ROLLO & RIDLEY
GEOTECHNICAL ENGINEERS & SCIENTISTS

GEOTECHNICAL INVESTIGATION
7 Jose Patio
Stinson Beach, California

Stinson Sandpiper, LLC
Stinson Beach, California

October 14, 2021
Project No. 1714.1

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7 Jose Patio
Stinson Beach, California 94970

Subject: Geotechnical Investigation
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Dear Madams & Sirs:

This report presents the results of the geotechnical investigation performed by Rollo & Ridley, Inc. for the proposed new single-family residence at 7 Jose Patio in Stinson Beach, California. Additional copies of this report have been distributed as indicated at the end of this report. The property is at the west end of the Jose Patio cul-de-sac and is bound by private properties to the north and south and the bluff adjacent to the beach and Pacific Ocean to the west in Stinson Beach as shown on the Site Location Map, Figure 1.

The services described in this report were performed in accordance with our proposal dated July 7, 2021 and the professional services agreement executed on August 2, 2021. Conclusions and recommendations presented herein are based on: 1) discussions and correspondence with Federico Engel & Jay Montes of Butler Armsden Architects (project architects), and Bob Reed of GFDS Engineers (project structural engineers, 2) Schematic Drawings prepared by Butler Armsden Architects dated May 3, 2020, a Topographic Map of the Property Prepared By CSW / Stuber-Stroeh Engineering Group, Inc. dated August 3, 2020 and available subsurface information from sites in the vicinity, 3) a site specific field investigation and detailed engineering analysis, and 4) our experience with other projects in the vicinity of the site.

The lot is flag-shaped with the rectangular portion of the lot having maximum plan dimensions of approximately 75 feet by 95 feet. The site is relatively flat sloping gently up from east to west and is occupied by a two-story, single-family residence. According to the topographic survey site grades vary from approximately Elevation 11.2 Feet (NAVD88)[1] at the entry driveway at the east property line to approximately Elevation 15.5 Feet at the northwest corner of the site.

We understand the existing site improvements (including the residence) will be demolished and removed. The proposed plans include the construction of a new, two-story single-family residence with approximately 3,777 square feet of interior space with outdoor living spaces, along with paving, landscaping, site walls and additional site improvements. The design includes raising the main level approximately eight feet above adjacent site grades.

[1] Elevations presented in this report are in Feet based on NAVD88 as presented on a survey prepared by CSW / Stuber-Stroeh Engineering Group, Inc. dated August 3, 2020

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The focus of our investigation was to determine the depth to competent soils and the properties of the underlying soil so that conclusions and recommendations regarding the foundation and other geotechnical design criteria for the proposed development of the property could be made, as necessary.

The existing site conditions and footprint of the proposed residence are presented on the Site Plan, Figure 2.

SCOPE OF SERVICES

As outlined in our proposal, our scope of services included exploring the location of the proposed improvements, performing engineering analyses, and developing conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- appropriate foundation type(s) for the proposed residence
- design criteria for the recommended foundations
- estimates of foundation settlement
- slab on grades
- site seismicity and seismic hazards
- site grading, including fill and compaction criteria
- landscape retaining wall design criteria
- pavement & hardscape design criteria
- 2019 California Building Code (CBC) seismic criteria
- construction considerations

During our investigation, we consulted with members of the design team and provided information as it became available.

FIELD INVESTIGATION

To explore the subsurface conditions, we logged the conditions revealed in two borings drilled at the site on August 26, 2021. Prior to beginning our field investigation, we obtained a drilling permit from the Marin County Environmental Health Services (MCEHS). We also contacted Underground Service Alert (USA), as required by law before commencing our field exploration.

The locations of the borings are shown on the Site Plan, Figure 2. The borings, designated as RR-1 and RR-2, were drilled to depths of approximately 31.5 to 51.5 feet below the existing ground surface. The borings were drilled using a truck-mounted drill rig equipped

with hollow-stem augers operated by HEW Drilling of East Palo Alto, California. The boring locations were determined by measuring the distances from known points on the site survey and should be considered approximate.

Our field engineer logged the soil encountered and obtained samples of the material encountered for visual classification and laboratory testing. The logs of the borings RR-1 and RR-2 are presented in Appendix A as Figures A-1 and A-2. The soil encountered was classified in accordance with the soil classification system presented in Appendix A as Figure A-3.

Soil samples were obtained using two different types of samplers as follow:

- Sprague and Henwood (S&H) sampler with a 3.0-inch outside and 2.43-inch inside diameter

- Standard Penetration Test (SPT) sampler with a 2.0-inch outside and 1.5-inch inside diameter

The sampler types were chosen on the basis of soil type and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the density of sandy soil.

Both the S&H and SPT samplers were driven with a 140-pound, automatic hammer falling about 30 inches. Where the S&H sampler was used, the blow counts required to drive the sampler the final 12 inches of an 18-inch drive were corrected, using a factor of 0.8 to approximate SPT blow counts. Where the SPT sampler was used, the blow counts required to drive the sampler the final 12 inches of an 18-inch drive were corrected, using a factor of 1.2 to calculate SPT blow counts. Both the actual and converted blow counts are shown on the boring logs.

Upon completion of drilling, the borings were backfilled as required by Marin County Guidelines.

Representative samples of the soil obtained from the borings were collected and reviewed in our office to correlate soil properties and to evaluate engineering properties of the soil at the site. Select samples were sent to the laboratory for testing and the results of the lab tests are presented on the boring logs in appendix A.

SUBSURFACE CONDITIONS

As presented on Figure 3, Idealized Subsurface Profile, A-A', the property is underlain by Beach sand, clay to clayey sand and Marine sand. The Beach sand blanketing the site is fine grained and loose to medium dense. The upper couple feet of the sand may be fill consisting of sand which was graded to create level building pads in the area when the structure was originally constructed. Below a depth of about 10 feet, the Beach sand becomes coarse, contains shells and is dense to very dense. Below the Beach sand is a thin layer, approximately 2- to 5- feet-thick, layer of clay to clayey sand. Blowcount data indicates this layer is soft or loose. Below the clay and clayey sand, dense to very dense Marine sand was encountered to the maximum depth explored of 51.5 feet.

As shown on Figure 4, Map of Regional Geology, the site is underlain by Beach sand (Marine/Dune) which is consistent with our findings.

Groundwater was encountered at depths ranging from 10 to 11 feet below the ground surface at the property during drilling. We judge groundwater is tidally influenced at likely matches the surface of the nearby Pacific Ocean. Higher groundwater should be expected as a result of sea level rise and the team should consult the County for guidelines related to predicted sea level changes.

SEISMICITY AND SEISMIC HAZARDS

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 5. 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[2] [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
N. San Andreas - North Coast	2	West	7.51
N. San Andreas (1906 event)	2	West	8.05
N. San Andreas - Peninsula	15	Southeast	7.23
San Gregorio Connected	15	South	7.50
Point Reyes	17	West	6.90
Total Hayward	27	Northeast	7.00
Total Hayward-Rodgers Creek	27	Northeast	7.33
Rodgers Creek	28	Northeast	7.07
West Napa	46	Northeast	6.70
Green Valley Connected	53	East	6.80
Mount Diablo Thrust	54	East	6.70
Total Calaveras	57	East	7.03
Monte Vista-Shannon	63	Southeast	6.50

[2] 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.

Figure 5 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 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 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 46 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

GEOLOGIC 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 site. Very strong shaking during an earthquake can result in ground failure such as that associated with fault rupture, soil liquefaction[3], lateral spreading[4], and differential compaction[5] and earthquake induced landsliding. We used the results of our field investigation as well as those by others in the vicinity to evaluate the potential of these phenomena occurring at the project site.

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 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) and consequent secondary ground failure from an unknown fault is low.

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[3] 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.

[4] 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.

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

Liquefaction, Lateral Spreading, Differential Compaction, and Earthquake Induced Landsliding

We used the results of our on-site borings to evaluate the potential for liquefaction lateral spreading, and settlement from differential compaction. As shown on the Figure 6, Liquefaction Susceptibility Map, the site is in an area on the map where liquefaction associated permanent ground displacement is expected to occur ("Very High Liquefaction Susceptibility"). Groundwater was observed at the site during our field investigation in the coarse Beach sand layer which is predominately dense. However, because the upper coarse Beach sand may be medium dense and that groundwater levels are predicted to rise within the fine Beach sand layer, liquefaction may occur within a 10 foot zone of sand. Therefore, we conclude liquefaction-induced settlements on the order of 2 to 3 inches may occur at the site. Because the site is relatively flat, we judge the risk of lateral spreading to be low.

In addition, strong ground shaking can cause unsaturated sand above the groundwater table to densify and settle (referred to as differential compaction). Up to approximately 10 feet of loose to medium dense sand above the groundwater table was encountered at the site. This layer may densify as a result of a major earthquake on one of the nearby faults. Therefore, we conclude the site is susceptible to cyclic densification (differential compaction). We estimate that during a major earthquake on one of the nearby faults about 1 to 2 inches of settlement from differential compaction could occur at the site.

CONCLUSIONS AND RECOMMENDATIONS

We conclude from a geotechnical engineering standpoint that the proposed new single-family residence can be constructed as planned provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction.

Because the upper 10 to 15 feet of sand is loose to medium dense and may densify and liquefy up to 5 inches during a major earthquake on one of the nearby faults, we judge drilled piers should be used to support the new residence. Drilled piers should extend through the Beach sands layers that may densify and liquefy and terminate in the dense to very dense Marine sand layer. We anticipate shallow foundations consisting of footings, a mat or thickened edge slab can be used for landscape retaining walls or ancillary structures where seismically induced settlement is deemed acceptable as outlined in the remainder of the report.

A properly constructed drilled pier and shallow foundation system founded in dense Marine sands supporting the new residence using the design guidelines and parameters presented in this report should experience less than 1- and ½- inch of total and differential settlements from anticipated loads, respectively.

Drilled Piers - House

Drilled, cast-in-place concrete piers should be a minimum of 18 inches in diameter and extend a minimum of 7 feet into the Marine sand layer. An 18-inch-diameter pier with 7 foot of embedment into Marine sand should be designed for an allowable dead plus live vertical compression capacity of 56 kips. Piers with a diameter of 24-inches with 7 foot embedment into Marine sand should be designed for an allowable dead plus live vertical compression capacity of 75 kips. Capacities may be increased by 1/3 for seismic and/or wind loads. For each additional foot of embedment below 7 feet of Marine sand, the piers may be designed using an allowable skin friction value of 1,000 psf. Uplift resistance may be calculated using

two-thirds of the compression value as described above, plus the weight of the pier. We estimate piers will extend between 25 and 30 feet below existing site grades extending towards the Pacific Ocean from east to west, respectively, across the footprint of the proposed residence.

Once the layout of the piers and grade beam size is determined, we should confirm the lengths and provide the structural engineer with a more precise estimate of the pier lengths required at each location to achieve proper embedments.

Piers will provide lateral resistance from their structural rigidity. Lateral resistance of piers will depend on the pier diameter, pier head condition (restrained or unrestrained), allowable deflection of the pier top, and the bending moment resistance of the piers. We have performed lateral load analyses for an isolated, 18- and 24- inch-diameter pier for a deflection of 1/2 inch at the pier head. The results of our analyses are presented in Tables 3.

TABLE 3

Results of Lateral Load Analyses for ½-inch Deflection at Pier Top

Pier Diameter (inch)	Pier Top Condition	Computed Lateral Load at ½-inch Deflection (kips)	Computed Maximum Bending Moment (kip-feet)	Depth to Maximum Bending Moment (feet)
18	Restrained (fixed)	13.5	58	0
18	Unrestrained (free)	8.0	23	4.5
24	Restrained (fixed)	23.5	137	0
24	Unrestrained (free)	12.5	40	5

Piers should be spaced a minimum of three diameters measured center-to-center or at least six diameters to utilize the full lateral capacity without reduction due to group effects.

The bottom of the drilled holes should be free of debris, loose soil and water before placement of concrete. Drilling should be observed by a representative of Rollo & Ridley Inc. to confirm the correct embedments are obtained and that the foundation soil is similar to that encountered in our field investigation. The pier depths presented previously are approximate and the contractor should plan for deeper piers as necessary as conditions exposed during drilling may dictate up to three feet of variation in the total pier depths.

Since drilled piers will extend below the groundwater, the contractor should anticipate needing casing and/or the use of slurry to prevent caving (flowing or collapsing soil due to groundwater). In addition, if pumps cannot remove groundwater for concrete placement or

drilling slurry is used, concrete should be placed by tremie method (steel tremie pipe extending to the bottom of the hole).

Shallow Foundations – Ancillary Structures

The foundations for ancillary structures may be supported on shallow foundations consisting of footings, a mat or a thickened edge slab bearing on Beach sand. It should be noted that any structures founded on shallow foundations will experience seismically induced settlements during a major earthquake which likely will cause major damage that may necessitate the demolition and replacement of those structures after a major earthquake. If used, we recommend footings bearing on medium dense sand be designed for a maximum dead plus live load bearing pressure of 2,000 pounds per square foot (psf). This value may be increased by 1/3 (2,700 psf) for total loads, including wind and/or seismic. Continuous footings should be at least 18-inches wide and isolated interior footings at least 24-inches square. As a minimum, footings should be founded 18 inches below the lowest adjacent grade. The thickened edge of the mat should be at least 30 inches wide.

Resistance to lateral forces can be obtained from passive pressure against the sides of foundation elements. Passive resistance may be calculated using an equivalent fluid pressure of 250 pounds per cubic foot (pcf). Frictional resistance should be computed using a base friction coefficient of 0.30 (bearing on sand). If waterproofing will be used below footings, this value may need to be reduced depending on recommendations by the waterproofing manufacturer. The passive and friction values include a factor of safety of about 1.5 and may be used in combination without reduction. All footing excavations should be moisture conditioned and compacted prior to steel placement to achieve at least a medium dense state.

The foundation subgrade should be free of standing water, debris, loose or soft material prior to placement. In addition, all excavation subgrades should be kept in a moist condition until the 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.

Lowest Floor Slab

We understand except for the garage portion of the residence, the structure will be raised and no living space is planed near existing site grades. If plans change, and a slab on grade is incorporated into the project plans below living space, it should be underlain by a waterproofing membrane.

At the discretion of Butler Armsden Architect and County standards, the garage floor may be underlain by a waterproofing membrane or 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 although it is not anticipated for use on this project unless used below the garage portion of the site where a lower standard is usually allowed by the architect or waterproofing consultant.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock compacted smooth with a vibratory plate so that gravel/rock edges do not puncture vapor retarder.

The vapor retarder (15-mil Stego Wrap © or equivalent) should meet the requirements for Class C vapor retarders stated in the most current version of ASTM E1745 and the vapor retarder to be placed in accordance with the most current version of the requirements of ASTM E1643. 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. Design parameters for the gravel/crushed rock and sand are presented in Table 4.

TABLE 4
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

If the sand overlying the membrane is not 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 may be covered with plastic sheeting to avoid wetting. If the sand becomes wet, the placement of concrete should be avoided 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, we judge that one design parameter for the floor slab concrete be that it has 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 may be increased by adding plasticizers.

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

Landscape Retaining Walls

Landscape retaining walls should be designed using the foundation systems outlined previously in this report and be designed to resist lateral earth pressures and surcharges.

Lateral earth pressures on freestanding retaining walls will depend upon the steepness of the slopes behind the walls. Recommended lateral earth pressures for walls free to rotate for various slope inclinations are presented in Table 5. It should be noted that retaining walls designed to rotate, will move outward near the top of the wall over time (over several months or years), causing concrete cracking to the wall and ground settlement of the retained soil near the top of the wall. Where walls will not be allowed to rotate and designed for an at-rest condition, the values presented in Table 5 should be increased by a factor of 1.60.

TABLE 5
Retaining Wall Pressures

Slope Angle, β (degrees)	Walls Free to Rotate
0	40 pcf
18 (3:1)	60 pcf

If the design team identifies any surcharges, we can assist the design team to develop the appropriate surcharge loading conditions.

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). In lieu of a four-inch collector pipe and gravel, a thicker drainage paneling (such as Hydroduct® Coil 600 or equivalent) is acceptable. We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. The collector pipes or the thicker drainage paneling should be connected 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.

Even with drainage installed, dampness and discoloration on the walls should be expected due to natural percolation of rainwater, irrigation or other water introduced behind the walls. If this is not acceptable, the walls should be waterproofed. If used, a waterproofing system should be designed by the architect and/or waterproofing consultant.

Site Preparation and Grading

Prior to construction, the area 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. We anticipate that the on-site, near surface gravelly clay (fill) will not meet the fill requirements and should not be used as engineered fill except for landscaping areas if approved by the architect. 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.

In areas that will receive vehicular traffic, the upper eight inches of the soil subgrade should be scarified, moisture conditioned, 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 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 (including aggregate baserock) is used as backfill, however, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill is not permitted.

On Grade Exterior Hardscape

To mitigate the effects of weak near-surface soil and organic matter found in the near-surface soil, sidewalks, patios, concrete pavers, and other concrete flatwork not supported on drilled piers should be underlain by at least 12 inches of compacted soil. To achieve the 12 inches, the existing soil should be stripped to a depth of at least 6 inches, the subgrade scarified and recompacted, and the stripped soil replaced as compacted fill. A geotextile may be required to stabilize the subgrade. If the surficial soil does not meet the requirements for fill (such as topsoil) or the design team/owner wants better long-term performance of exterior hardscape slabs, an approved imported fill consisting of Class 2 aggregate baserock

(AB) should be used in lieu of the native soil for the upper 6 inches. AB should conform to current Caltrans Standard Specifications.

If any of the concrete hardscape will be subjected to vehicular loading, slabs should consist of a minimum of six inches of Portland cement concrete over 12 inches of aggregate base. The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 10-foot spacing or less (depending on recommendations of the civil or structural engineer). Where the outer edge of a concrete pavement meets asphalt pavement (or terminates), the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. If the concrete will experience traffic from larger vehicles (SUVs or vans) and/or occasional truck traffic, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch spacing in both directions; however, steel reinforcing (size and spacing) should be designed by the structural or civil engineer. Recommendations for subgrade preparation and aggregate baserock compaction for concrete pavement are as outlined in previous sections of this report.

Drainage

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, roadways, pavements or slopes. Surface runoff should be directed away from foundations to an acceptable City outlet. In addition, all roofs should have gutters and downspouts that are connected to a closed pipe system and outlet away from the structure to the city sewer and/or storm drain system or a dissipater system as designed by the civil engineer as appropriate. If any of the perimeter foundations (such as for the garage structure) act as partial basement walls (when the soil on the outside is higher than the floor slab on the inside), the outside of the foundation walls should be waterproofed and back drained. One acceptable method for backdraining a wall is to place a prefabricated drainage panel against the waterproofed covered concrete wall. The drainage panel should extend down to a perforated PVC (schedule 40) collector pipe at the base of the wall (below the level of the interior slab). 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). A thicker drainage paneling (such as Hydroduct® Coil 600 or equivalent) may be used in lieu of a perforated PVC pipe. The collector pipes should drain to a suitable discharge location.

Temporary Slope Cuts

Excavations deeper than four feet should be shored or sloped for safety in accordance with CAL-OSHA standards. We recommend temporary cut slopes in the sand should conform to OSHA Type C soil 1-1/2 to 1 (horizontal to vertical) except near the toe of the cut, where a vertical cut with a maximum height of three feet can be made. Wooden formwork or products such as Stayform may be required to maintain the temporary vertical cuts in the sandy soil. Steeper slopes may be made but only after review in the field by a representative from our office.

Although not envisioned for this project, 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. Completed (permanent) cut slopes should be blanketed with erosion control, as specified by the civil engineer or architect.

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, at a minimum, the seismic design should be in accordance with the provisions of 2019 California Building Code (CBC) and ASCE 7-16 including the following:

- Risk Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 2.361g and 0.998g, respectively.
- Site Class D (Provided a deep foundation is installed)
- Site Coefficients: F_a=1.0, F_v, see Section 11.4.8 of ASCE 7-16
- MCE_R spectral response acceleration parameter at short period, S_{MS}, of 2.361g
- MCE_R spectral response acceleration parameter at one-second period, S_{M1}, see Section 11.4.8 of ASCE 7-16
- Design Earthquake (DE) spectral response acceleration parameter at short period, S_{DS}, of 1.574g
- Design Earthquake (DE) spectral response acceleration parameter at one-second period, S_{D1}, see Section 11.4.8 of ASCE 7-16

ADDITIONAL SERVICES

We should review the structural drawings including the contractor's submittal for the proposed drilled pier installations prior to construction; this will allow us check for conformity with this report. During construction, we should observe drilled pier foundation installations, subgrade compaction of shallow foundations, backfill and compaction of utility trenches, slab subgrades and pavement layers, and excavations (temporary slope cuts). These observations should allow us to compare the actual with the anticipated soil conditions and to verify the contractor's work conforms to the geotechnical aspects of the plans and specifications. Once the project schedule is available, we will prepare a proposal and fee estimate to provide construction observation services.

LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and construction conditions as we have described them and are the result of limited engineering studies and our interpretations of the available subsurface data and existing geotechnical conditions. Actual subsurface conditions may vary. Should conditions differ substantially from those we anticipate, some modifications to our conclusions and recommendations may be required. Furthermore, 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. We make no representation, warranty or guarantee, 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.

If you have any questions, please call me at 415-999-1431.

Best regards,
ROLLO & RIDLEY, INC.

REGISTERED PROFESSIONAL ENGINEER
FRANK J. ROLLO
GE 2370
Exp. 9/30/22
GEOTECHNICAL
STATE OF CALIFORNIA

Frank J. Rollo, P.E., G.E.
Principal

1714.1rpt

Distribution: John Wright & Ruth Kiskaddon
Federico Engel – Butler Armsden Architects
Bob Reed – GFDS Engineers

Attachments: References
Figures
Figure 1 – Site Location Map
Figure 2 – Site Plan
Figure 3 – Idealized Subsurface Profile A-A'
Figure 4 – Map of Regional Geology
Figure 5 – Map of Major Faults and Earthquake Epicenters in the San Francisco Bay Area
Figure 6 – Liquefaction Susceptibility Map

Appendix A
Figure A-1 – Log of Boring, RR-1
Figure A-2 – Log of Boring, RR-2
Figure A-3 – Classification Chart

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FIGURES

Base map: The Thomas Guide
Marin County
2002

N

0 1/4 1/2 mile

Approximate Scale

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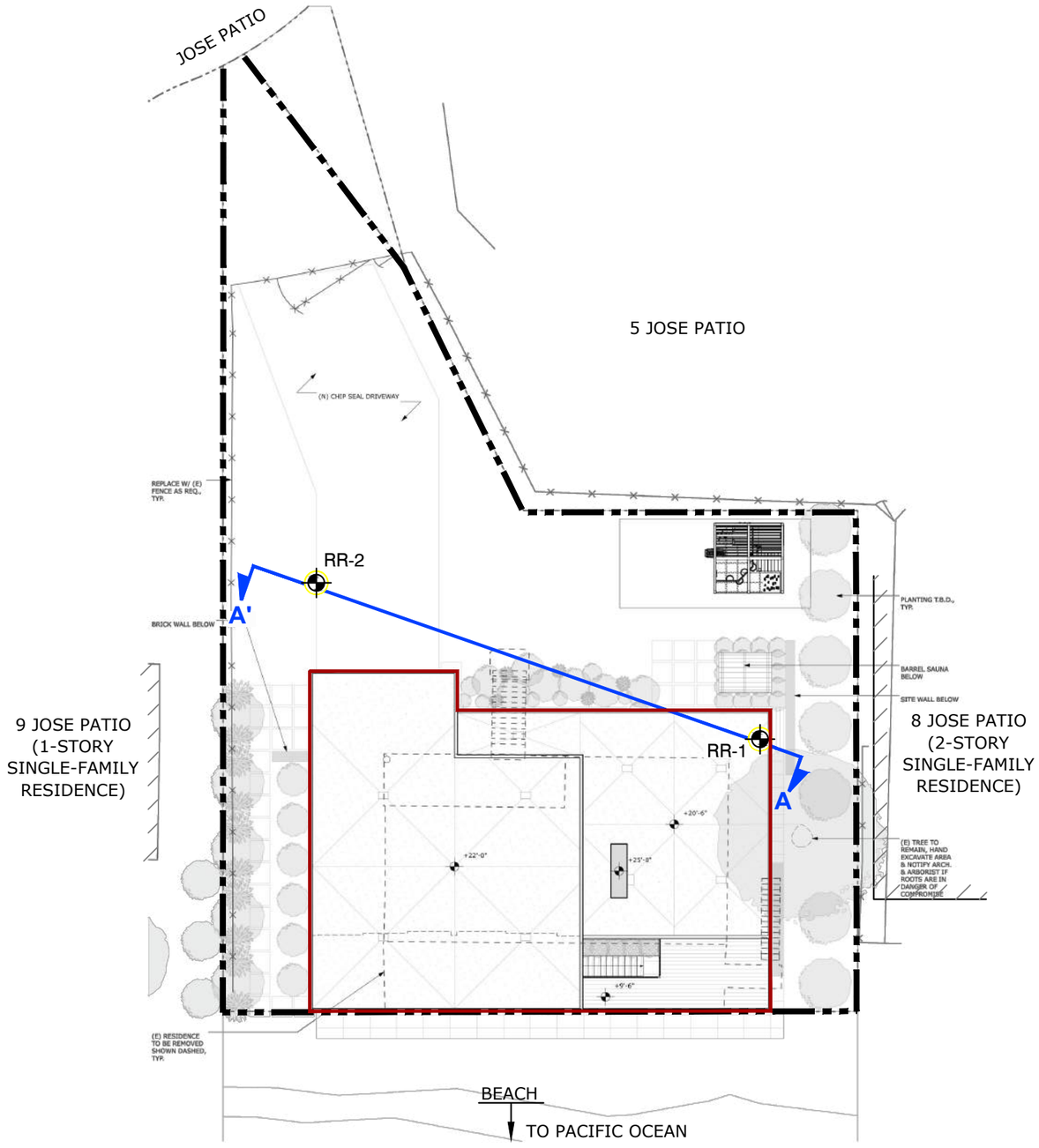
SITE LOCATION MAP
7 JOSE PATIO
Stinson Beach, California

PROJECT No.	1714.1
DATE	09/10/21
FIGURE	1



EXPLANATION

RR-1 — Approximate location of boring by Rollo & Ridley, Inc., drilled on August 26, 2021

Property line (Approximate)

Proposed new residence

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

0 10 20
Scale (feet)

Reference: Base map from a drawing titled "Proposed Site Plan", Sheet A0.1 by Butler Armsden Architects, dated May 3, 2020.

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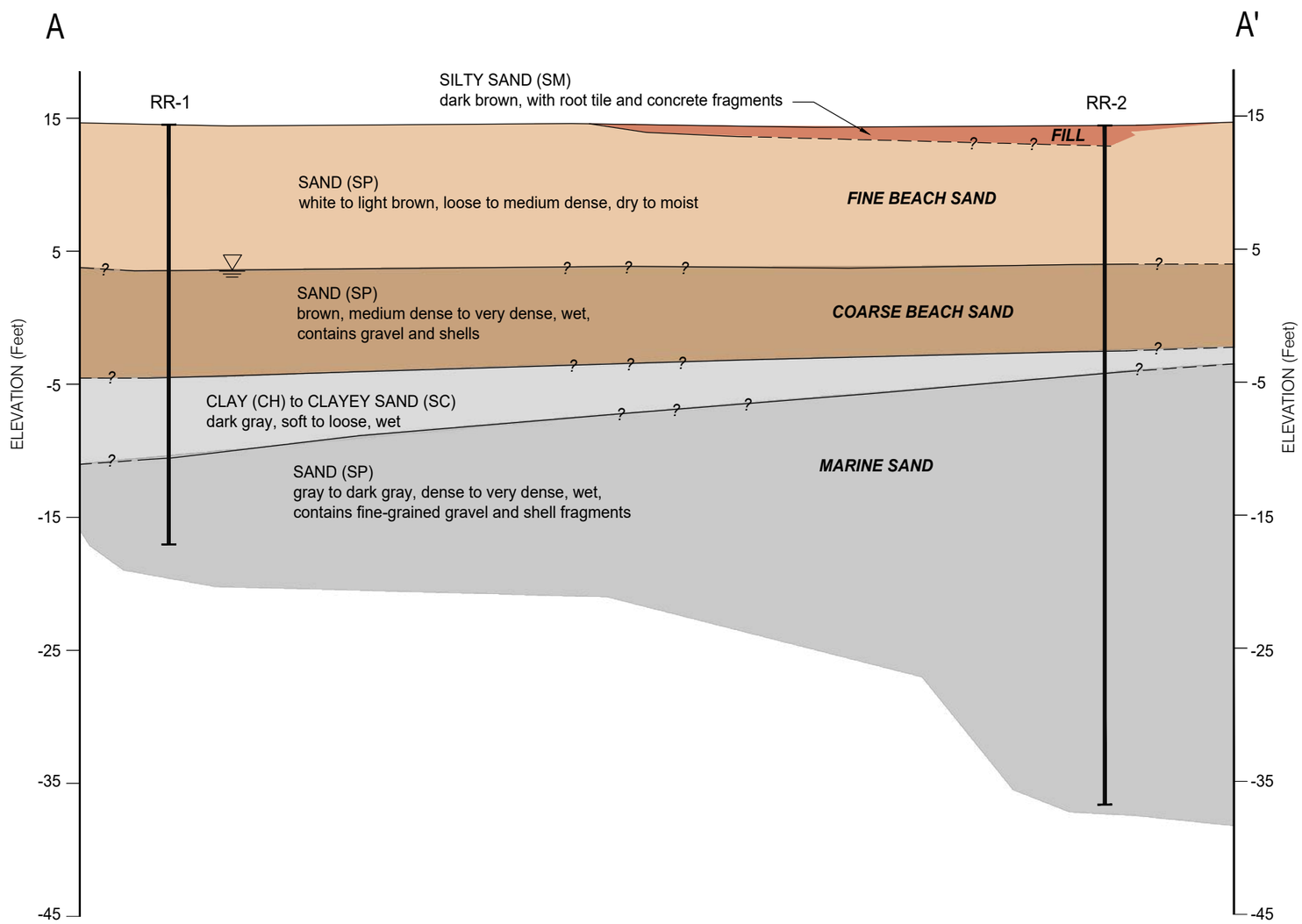
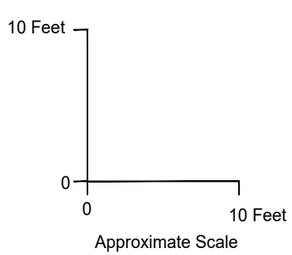
SITE PLAN
7 JOSE PATIO
Stinson Beach, California

PROJECT No.	1714.1
DATE	09/24/21
FIGURE	2



FACING WEST



Notes:
1) Elevations in feet, NAVD 88.
2) 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.

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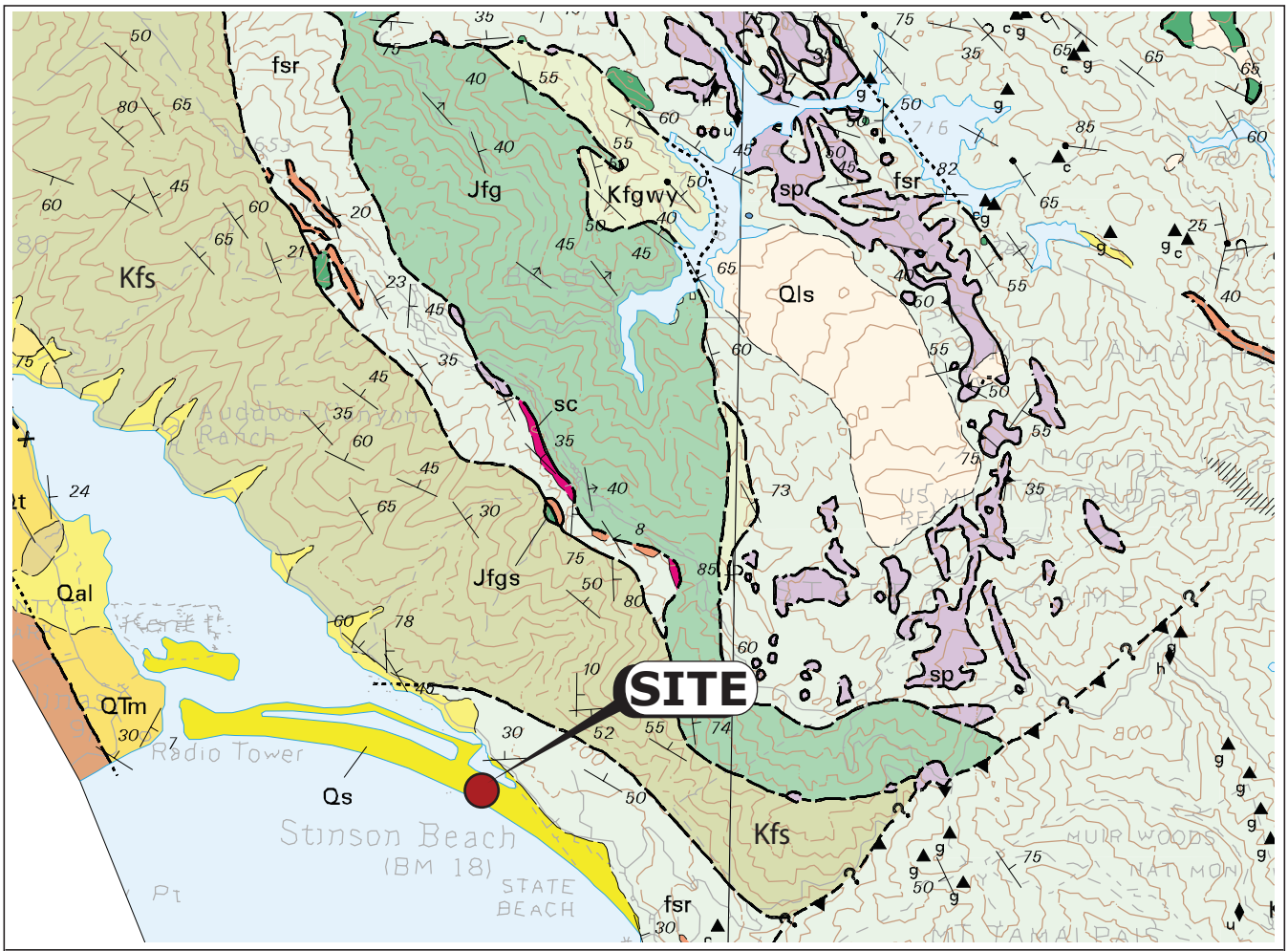
IDEALIZED SUBSURFACE PROFILE A-A'
7 JOSE PATIO
Stinson Beach, California

PROJECT No.	1714.1
DATE	09/30/21
FIGURE	**3**



Base map: Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California.

Qs	Beach sand (Quaternary)	Jfg	Greenstone (Jurassic)
Qls	Landslide deposits (Quaternary)	Jfgs	Greenstone (Jurassic)
Qal	Alluvium (Quaternary)	sp	Serpentinite (Jurassic)
QTm	Merced Formation (early Quaternary and late Pliocene)	sc	Silica-carbonate rock (Jurassic)
Kfgwy	Graywacke (Cretaceous)	Kfs	Sandstone and shale (Cretaceous)

35 **Strike and dip of bedding**

48 **Strike and dip of bedding, top indicator observed**

52 **Approximate strike and dip of bedding**

83 **Overturned bedding**

Vertical bedding

35 **Strike and dip of foliation**

Vertical foliation

70 **Approximate strike and dip of pillow lava**

Regions of hydrothermal alteration

u **High-grade mélange block**-- Letter code for rock type (u=Jfmgs, r=Jfmch, h=Jfmg)

s **Low-grade mélange block** -- Letter code for rock type (s=sp, c=KJfch, g=Jfgs, w=Kfgwy, n=sc)

Contact-- Depositional or intrusive contact, dashed where approximately located, dotted where concealed, queried where uncertain

76 **Fault**-- Dashed where approximately located, small dashes where inferred, dotted where concealed, queried where location is uncertain, x indicates an active fault. Arrow and number show fault dip where measured

Thrust fault-- Sawteeth on upper plate, dashed where approximately located, small dashes where inferred, dotted where concealed, queried where location is uncertain

Normal fault-- Tics on upper plate, dashed where approximately located, small dashes where inferred, dotted where concealed, queried where location is uncertain

N

0 1 2 Mile

Approximate scale

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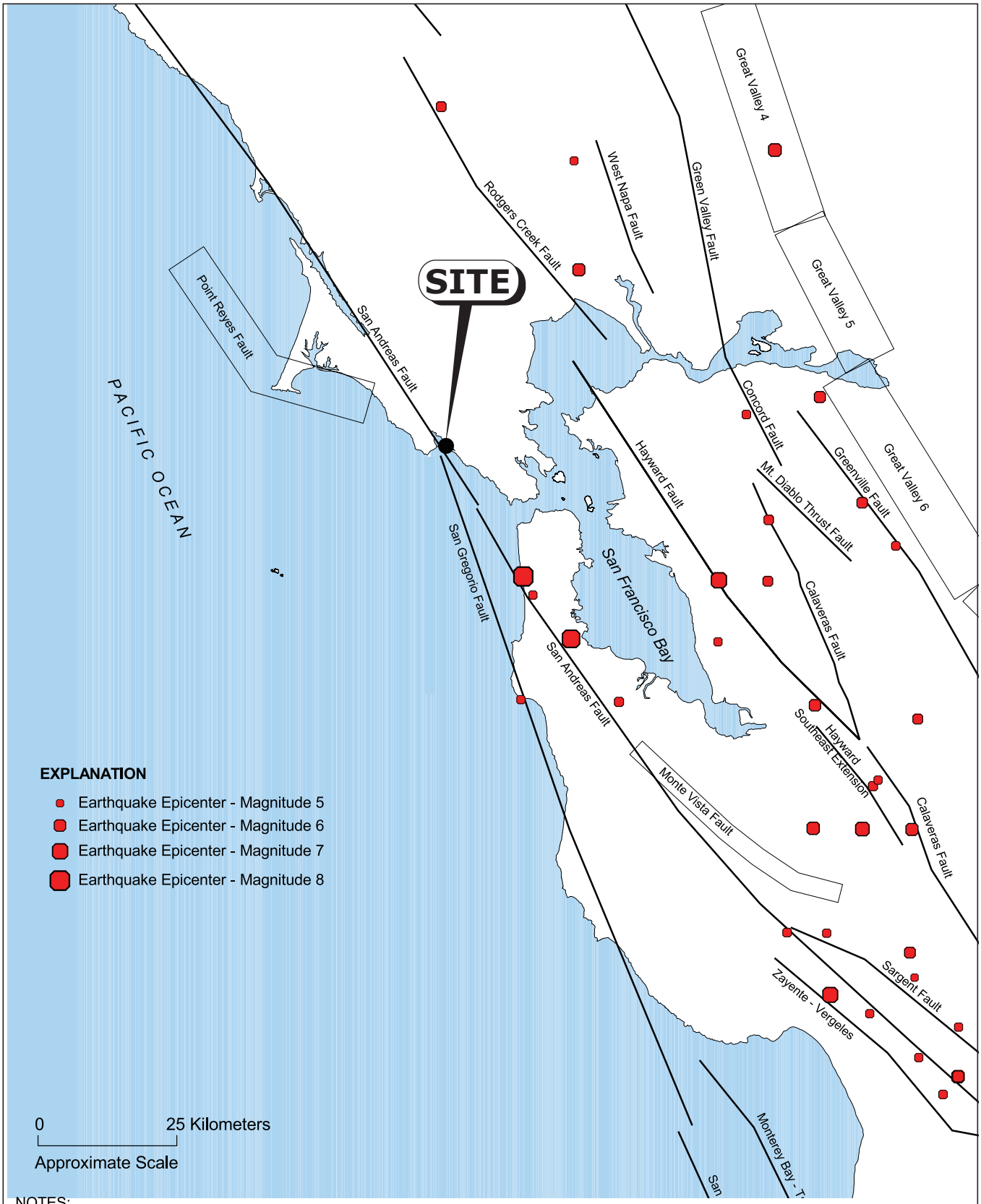
MAP OF REGIONAL GEOLOGY
7 JOSE PATIO
Stinson Beach, California

PROJECT No.	1714.1
DATE	09/10/21
FIGURE	4



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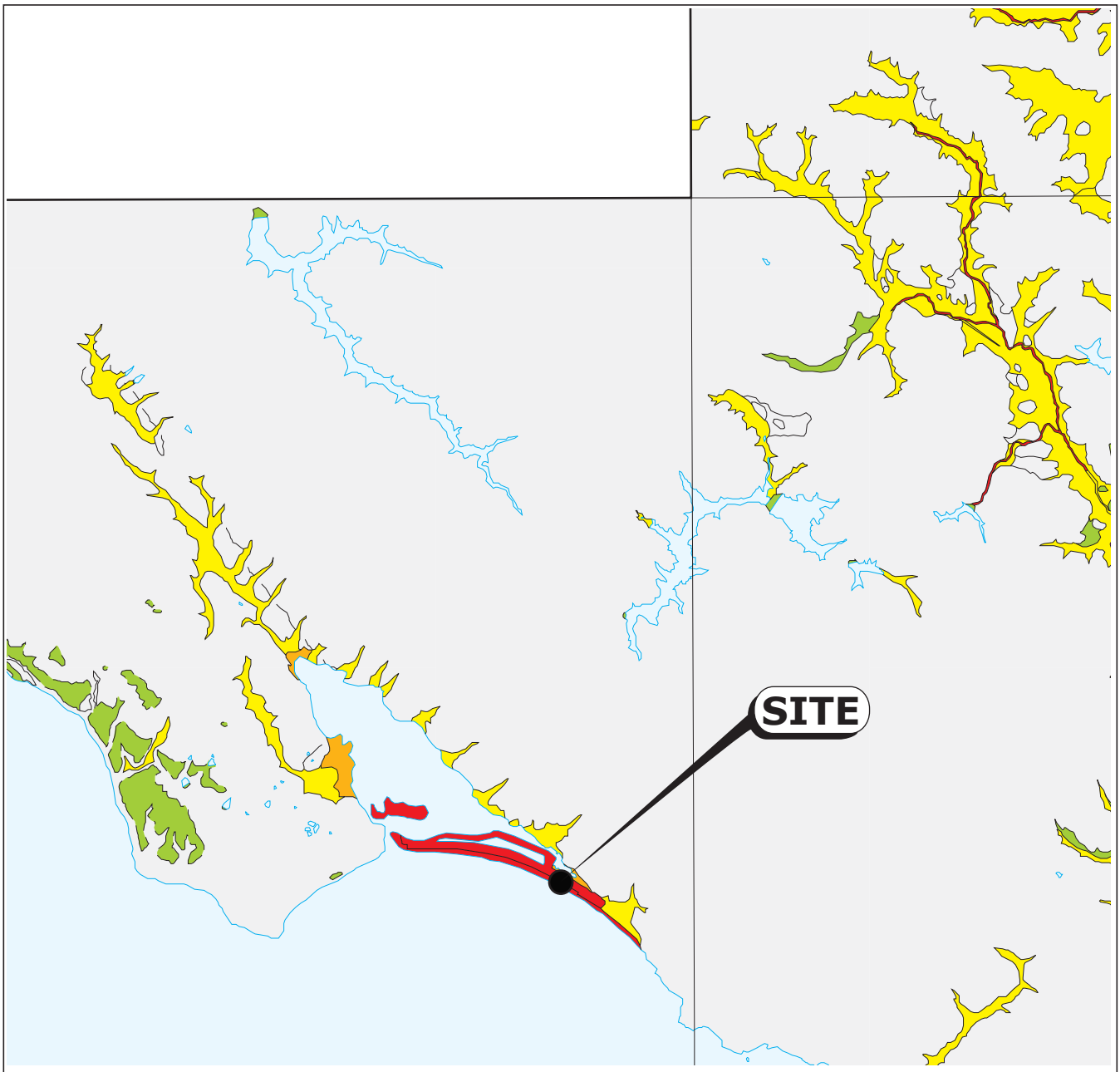
MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

7 JOSE PATIO
Stinson Beach, California

PROJECT No.	1714.1
DATE	09/10/21
FIGURE	5



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
7 JOSE PATIO
Stinson Beach, California

PROJECT No.	1714.1
DATE	09/10/21
FIGURE	6

APPENDIX A
Logs of Borings & Classification Chart

PROJECT: 7 JOSE PATIO
Stinson Beach, California

Log of Boring RR-1

Boring location: See Site Plan, Figure 2
Logged by: F. Rollo
Date started: 08/26/2021 | Date finished: 08/26/2021
Drilling method: Truck-Mounted Rig; Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic Hammer
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Ground Surface Elevation: 14.8 Feet[2]

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						SAND (SP) white to light brown, loose to medium dense, moist	FINE BEACH SAND						
3–4	SPT		3 5 5	12				-200			2	4.1	
5–6	SPT		5 7 7	17	SP	medium dense, moist							
10–11	SPT		5 7 11	22		▽ (08/26/2021; 8:45 AM)							
						SAND (SP) brown, wet, dense, coarse grained	COARSE BEACH SAND	-200			3	21.5	
15–16	SPT		9 18 45	76	SP	contains gravel and shells							
20–21	SPT		3 3 6	11		SANDY CLAY (CH) dark gray, soft to medium stiff, wet		-200			73	56.9	
					CH								
25–26	S&H		3 9 27	29		SAND (SP) gray, dense, wet	MARINE SAND						
					SP								
30–31	SPT		13 26 41	80		contains gravel rounded, very dense							

Boring terminated at a depth of 31.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 11 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.84 and 1.2, respectively, to account for sampler type and hammer energy.
[2] Elevations in feet, NAVD 88.

Project No.: 1714.1
Figure: **A-1**

PROJECT: 7 JOSE PATIO
Stinson Beach, California

Log of Boring RR-2

Boring location: See Site Plan, Figure 2
Logged by: F. Rollo
Date started: 08/26/2021 | Date finished: 08/26/2021
Drilling method: Truck-Mounted Rig; Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic Hammer
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 14.5 Feet[2])	Type of Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1					SM	SILTY SAND (SM) dark brown, moist, with roof tile and concrete fragments						
2						SAND (SP) white to light brown, medium dense, moist						
3–4	SPT		7 7 7	11								
5–6	SPT		4 5 6	13	SP	FINE BEACH SAND	-200			1	4.1	
7–9												
10						(08/26/2021; 10:45 AM)						
11	SPT		4 6 10	19		SAND (SP) brown, medium dense, wet contains gravel and shells	-200			3	23.1	
12–14					SP	COARSE BEACH SAND						
15–16	SPT		10 21 18	47								
17–18					CH	CLAYEY SAND (SC) dark gray, loose, wet	-200			37	52.6	
19						SAND (SP) dark gray, dense, wet, fine grained						
20–21	SPT		5 15 19	41								
22–24												
25–26	SPT		11 21 35	67	SP	shell fragments, some fine-grained gravel						
27–32						SAND MARINE						

Project No.: 1714.1
Figure: **A-2a**

PROJECT: 7 JOSE PATIO
Stinson Beach, California

Log of Boring RR-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
						SAND (SP) (continued)						
33												
34												
35												
36												
37												
38												
39												
40						contains shell fragmnets						
41	SPT		16 16 30	55	SP							
42												
43												
44												
45												
46												
47												
48												
49												
50												
51	SPT		15 38 51	107								
52												
53												
54												
55												
56												
57												
58												
59												
60												
61												
62												
63												
64												

MARINE SAND

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10 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, NAVD 88.

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Project No.: 1714.1

Figure: **A-2b**

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	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

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

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
7 JOSE PATIO
Stinson Beach, California

PROJECT No.	1714.1
DATE	09/10/21
FIGURE	A-3