and removed to make space for the proposed development.

We understand the project includes the construction of a six-story structure at grade (no basement levels) which will encompass the majority of the lot. The structure will include commercial space along Third Street with car-lift systems (stacker parking) over the remainder of the first floor accessed from Tamalpais and Lincoln Avenues. Floors 2 through 6 will include residential units and roof decks as the structure steps in at the perimeter and includes an interior atrium feature. The approximate limits of the property, existing and proposed new structures are shown on the Site Plan, Figure 2.

The services described in this report was performed in accordance with: 1) discussions (and correspondence) with members of the design team, 2) a review of the available preliminary drawings prepared by Van Meter Williams Pollack, LLP dated February 23, 2018 and two geotechnical reports at 930 Tamalpais Avenue and at Second & Lincoln, 3) the results of our field investigation and engineering analysis performed for the site and 4) our experience and knowledge of the subsurface conditions from other projects in the vicinity of the property.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our proposal dated March 20, 2018. These services consisted of reviewing previously performed geotechnical investigations in the vicinity of the site, exploring the subsurface conditions at the site, performing laboratory tests and engineering analyses, and developing conclusions and recommendations regarding:

- soil, bedrock and groundwater conditions at the site
- seismic hazards, including liquefaction and differential compaction
- appropriate foundation type(s) for the proposed structure
- design criteria for the recommended foundation type(s)
- estimates of total and differential foundation settlement
- design parameters for shoring (if necessary)
- lateral pressures for below grade walls, including a design earthquake increment
- construction monitoring
- site preparation and grading, including criteria for fill quality and compaction
- 2016 California Building Code (CBC) seismic criteria, as appropriate
- construction considerations

3.0 FIELD INVESTIGATION

To explore subsurface conditions at the property, we drilled four borings on May 22 and 23, 2018. The borings, designated as RR-1 through RR-4, were drilled to depths of approximately 35.3 to 46.4 feet below the existing grades. The locations of the borings are shown on the Site Plan, Figure 2.

Each boring was drilled using a truck-mounted drilled rig equipped with hollow-stem augers, operated by HEW Drilling of Palo Alto, California. Prior to commencing the drilling, we obtained a drilling permit from the San Rafael Environmental Health Services (SREHS). During drilling, our engineer logged the materials encountered and obtained samples for visual classification and laboratory testing. Logs of the borings are presented in Appendix A, as Figures A-1 through A-4. The soil and bedrock encountered was classified according to the classification chart and physical properties criteria for rock described in Appendix A as Figures A-5 and A-6, respectively.

Soil samples were obtained using two types of samplers:

- a Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter
- a Standard Penetration Test (SPT) sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter

Both samplers were driven with a 140-pound, above ground, automatic hammer falling 30 inches. The blow counts required to drive the S&H sampler the final 12 inches of an 18-inch drive were converted to SPT blow counts using a factor of 0.8 for sampler diameter. The blow counts required to drive the SPT sampler the final 12 inches of an 18-inch drive were converted to SPT blow counts using a factor of 1.2; both the actual and converted blow counts are presented on the boring logs. Each soil sample collected was re-examined in our office to confirm field classifications and select samples were tested to measure moisture, density and fines content. The results of the laboratory tests are presented on the boring logs.

At the completion of drilling, each boring was backfilled with cement grout as required by the San Rafael Environmental Health Services (SREHS).

4.0 SUBSURFACE CONDITIONS

As presented on Figures 3 and 4, Idealized Subsurface Profile A-A′ and B-B′, we judge the site is underlain by fill, alluvium/residual bedrock and bedrock of the Franciscan Complex Formation. A general description of each layer and its approximate extent are discussed below:

Fill – Fill was encountered in each boring from the ground surface to a depth ranging from 5 feet to 6.5 feet. The fill is predominantly sand with gravel and clayey sand and blow count data indicate it is loose to medium dense. The fill may have been placed during historical grading at the site to create level areas across the site. The fill should not be relied on for foundation support.

Alluvium – Alluvium consisting of clayey sand with gravel, sandy clay and sandy clay with gravel, underlies the fill layer. Blow count data indicates this layer is medium dense to dense, very stiff to hard and extends to depths of 19.5 to 34 feet below the existing ground surfaces. This material is moderately strong to strong and capable of withstanding low to moderate building loads.

Residual Bedrock – The alluvium is underlain by residual bedrock consisting of clayey sand with gravel and sandy clay with gravel, with occasional weathered bedrock fragments, consisting of sandstone and shale. Blow count data indicates this layer is very dense and hard extending to depths of 30.5 to 42.5 feet below the existing ground surfaces.

Franciscan Complex – Below the fill and alluvium and residual bedrock, Franciscan Complex bedrock was encountered to the maximum depth explored during this investigation (approximately 46 feet). This material consists of shale and sandstone bedrock. The bedrock encountered is intensely fractured, is low to moderately hard, and is friable to weak and deeply weathered. We anticipate the bedrock becomes less fractured, harder, stronger and less weathered with depth.

A published geology map of the vicinity, presented as Figure 5, shows the site is underlain by "Alluvium", which is consistent with our findings.

We encountered groundwater at a depth of approximately 24 feet during the field exploration. However, these groundwater measurements are considered "unstabilized" since the drilling permit requires us to backfill the boring with grout immediately after drilling. Borings performed by others for projects in the vicinity of the site recorded groundwater at depths varying from 9- to 19- feet. In addition, tidally influenced groundwater from the nearby San Rafael Creek and surface water infiltration (from rain or landscaping irrigation) may travel as perched water in pervious soil seams at a shallower depth or along the contact of the fill and alluvium and/or the alluvium and Franciscan Complex layers; seasonally fluctuations are likely.

5.0 REGIONAL GEOLOGY

The site is within the Coast Range geomorphic province that is characterized by northwest-trending valleys and ridges. Folds and faults that resulted from the collision of the Pacific (Farallon) and North American plates and subsequent strike-slip faulting along the San Andreas Fault Zone control the geology. Bedrock underlying the general region is primarily of the Franciscan Complex.

The Franciscan Complex is a disrupted assemblage of large and small inclusions of various hard rock types embedded in a fine-grained matrix of intensely sheared and crushed rock material. Inclusions of coherent rocks in the mélange matrix may range in size from an inch to several miles.

Sandstone and shale are the most abundant inclusion type, with lesser amounts of conglomerate, serpentinite, calcium-silicate rock, schist, and other metamorphic rocks.

6.0 SEISMICITY AND SEISMIC HAZARDS

6.1 Regional Seismicity and Faulting

The major active faults in the area are the San Andreas, Hayward and San Gregorio Faults. These and other active faults of the region are shown on Figure 6. For each of the active faults within about 65 kilometers (km) of the site, the distance from the site and the mean characteristic Moment magnitude[1] [2007 Working Group on California Earthquake Probabilities (WGCEP) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
Total Hayward	14	Northeast	7.00
Total Hayward-Rodgers Creek	14	Northeast	7.33
Rodgers Creek	15	Northeast	7.07
N. San Andreas - North Coast	15	West	7.51
N. San Andreas (1906 event)	15	West	8.05
N. San Andreas - Peninsula	21	South	7.23
San Gregorio Connected	24	Southwest	7.50
Point Reyes	27	West	6.90
West Napa	33	Northeast	6.70
Green Valley Connected	40	East	6.80
Mount Diablo Thrust	44	East	6.70
Total Calaveras	48	East	7.03
Great Valley 5, Pittsburg Kirby Hills	57	East	6.70
Hunting Creek-Berryessa	61	Northeast	7.10
Greenville Connected	61	East	7.00
Great Valley 4b, Gordon Valley	62	Northeast	6.80
Monte Vista-Shannon	64	Southeast	6.50

Figure 6 also shows earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through September 2014. Since 1800, at least three and possibly four major earthquakes have been recorded on the San Andreas Fault, which as shown on Table 1 is the closest major active fault to the 703 Third Street site. In 1836 an earthquake with an estimated Moment magnitude, M_w, of about 6.25 occurred east of Monterey Bay and may have been located along the San Andreas Fault (as per Toppozada and Borchardt 1998). In

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[1] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

1838 an earthquake with an estimated M_w 7.5 occurred along the Peninsula segment of the San Andreas Fault, rupturing possibly as far south as San Juan Bautista. The 1906 San Francisco Earthquake caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage; this San Andreas Fault earthquake created a surface rupture extending from Shelter Cove to San Juan Bautista, approximately 470 km in length. It had an estimated M_w of about 7.9 and was felt in Oregon, Nevada, and Los Angeles, as far as 560 km away.

The most significant and damaging earthquakes to recently affect the Bay Area were the M_w 6.9 Loma Prieta Earthquake of October 17, 1989 which occurred along the Santa Cruz Mountains segment of the San Andreas fault approximately 118 km south-southeast from the site, and the M_w 6.0 South Napa Earthquake of August 24, 2014 which occurred along the West Napa fault approximately 33 km northeast from the site.

In 1868 an earthquake with an estimated M_w of 6.8 occurred on the southern segment of the Hayward Fault (between San Leandro and Fremont). In 1861, an earthquake of unknown magnitude (possibly M_w of about 6.5) is believed to have occurred along the northern section of the Calaveras Fault. The most recent significant (greater than M6) earthquake on the Calaveras Fault was the M_w 6.2 Morgan Hill earthquake in 1984.

The 2014 WGCEP at the U.S. Geologic Survey has predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (for the period 2014-2043). Table 2 below presents specific probability estimates for a magnitude 6.7 or greater earthquake occurring somewhere along each of the major faults in the Bay Area, taken from the Earthquake Outlook for the San Francisco Bay Region 2014-2043 (USGS, 2016; https://pubs.usgs.gov/fs/2016/3020/fs20163020.pdf).

TABLE 2
WGCEP (2014) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward and Rodgers Creek	33
Calaveras and Paicines	26
San Andreas	22
Concord, Green Valley, Mount Diablo North and South, Greenville, Berryessa, Hunting	16
All Lessor-Known Faults in the San Francisco Bay Region	13
San Gregorio	6

6.2 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Very strong shaking during an earthquake can result in ground failure such as that associated with fault rupture, soil liquefaction[2], lateral spreading[3], and differential compaction[4]. We used the results of our field investigation to evaluate the potential of these phenomena occurring at the project site.

6.2.1 Fault Rupture

Historically, ground surface ruptures closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake

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[2] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[3] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[4] Differential compaction is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

Fault Zoning Act and no known active or potentially active faults exist on the site. Therefore, we conclude the risk of fault offset at the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of fault rupture (surface faulting) from an unknown fault and consequent secondary ground failure is low.

6.2.2 Liquefaction, Lateral Spreading and Differential Compaction

We used the results of our borings to evaluate the potential for liquefaction lateral spreading, settlement from differential compaction and earthquake induced landsliding. As presented on Figure 7, Liquefaction Susceptibility Map, the site (and vicinity) falls on the border where the susceptibility for liquefaction is moderate to very high. However, the results of our field investigation and evaluation of the site indicate dense to very dense clayey sand and clayey sand with gravel were encountered below the groundwater level in all borings. Therefore, we conclude the potential for liquefaction-induced settlement and lateral spreading at the project site is low.

Strong ground shaking can cause unsaturated sand above the groundwater table to densify and settle (referred to as differential compaction). We encountered up to 6.5 feet of loose to medium dense sand with gravel and clayey sand across the footprint of the site. We judge the densification settlement could be as much as one inch. Because the proposed foundations will bear below the fill as outlined later in this report, differential compaction should not affect the proposed structure. However, surrounding streets, sidewalks and utilities, which are embedded in the loose to medium dense sand, may settle up to one inch adjacent to the site.

We did not observe any surficial evidence of historical landsliding or find any published maps indicating historical landsliding on-site; therefore, we conclude the potential for earthquake induced landsliding within the footprint of the proposed improvements is low.

7.0 DISCUSSION AND CONCLUSIONS

On the basis of our field investigation, laboratory testing and engineering analysis, we conclude the project is feasible from a geotechnical standpoint and subsurface conditions at the site are suitable for the proposed development.

The primary geotechnical issues for this project are:

- the presence of up to 6.5 feet of undocumented loose to medium dense sandy fill across the footprint of the site, and
- foundation support

These and other considerations are addressed in the remainder of the section.

7.1 Foundations

On the basis of our studies and experience with other projects with similar subsurface conditions, we judge the proposed development may be supported on a shallow foundation consisting of either: 1) a reinforced concrete a mat bearing a minimum of 18 inches into competent alluvial soils (below the fill at a depth of approximately 8 feet) or 2) an interconnected reinforced concrete grid of continuous footings or a mat at grade supported on drilled displacement columns (DDCs) which extend through the fill and into the competent native soil (alluvium). Total and differential settlements (between normally spaced columns) under either option using the allowable bearing pressures presented below should be less than one inch.

7.2 Construction Considerations

The near surface soil to be excavated consists mainly of sandy soil with gravel, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. Excavations made near the property line may require shoring if space at the sidewalk is not allowed to make a sloped cut.

Groundwater was encountered at depths of approximately 24 feet during our field investigation and at depths of 9- to 19- feet by others in the vicinity. Hence, we do not anticipate wet soil conditions in any of the shallow excavations (anticipated to be 8 feet or less) unless near surface water infiltration (from tidal, rain or landscaping irrigation) enters the excavations as perched water. If perched water is encountered, perimeter subdrains may be required during construction to keep the excavation dry. However, during construction, the subgrade of excavations should not be allowed to become dry and should be kept moist prior to foundation installations.

8.0 RECOMMENDATIONS

Recommendations for site preparation, grading and drainage, foundations, lowest floor slab, basement and retaining walls, sloped cuts and seismic design are presented in this section of the report.

8.1 Site Preparation, Grading and Drainage

Prior to construction, the areas of the site to be improved should be cleared of vegetation and soil containing greater than four percent organic materials by dry weight of soil. Stripped materials should be removed from the site or stockpiled for later use in landscaped areas, if approved by the architect.

If fill is required, it should consist of on-site or imported soil that is free of organic matter, non-corrosive, non-hazardous, contains no rocks or lumps larger than three inches in greatest dimension, has a liquid limit less than 40 and plasticity index (PI) less than 15, and is approved by the geotechnical engineer. However, the lower PI material encountered on-site may be used as fill. Fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[5].

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[5] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the latest ASTM D1557 laboratory compaction procedure.

In areas that will receive vehicular traffic, the upper eight inches of the subgrade should be compacted to at least 95 percent relative compaction to achieve a firm, unyielding subgrade. The soil subgrade should be kept moist until it is covered by aggregate base. Aggregate base should be compacted to at least 95 percent relative compaction.

The geotechnical engineer should approve all sources of imported engineered fill at least three days before its planned use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed import material. If the on-site material is to be exported, analytical testing of the soil may be required by the party or parties receiving the soil.

Backfill for utility trenches and other excavations is also considered fill, and it should be compacted according to the recommendations provided above. If imported or existing clean sand or gravel is used as backfill, however, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill is not permitted.

As a minimum, new sidewalks, curbs and gutters should be constructed as outlined by the City of San Rafael and observed and tested by a special inspection agency (or us) as appropriate.

We should review the civil drawings prior to plans being submitted for permit.

8.2 Foundations

Our conclusions and recommendations regarding either a deepened mat foundation or shallow foundations supported by DDCs (improved ground) are presented below.

8.2.1 Deepened Mat Foundation

If a deepened mat foundation is the selected foundation type, the upper 5 to 6.5 feet of undocumented sandy fill should be removed to expose competent native soil consisting of dense to very dense clayey sand with gravel or very stiff to hard sandy clay and sandy clay with gravel (alluvium). The mat should extend and bear at least 18 inches into the alluvial layer so therefore the total excavation depth will be 8 feet (to approximately Elevation 2 feet, NAVD88). A mat with a minimum thickness of 18-inches bearing at least 18 inches into undisturbed alluvium may be designed for an allowable dead plus live load bearing pressure of 4,000 pounds per square foot (psf). The dead plus live load value may be increased by 1/3 for total loads, including wind and/or seismic. In addition, the mat should be designed (by the structural engineer) so it is capable of spanning an unsupported, two-way, dimension of 7 feet.

If elastic analyses are used, we recommend the foundation be analyzed using a modulus of subgrade reaction ranging from 35 to 110 pounds per cubic inch (pci) with a design value of 55 pci for the static and dynamic load cases. No reduction or scaling is required. The modulus range is representative of the anticipated settlement under the building loads (with short and long term static loading being the lower part of the range and seismic loading being the higher). We recommend the analysis include the ranges in moduli and their effects on the reinforcing in the foundations as well as in checking the forces anticipated in the columns (up through the building) during a seismic event with the lower values used for foundation design and the higher for column design. After the mat analysis is completed, we should review the computed settlement and bearing pressure plots to check that the selected range of modulus values used is appropriate.

Static total and differential settlements of properly designed and constructed mat with typical column spacing should be less than one-inch and one-half inch, respectively.

Resistance to lateral forces can be obtained from passive pressure against the sides of foundation elements. Passive resistance may be calculated using a rectangular pressure of 1,000 psf if the footings or mat are cut neat and cast directly against undisturbed alluvium. If backfill is planned adjacent to the footings or mat (formed construction techniques), an equivalent fluid pressure of 250 pounds per cubic feet (pcf) should be used when calculating the passive resistance. Frictional resistance should be computed using a base friction coefficient of 0.35. If waterproofing will be used, this value may need to be reduced depending on recommendations by the waterproofing manufacturer. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

The footing excavations should be free of standing water, debris, loose or soft material prior to placing concrete. In addition, footing subgrade should be kept in a moist condition until concrete is poured. We should check the excavations prior to placement of reinforcing steel to confirm the exposed subgrade is suitable to support the design bearing pressures. If loose, overly-saturated, soft or undesirable soil is encountered in the excavations, it should be removed and the over-excavation(s) backfilled with lean or structural concrete.

8.2.2 Shallow Foundations on Drilled Displacement Columns (DDCs)

In lieu of over-excavation to 8 feet to reach the alluvium (bearing) layer, a reinforced concrete grid of footings or mat for the proposed development may be supported at-grade on the existing sandy fill soil which has been improved using ground improvement techniques. We judge a non-vibration, drilled displacement columns (DDCs) ground improvement method may be used to improve the loose to medium dense sandy fill and provide foundation support for the use of a shallow foundation system for the proposed structure. DDCs are drilled holes filled with concrete which are designed on a grid spacing below shallow foundations to transfer building loads through the sandy fill layer to the underlying more competent alluvium.

DDCs at the site should be used to support the proposed foundation elements including site walls and the slab(s) on grade (as designed by the project structural engineer) and should extend to depths achieving at least 7- to 10- feet of embedment or drill refusal in the dense to very dense clayey sand with gravel or very stiff to hard sandy clay layers as determined by the design build engineer and/or specialty contractor. Our assumption for this project is that the DDC method, no ground anchors for tension loads are required. In addition a compacted aggregate cushion consisting of crushed rock will be required below all footings; the DDC shall be installed to about 6 to 12 inches of the proposed base of footing allowing for the compacted engineered fill (cushion).

Furthermore, we recommend at least two load tests be performed (however, more may be required by the design build engineer) on DDC to verify loads can be supported with acceptable settlements (less than one inch).

Prior to commencement of the DDC installations including the load tests, Rollo & Ridley and the design team should review the proposed DDC design-build submittal by others. In return, we will prepare a review letter accepting the proposed approach as required by San Rafael and the County of Marin.

With the accepted and approved installation of DDC ground improvement, a shallow foundation system may be designed using an allowable dead plus live load of 5,000 pounds per square foot (psf) and limit total and differential settlements to 1 inch and ½ inch, respectively. This value may be increased by 1/3 for total loads, including wind and/or seismic. Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the footings and friction along their base. Passive resistance may be calculated using lateral pressures corresponding to an equivalent fluid weight of 250 pounds per cubic foot (pcf); the upper foot of soil should be ignored unless confined by a concrete slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30; this friction value assumes no reduction in the friction as a result of the waterproofing membrane (if used). These values include a factor of safety of about 1.5 and may be used in combination without reduction.

The footing excavations should be free of standing water, debris, loose or soft material prior to placing concrete. In addition, footing subgrade should be kept in a moist condition until concrete is poured. We should check the excavations prior to placement of reinforcing steel to confirm the exposed subgrade is suitable to support the design bearing pressures. If loose, overly-saturated, soft or undesirable soil is encountered in the excavations, it should be removed and the over-excavation(s) backfilled with lean or structural concrete.

8.3 Lowest Floor Slab

Even though the lowest level floor will be above the groundwater table, water and water vapor may occasionally be present within the subgrade soil. If water vapor transmission through the slab (or mat) is undesirable, the slab or mat should be waterproofed. As a minimum, we recommend the slab or mat be underlain by a capillary moisture break and vapor retarder. Waterproofing and vapor retarders are not equivalent systems. Waterproofing is designed to stop virtually all moisture transmission, while a vapor retarder can only reduce the amount and rate of moisture migration. The remainder of this section provides our recommendations for a capillary moisture break and vapor retarder system.

Where water vapor transmission through the floor slab is undesirable (e.g., where floor covering will be placed), a capillary moisture break and a water vapor retarder may be installed beneath the floor. A capillary moisture break and vapor retarder are generally not required below parking slabs-on-grade because there is sufficient air circulation to limit condensation of moisture on the slab surface.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. At the discretion of the project structural engineer, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand, if used, overlying the membrane should be dry at the time concrete is placed. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured.

Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

8.4 Basement Walls and Retaining Walls

Basement walls and retaining walls should be designed to resist lateral earth pressures and surcharges. Restrained walls (such as below grade basement walls or elevator pit walls or sunken utility rooms/vaults) should be designed for at-rest soil pressures, while unrestrained walls (such as landscaping retaining walls), which are free to rotate at the top may be designed for active pressures.

8.4.1 Basement Walls

Basement walls should be supported on foundations designed using the appropriate design values presented in the foundation support section of this report (Section 8.2).

Basement walls include all below grade walls associated with the structure (including elevator pits and sunken utility rooms/vaults, etc.). Basement walls should be designed to resist lateral pressures created by the soil and adjacent surcharges. In addition, because the site is in a seismically active area, all below grade walls should be designed to resist pressures associated with seismic forces. New research on basement wall pressures has been recently published by the University of California, Berkeley. This research reached two important conclusions. These are: (1) the seismic increment increases with depth and can be reasonably approximated by an equivalent fluid pressure (triangular distribution) and (2) the seismic increment occurs under the active condition. Using the procedure outlined in the SEAOC 2010 Convention Proceedings for Seismic Earth Pressures on Deep Building Basements, we recommend the following pressures presented in Table 4 be used in design for permanent basement walls with level backfill.

TABLE 4

Design Pressures for Basement Walls

Loading Condition	Restrained Walls	
	Drained Condition	Undrained Condition
Static	At-rest pressures corresponding to an equivalent fluid weight of 60 pounds per cubic foot (pcf)	At-rest pressures corresponding to an equivalent fluid weight of 90 pounds per cubic foot (pcf)
Dynamic	Greater of either at-rest condition (60 pcf) or active (40 pcf) plus a seismic pressure increment of 20 pcf (equivalent fluid weight, triangular distribution)	Greater of either at-rest condition (90 pcf) or active (80 pcf) plus a seismic pressure increment of 20 pcf (equivalent fluid weight, triangular distribution)

If traffic loads are expected within 10 feet of the walls, an additional design load of 100 psf should be applied to the upper ten feet of the wall.

The lateral earth pressures given for drained conditions assume the walls are properly backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the walls. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the walls. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or clean ¾-inch drainrock wrapped in filter fabric (Mirafi 140N or equivalent). We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. A thicker drainage paneling (such as Hydroduct® Coil 600 or equivalent) may be used in lieu of a PVC pipe surrounded by gravel. The collector pipes should drain to a suitable discharge location (sump). If walls are not backdrained (to avoid having to install drainage pipes and a sump in the basement) the lateral earth pressures given for undrained conditions in Table 4 should be used.

Dampness and discoloration on the walls should be expected due to natural percolation of rain water, irrigation, broken or leaking utilities or other water introduced behind the walls. If this is not acceptable, the walls should be waterproofed. The waterproofing should be installed directly behind the wall (sandwiched between the wall and drainage panel). Final waterproofing recommendations should be determined by the architect and/or waterproofing consultant.

8.4.2 Retaining Walls

Lateral earth pressures on freestanding retaining walls (if necessary for landscaping on this project) will depend upon the steepness of the slopes behind the walls. Recommended lateral earth pressures for walls retaining soil that are free to rotate for various slope inclinations are presented in Table 5.

TABLE 5
Exterior Retaining Wall Pressures

Slope Angle (degrees)	Walls Free to Rotate (active pressure)
0	40 pcf
27 (2:1)	60 pcf
34 (1.5:1)	80 pcf

Where traffic loads are expected within 10 feet of the walls, an additional design load of 100 psf should be applied to the upper ten feet of the wall.

It should be noted that retaining walls designed to rotate (using active pressures), will move outward near the top of the wall over time (over several months), causing minor concrete cracking to the wall and ground settlement of the retained soil near the top of the wall. Alternatively, walls can be designed to be restrained to limit top deflection by applying at-rest pressures as discussed in Basement Walls section of this report.

The lateral earth pressure given assumes the walls are properly backdrained to prevent the buildup of hydrostatic pressure. The backdrains may consist of prefabricated drainage panels (Miradrain 6000 or equivalent) placed against the back of the wall. The drainage panels should extend down to a collector pipe consisting of four-inch-diameter, perforated PVC pipe surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140N or equivalent). A prefabricated Hydroduct® Coil 600 (or equivalent) may be used in lieu of a PVC pipe. The collector pipes should drain to a suitable discharge location.

Another acceptable alternative is to backdrain the wall with crushed rock material at least one foot wide extending down to the base of the wall. A perforated PVC pipe should be placed at the bottom of the drain to collect water and transmit it to a suitable discharge point. The pipe and crushed rock should be surrounded by filter fabric. The top of the gravel should be capped with at least 18 inches of clayey soil or a concrete v-ditch sloping to a discharge point.

Alternatively, weep holes at the base of the wall could be used to drain water collected in the drainage paneling and/or crushed rock from the back of the wall. Weep holes should be spaced no greater than 4 feet apart and be a minimum of 3 inches in diameter. The back of the weep hole should be covered with filter fabric to prevent retained soil from being transported through the weep holes. Weep holes continue to drain after rainfall stops. If hardscape is planned below the walls, it should be noted that it may remain wet. The design team and owner should discuss the appropriateness of weep holes and introducing water onto flatwork below the walls.

8.5 Slope Cuts

Temporary excavations deeper than 4-1/2 feet should be shored or sloped for safety in accordance with CAL-OSHA standards. Temporary slope cuts made during construction should be no steeper than 1 to 1 (horizontal to vertical) and not greater than 5 feet in height, except near the toe of the slope, where a vertical cut with a maximum height of three feet can be excavated (adjacent to the outside face of the foundation elements). If permanent slope cuts are required for landscaping, they should be no steeper than 3 to 1 and not greater than 3 feet in height, unless approved by the geotechnical engineer. All slope cuts should start at least 3 feet away from the property line. In addition, the geotechnical engineer should review the grading or landscaping plans to evaluate the safety of the proposed slope cuts and whether they impact any of the neighboring improvements.

8.6 Shoring

Shoring will likely be required to retain the surrounding sidewalks and streets if the deepened mat foundation option is selected to support the building. Shoring may also be required for any proposed sidewalk vaults or other below grade rooms.

Where space is limited and a sloped cut is not feasible, we judge that a cantilevered shoring system would be the most practical form of shoring. The shoring wall may be designed for active pressure of 40 pcf. Additional surcharges pressure from adjacent street traffic or construction equipment should be added by the shoring engineer on a case-by-case basis. Lateral earth pressures may be resisted by a passive resistance (as detailed in the Foundation section) against the embedded portion of the piers and applied over a width of three times the pier diameter (shape factor equal to three). We anticipate that a temporary soldier beam and wood-lagging shoring system would be most appropriate technique to shore the excavation. The soil between the piers should be retained using timber lagging. Wood lagging should be installed by hand, one board at a time, from the top down with an emphasis placed on not allowing the soil behind to cave. Any small voids behind the lagging should be immediately filled with soil and large voids filled to cement grout pumped through drilled holes in the lagging.

8.7 Flatwork

We understand flatwork consisting of concrete strips (pavers) and exposed concrete aggregate may be installed in open space areas. Pavers should be placed on two inches of sand overlying Class 2 aggregate base. Typically the required thickness of aggregate base is 4 to 6-inches for pavers receiving only foot-traffic and 12 inches for driveways not subjected to repeated truck loading. The final thickness selected for this project should be selected in conjunction with the project civil engineer.

If native soil is present at subgrade, the upper six inches should be moisture-conditioned near optimum and compacted to at least 95 percent relative compaction. Aggregate base should conform to current Caltrans Standard Specifications. All aggregate base should be compacted to at least 95 percent relative compaction.

8.8 Drainage and Infiltration

Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly adjacent to structures, or on driveways. Surface runoff should be directed away from foundations to an acceptable City outlet or a slope dissipation system designed by the project civil engineer. In addition, all roofs should have gutters and downspouts that are connected to a dissipater or acceptable outlet.

We understand that a portion of the project may include permeable pavers, porous pavement, tree wells, or other infrastructure that has infiltration capabilities. The near-surface soil consists of primarily of fill. The fill material is typically loose to medium dense clayey sand and sandy clay that contains organics. Using the United States Department of Agriculture, Natural Resources Conservation Service National Engineering Handbook, we judge the soil should be classified as Hydrologic Soil Group (HSG) D (the lowest category), having an infiltration rate of approximately 0.14 inches/hour. A higher value may be obtained if percolation tests are performed. Infiltration rates may vary by orders of magnitude in fill and cannot be completely predicted by geotechnical investigations (even if percolation tests are performed); therefore, appropriate precautions should be taken in case the estimated infiltration rate over predicts seepage rates. Precautions (to prevent flooding) should include, but are not limited to, overflow drains, backup gutters and sumps/pumps, as appropriate.

8.9 Seismic Design

The San Francisco Bay Area is a seismically active region and the structure is likely to experience periodic minor earthquakes and possibly a major earthquake (Richter magnitude greater than 7) on one of the nearby active faults. Therefore, the seismic design should be in accordance with the provisions of 2016 California Building Code (CBC) including the following:

- Risk Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.500g and 0.600g, respectively.
- Site Class D
- Site Coefficients; F_a=1.0, F_v=1.5
- MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.500g and 0.900g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.000g and 0.600g, respectively.

9.0 ADDITIONAL GEOTECHNICAL SERVICES

Because portions of the site are covered by existing structures (which will be demolished), variations between expected and actual soil conditions may be found in areas not previously explored including the northeast, northwest and southwest corners of the site. Consequently it may be necessary to perform additional field explorations after demolition of the existing structures and prior to foundation construction depending on the foundation system selected. These additional studies will allow us to verify the areas of the site where access was not obtainable and make supplemental recommendations to this report if deemed necessary.

Prior to construction, we should review the final foundation plans and specifications to check that they are in general conformance with the intent of our recommendations. During construction, we should observe the installation and load test of the drilled displacement columns (DDC), if used, site grading and excavation of slope cuts, shoring, if used, installation of shallow foundations, placement and compaction of fill, slab subgrade preparation as necessary. We will, in turn, compare actual to anticipated soil conditions, and check the contractor's work conforms to the geotechnical aspects of the plans and specifications as outlined in the CBC.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the existing geotechnical conditions and available subsurface data. Actual subsurface conditions may vary. If any variations or unforeseen conditions are encountered during construction, or if the proposed construction will differ from that which is described in this report, Rollo & Ridley, Inc. should be notified so that supplemental recommendations can be made.

Our firm has prepared this report for the exclusive use of our client and their representatives on this project in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty is expressed or implied. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by our firm during the construction phase in order to evaluate compliance with our recommendations. If we are not retained for these services, the client must assume Rollo & Ridley's responsibility for potential claims that may arise during or after construction.

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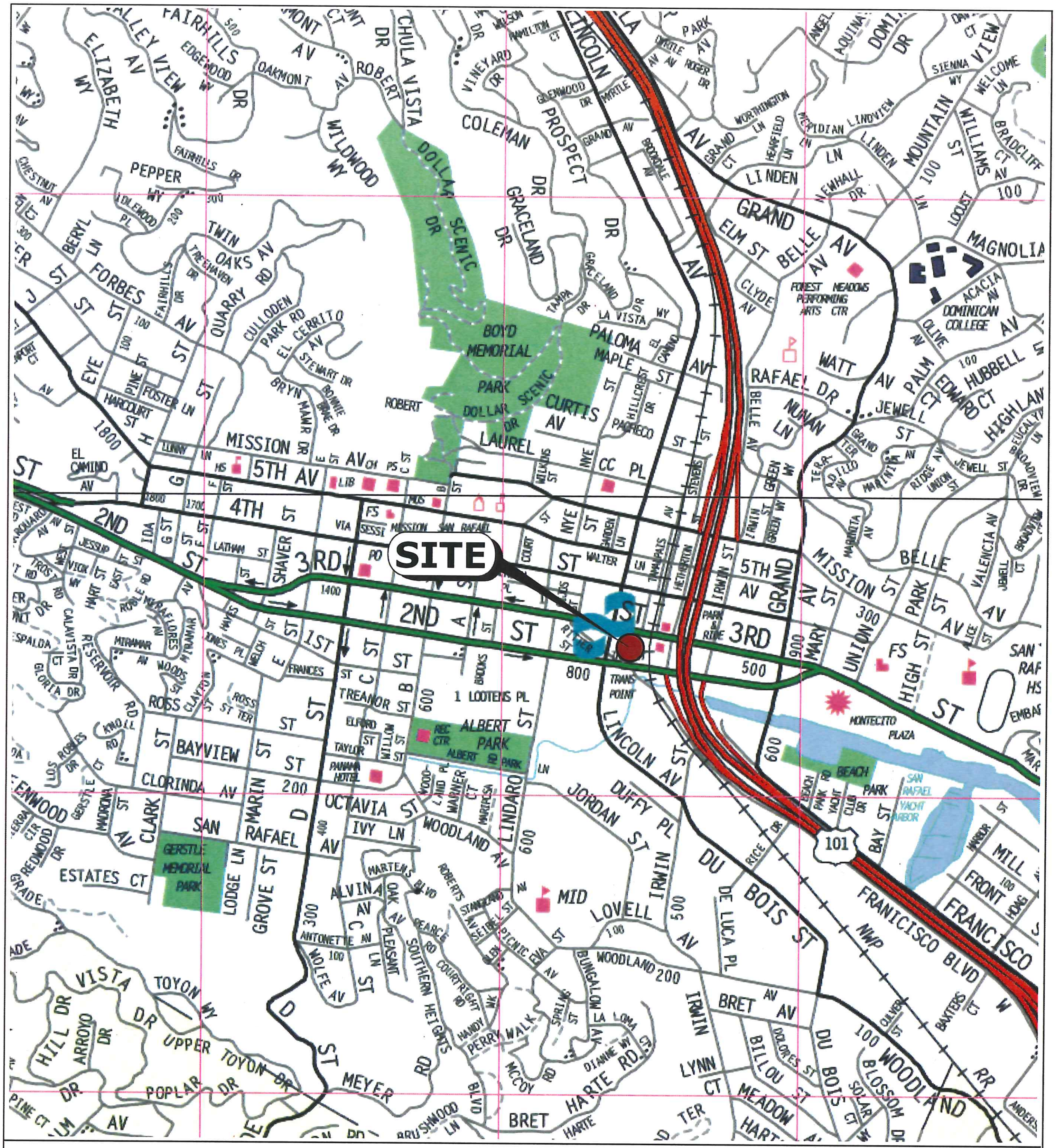
FIGURES



Base map: The Thomas Guide
Marin County

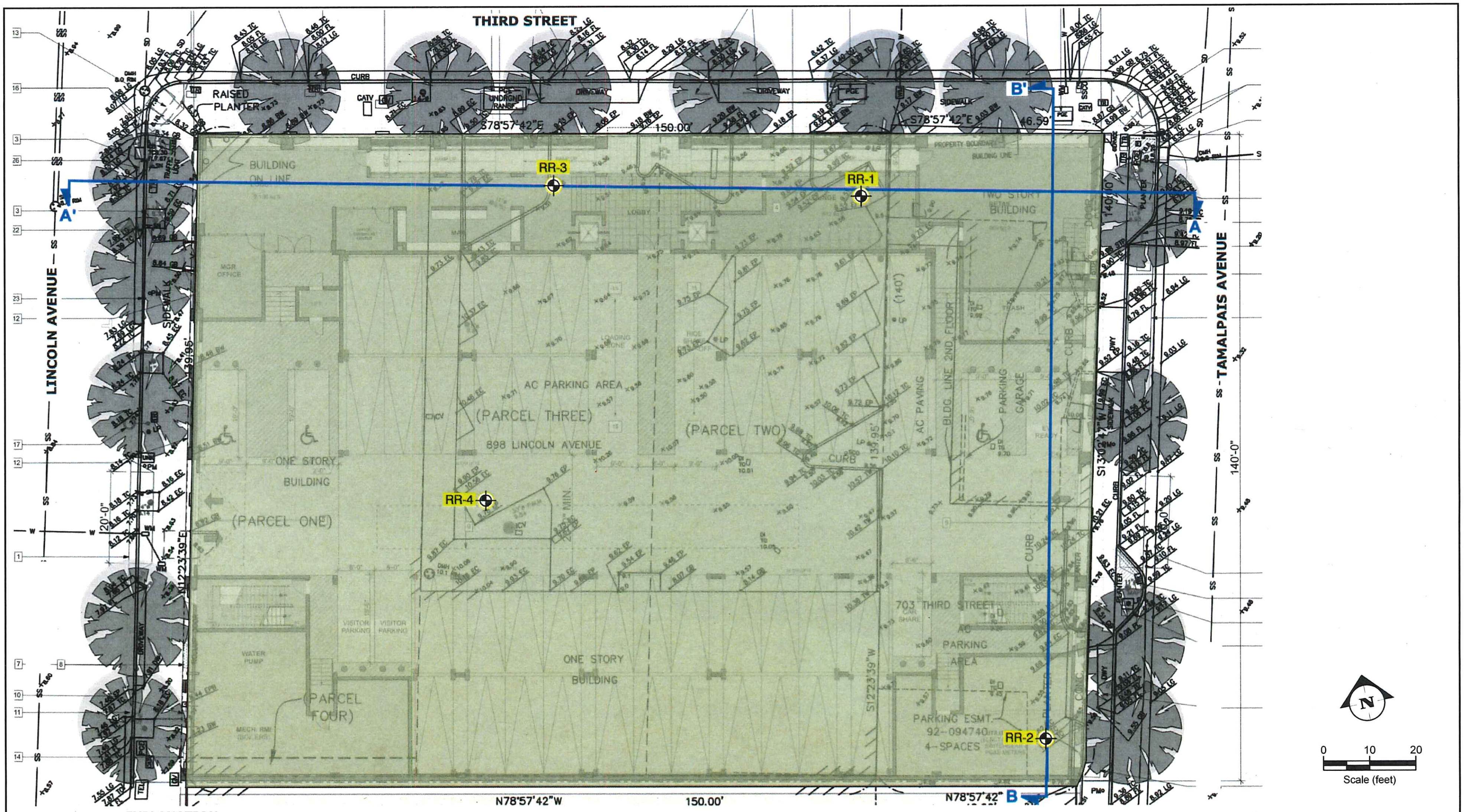
N

0 1/4 1/2 mile
Approximate Scale

ROLLO & RIDLEY
GEOTECHNICAL ENGINEERS & SCIENTISTS

SITE LOCATION MAP
703 THIRD STREET
San Rafael, California

PROJECT No.	1570.1
DATE	05/25/18
FIGURE	1



EXPLANATION

RR-1 ⊕ Approximate location of boring performed by Rollo & Ridley Inc. on May 22 and 23, 2018

🟩 Proposed 703 3rd Street Development (Ground Floor Level)

A ⌴ A' Approximate location of Idealized Subsurface Profile A-A'

Reference: Base map from a drawing titled "Existing Conditions Plan" by Oberkamper & Associates dated March 13, 2018 and "703 Third Street - 1st Fllor Plan" by Van Meter Villiams Pollack dated March 20, 2018. Spot elevations and existing conditions (hatched) based on A Boundary & Topographic Survey prepared by Oberkamper & Associates dated 3/13/2018.

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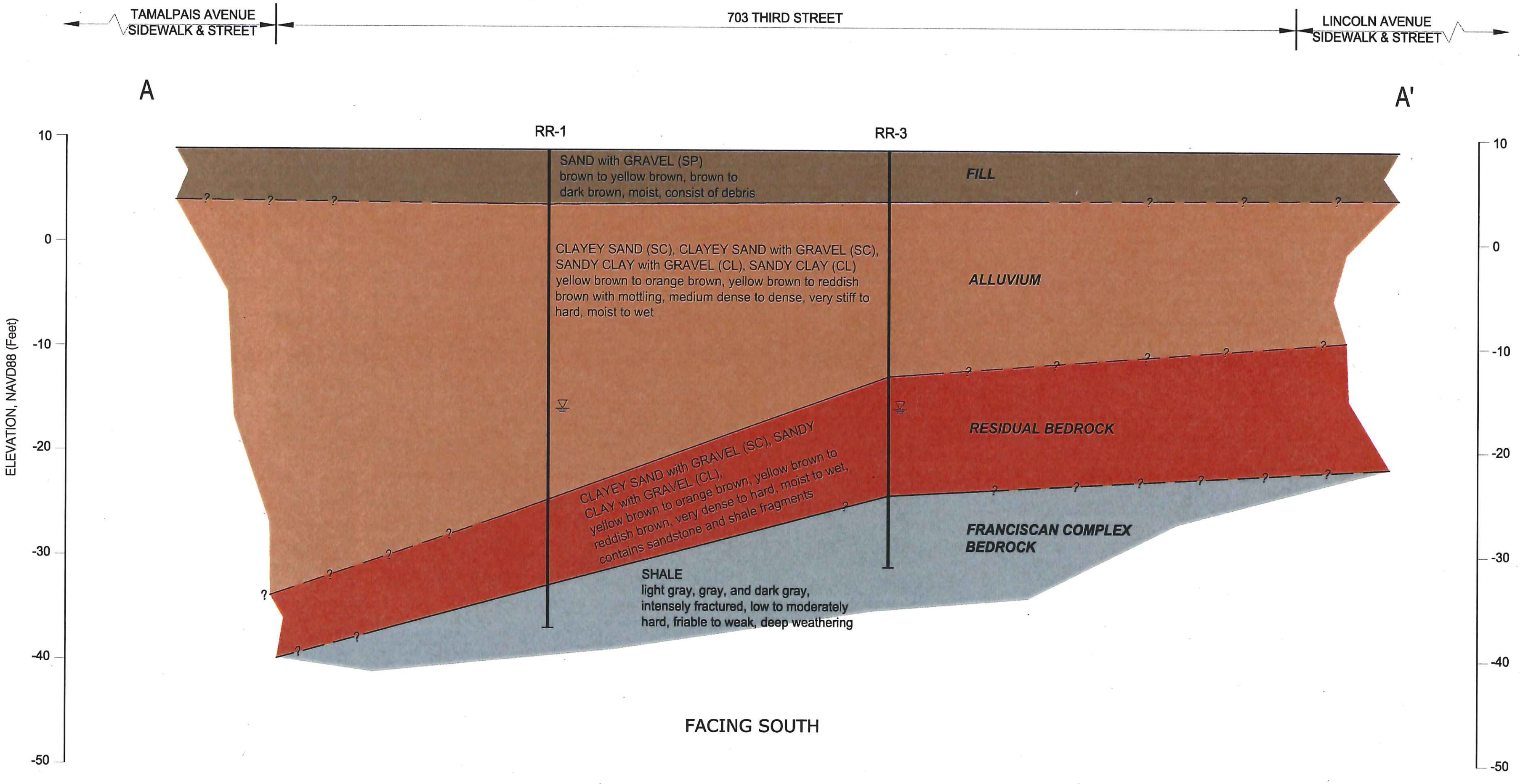
SITE PLAN
703 THIRD STREET
San Rafael, California

PROJECT No.	1570.1
DATE	06/28/18
FIGURE	2



10 Feet
0
0
20 Feet
Approximate Scale

Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.
2. Unstabilized groundwater levels shown are based on observations made during drilling which may vary, see report for further details.
3. Elevations in feet, NAVD88 based on Boundary & Topographic Survey performed by Oberkamper & Associates dated March 13, 2018.

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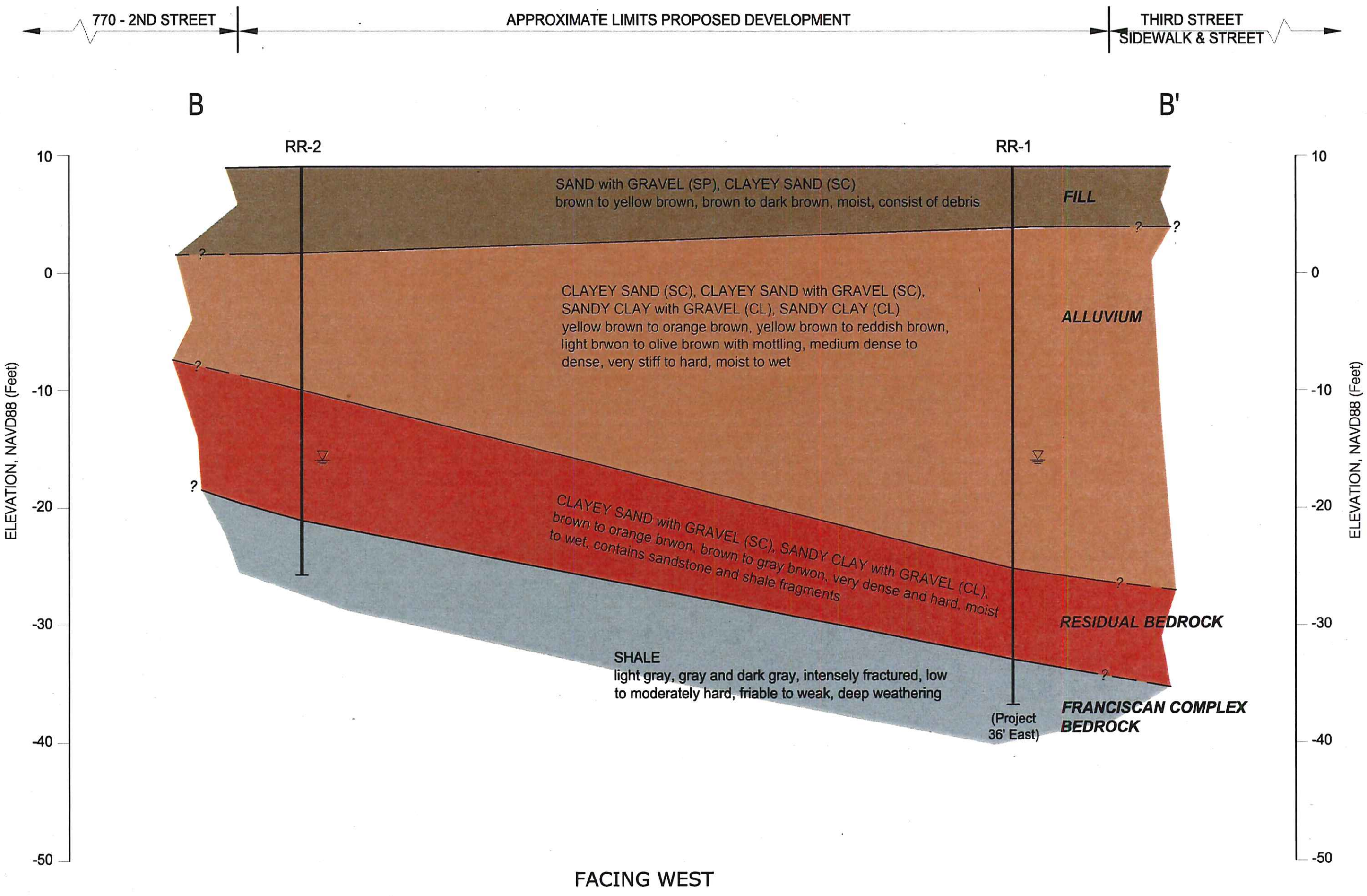
IDEALIZED SUBSURFACE
PROFILE A-A'
703 THIRD STREET
San Rafael, California

PROJECT No.	1570.1
DATE	07/03/18
FIGURE	3



Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.
2. Unstabilized groundwater levels shown are based on observations made during drilling which may vary, see report for further details.
3. Elevations in feet, NAVD88 based on Boundary & Topographic Survey performed by Oberkamper & Associates dated March 13, 2018.

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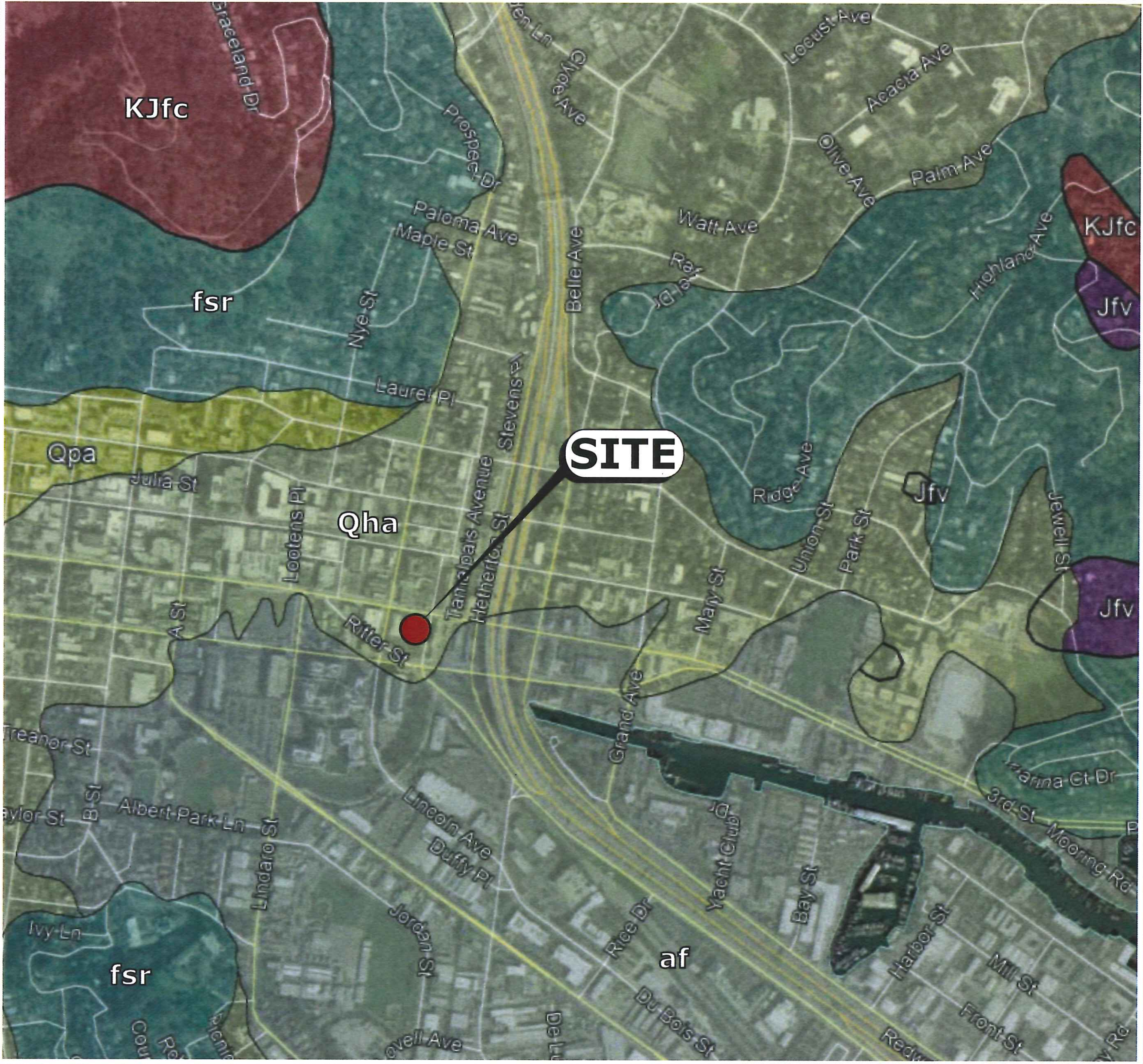
IDEALIZED SUBSURFACE
PROFILE B-B'
703 THIRD STREET
San Rafael, California

PROJECT No.	1570.1
DATE	07/03/18
FIGURE	4



Base map: Goolge Earth with U.S. Geological Survey (USGS), Marin County, 2017.

EXPLANATION

af	**Artificial Fill**
Qha	**Alluvium (Holocene)**
Qpa	**Alluvium (Pleistocene)**
fsr	**Franciscan Complex melange (Eocene, Paleocene, and (or) Late Cretaceous)**
KJfc	**Franciscan Complex chert (Early Cretaceous and (or) Late Jurassic)**
Jfv	**Franciscan Complex volcanic rocks (Jurassic)**

Geologic contact:
dashed where approximate and dotted where concealed, queried where uncertin

N

0 1,000 2,000 Feet

Approximate Scale

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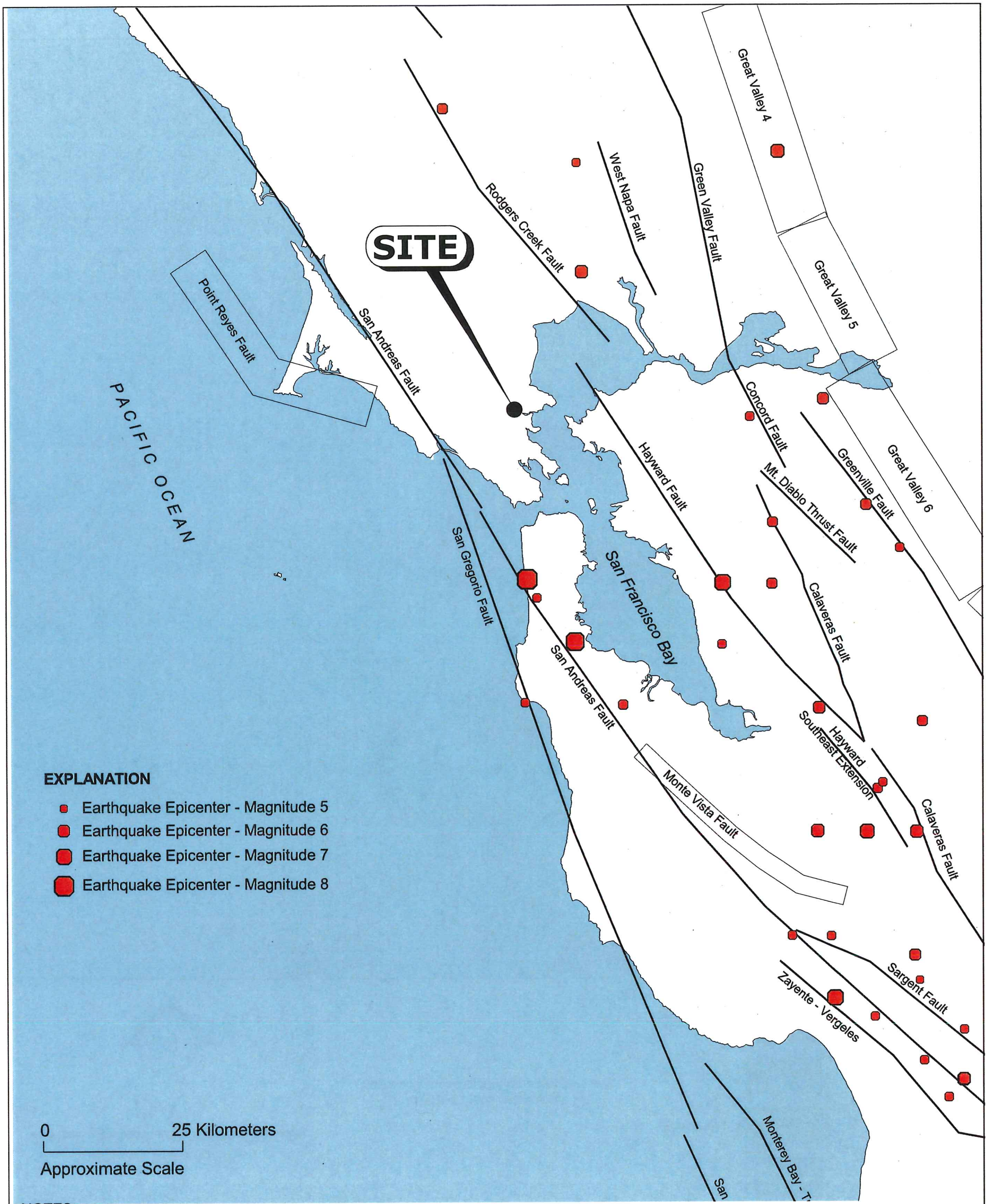
MAP OF REGIONAL GEOLOGY
703 THIRD STREET
San Rafael, California

PROJECT No. 1570.1
DATE 05/28/18
FIGURE 5



NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

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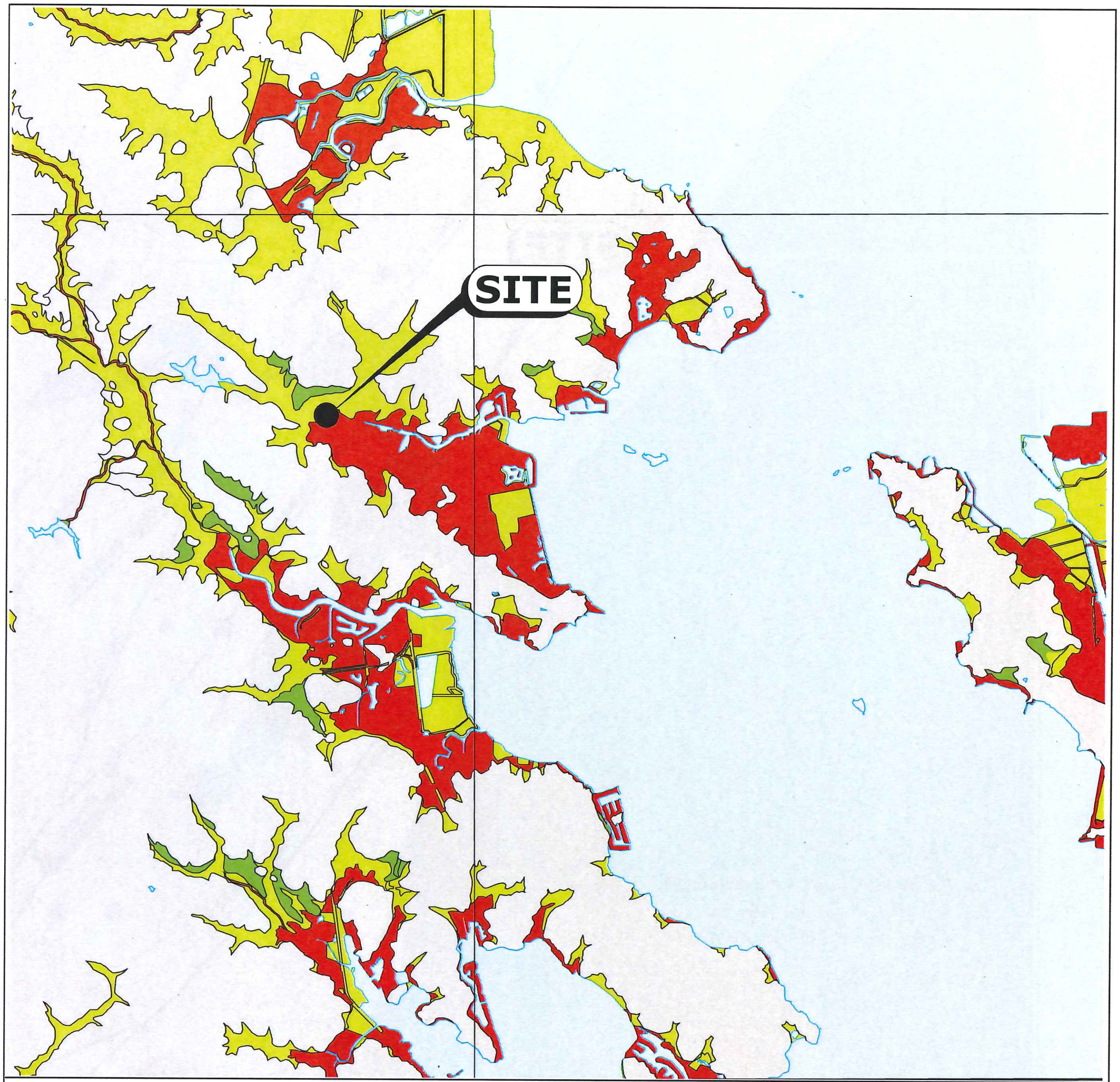
MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

703 THIRD STREET
San Rafael, California

PROJECT No. 1570.1
DATE 05/28/18
FIGURE
6



LIQUEFACTION SUSCEPTIBILITY

🟥	VERY HIGH
🟧	HIGH
🟨	MODERATE
🟩	LOW
⬜	VERY LOW

Lines

———————

Contact, dashed where location uncertainty is greater than about +- 100 m.

N

0 1/4 1/2 mile

Approximate Scale

Reference:
Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, 2006

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LIQUEFACTION SUSCEPTIBILITY MAP
703 THIRD STREET
San Rafael, California

PROJECT No.	1570.1
DATE	05/28/18
FIGURE	7

APPENDIX A

Logs of Borings and Classification Charts

PROJECT:	**703 THIRD STREET** San Rafael, California	**Log of Boring RR-1** PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 5/22/18 Date finished: 5/22/18

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: C. Tan

LABORATORY TEST DATA

MATERIAL DESCRIPTION

Approximate Ground Surface Elevation: 9.5 Feet[2]

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						2 inches of asphalt							
						8 inches of baserock	FILL						
1–4					SP	SAND with GRAVEL (SP) brown to yellow brown, medium dense, moist	FILL						
5–6	S&H	■	12 17 22	31	CL	SANDY CLAY (CL) yellow brown to orange brown with mottling, very stiff to hard, moist	ALLUVIUM				59	20.4	101
10–11	S&H	■	12 22 29	41	SC	CLAYEY SAND with GRAVEL (SC) yellow brown to reddish brown with olive mottling, dense, moist	ALLUVIUM				36	14.3	115
15–16	SPT	◪	8 11 20	37	CL	SANDY CLAY with GRAVEL (CL) brown to orange brown, hard, moist	ALLUVIUM						
20–21	S&H	■	9 13 15	22	CL	very stiff	ALLUVIUM						
24						∇	ALLUVIUM						
25–26	SPT	◪	12 16 18	41	CL	SANDY CLAY (CL) yellow brown to orange brown, hard, moist	ALLUVIUM						
27–30					CL		ALLUVIUM						

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Project No.: 1570.1

Figure: A-1a

PROJECT: **703 THIRD STREET**
San Rafael, California

Log of Boring RR-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	SPT	◪	10 16 12	34		SANDY CLAY with GRAVEL (CL) yellow brown to reddish brown, hard, wet	ALLUVIUM						
32					CL								
33													
34						SANDY CLAY with GRAVEL (CL) brown to orange, brown, hard, wet, with sandstone fragments	RESIDUAL BEDROCK						
35													
36	SPT	◪	18 21 32	64									
37													
38					CL								
39													
40						sandstone and shale fragments							
41	SPT	◪	36 48 32	96									
42													
43						SHALE light gray to gray, intensely fractured, low to moderately hard, friable to weak, deep weathering	FRANCISCAN COMPLEX BEDROCK						
44													
45													
46	SPT	◪	27 41 50/5"	121									
47													
48													
49													
50													
51													
52													
53													
54													
55													
56													
57													
58													
59													
60													

Boring terminated at a depth of 46.4 feet below ground surface.
Boring backfilled with cement grout..
Unstabilized groundwater encountered at a depth of 24 feet during drilling.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.2, respectively, to account for sampler type and hammer energy.

[2] Elevations in feet based on NAVD 88.

ROLLO & RIDLEY
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Project No.: 1570.1

Figure: A-1b

PROJECT: **703 THIRD STREET**
San Rafael, California

Log of Boring RR-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/22/18
Date finished: 5/22/18
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic Hammer
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: C. Tan

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value^1		Approximate Ground Surface Elevation: 9.6 Feet^2		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches of asphalt							
						8 inches of baserock							
1					SC	CLAYEY SAND (SC) brown to dark brown, medium dense, moist, consist of debris	FILL						
6	S&H	■	4 5 7	10		dense, moist					36	17.1	111
7					CL	SANDY CLAY (CL) light brown to olive brown with mottling, stiff to very stiff, moist	ALLUVIUM						
11	S&H	■	10 16 20	29							60	22.5	100
16	SPT	◩	10 10 12	26									
20					SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, with sandstone fragments	RESIDUAL BEDROCK						
21	SPT	◩	29 50 50/3"	120/ 9"									
24					CL	∇ SANDY CLAY with GRAVEL (CL) brown to gray brown, hard, moist to wet, with shale fragments							
26	SPT	◩	13 23 33	67									
30													

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Project No.: 1570.1
Figure: A-2a

PROJECT: **703 THIRD STREET**
San Rafael, California

Log of Boring RR-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT	◩	27 50/6"	64/6"		**SHALE** gray to dark gray, intensely fractured, low to moderately hard, friable to weak, deep weathering	**FRANCISCAN COMPLEX BEDROCK**						
31													
32													
33													
34													
35	SPT	◩	50/3"	60/3"									
36													
37													
38													
39													
40													
41													
42													
43													
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Boring terminated at a depth of 35.3 feet below ground surface.
Boring backfilled with cement grout.
Unstabilized groundwater encountered at a depth of 24 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.2, respectively, to account for sampler type and hammer energy.
[2] Elevations in feet based on NAVD 88.

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Project No.:	1570.1
Figure:	A-2b

RR 1570.1.GPJ TR.GDT 7/3/18

PROJECT: **703 THIRD STREET**
San Rafael, California

Log of Boring RR-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/23/18
Date finished: 5/23/18
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic Hammer
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: C. Tan

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 9.5 Feet[2]		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches of asphalt							
						8 inches of baserock							
1					SP	SAND with GRAVEL (SP) brown to dark brown, medium dense, moist	FILL						
6	S&H	■	5 10 20	24	SC	CLAYEY SAND with GRAVEL (SC) brown to yellow brown with mottling, medium dense, moist	ALLUVIUM				45	18.4	109
11	S&H	■	9 10 15	20		with trace silt and decreased gravel							
16	S&H	■	10 16 24	32		increased gravel							
21	SPT	◩	12 18 18	43	CL	SANDY CLAY with GRAVEL (CL) brown to orange brown, hard, moist	RESIDUAL BEDROCK						
24						∇							
26	SPT	◩	8 11 19	36		with fine sandstone fragments							
30													

RR 1570.1.GPJ TR.GDT 7/3/18

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Project No.: 1570.1
Figure: A-3a

PROJECT: **703 THIRD STREET**
San Rafael, California

Log of Boring RR-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ 6"	SAMPLES SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		10 12 25	30		SANDY CLAY with GRAVEL (CL) (continued) very stiff to hard, wet	RESIDUAL BEDROCK						
32													
33					CL								
34													
35	SPT		50/4"	60/4"		SANDSTONE brown to olive brown, intensely fractured, low hardness, friable to weak, deep weathering	FRANCISCAN COMPLEX BEDROCK						
36													
37													
38													
39													
40													
41	SPT		44 42 50/4"	128		SHALE gray to dark gray, intensely fractured, low to moderately hard, friable to weak, deep weathering							
42													
43													
44													
45													
46													
47													
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60													

Boring terminated at a depth of 41.3 feet below ground surface.
Boring backfilled with cement grout.
Unstabilized groundwater encountered at a depth of 24 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.2, respectively, to account for sampler type and hammer energy.

[2] Elevations in feet based on NAVD 88.

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Project No.: 1570.1

Figure: A-3b

PROJECT: **703 THIRD STREET**
San Rafael, California

Log of Boring RR-4

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 5/23/18 Date finished: 5/23/18

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: C. Tan

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 9.8 Feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches of asphalt						
1						15 inches of baserock						
2						CLAYEY SAND (SC) olive brown to yellow brown, medium dense, moist						
3												
4					SC	trace organics FILL						
5												
6	S&H	■	4 5 9	11						55	17.3	111
7						CLAYEY SAND (SC) olive brown to yellow brown with mottling, dense, moist						
8					SC							
9												
10												
11	S&H	■	15 26 40	53		CLAYEY SAND with GRAVEL (SC) yellow-brown with rust-colored mottling, very dense, moist				52	19.0	106
12												
13						ALLUVIUM						
14												
15												
16	SPT	◪	9 15 20	42	SC	dense						
17												
18												
19												
20												
21	SPT	◪	12 19 21	48								
22						SANDY CLAY with GRAVEL (CL) reddish brown, hard, moist to wet						
23												
24						▽						
25						RESIDUAL BEDROCK						
26	SPT	◪	12 16 24	48	CL	rust-colored mottling or orange brown						
27												
28												
29												
30												

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Project No.: 1570.1

Figure: A-4a

PROJECT: **703 THIRD STREET**
San Rafael, California

Log of Boring RR-4

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT	◩	13 14 22	43	SC	CLAYEY SAND with GRAVEL (SC) yellow brown, dense, wet						
32												
33												
34												
35						very dense, with sandstone fragments						
36	SPT	◩	20 27 31	70		RESIDUAL BEDROCK						
37												
38												
39												
40												
41	SPT	◩	19 32 50/2"	98/8"		SHALE gray to dark gray, intensely fractured, low to moderately hard, friable to weak, deep weathering						
42	SPT	▬	50/1"	60/1"		FRANCISCAN COMPLEX BEDROCK						
43												
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Boring terminated at a depth of 42.1 feet below ground surface.
Boring backfilled with cement grout.
Unstabilized groundwater encountered at a depth of 24 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.2, respectively, to account for sampler type and hammer energy.

[2] Elevations in feet based on NAVD 88.

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Project No.: 1570.1

Figure: A-4b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
◩	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
☒	Disturbed sample
⊡	Sampling attempted with no recovery
◫	Core sample
◙	Analytical laboratory sample
▥	Sample taken with Direct Push sampler
▥	Sonic

▽	Unstabilized groundwater level
▼	Stabilized groundwater level

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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CLASSIFICATION CHART
703 THIRD STREET
San Rafael, California

PROJECT No. 1570.1
DATE 06/28/18
FIGURE **A-5**

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

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PHYSICAL PROPERTIES CRITERIA
FOR ROCK DESCRIPTIONS
703 THIRD STREET
San Rafael, California

PROJECT No. 1570.1
DATE 06/28/18
FIGURE
A-6

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