ROLLO & RIDLEY
GEOTECHNICAL ENGINEERS & SCIENTISTS

GEOTECHNICAL INVESTIGATION
WRIGHT RESIDENCE
726 Point San Pedro Road
San Rafael, California

John Wright & Ruth Kiskaddon
Novato, California

August 26, 2021
Project No. 1693.1

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John Wright & Ruth Kiskaddon
51 Moore Road
Novato, California 94949

Subject: Geotechnical Investigation
Wright Residence
726 Point San Pedro Road
San Rafael, California

Dear Mr. Wright & Ms. Kiskaddon:

This report presents the results of the geotechnical investigation performed by Rollo & Ridley, Inc. for the proposed new single-family residence at 726 Point San Pedro Road in San Rafael, California. Additional copies of this report have been distributed as indicated at the end of this report. The property is on the east side of Point San Pedro Road[1] approximately 3 miles from US 101 and Central San Rafael as shown on the Site Location Map, Figure 1.

The services described in this report were performed in accordance with our proposal and executed professional services agreement dated March 7, 2021. Conclusions and recommendations presented herein are based on: 1) discussions and correspondence with you and Bill Engelhardt of Engelhardt Architecture, 2) a review of the Issued for Planning Approval drawings prepared by Engelhardt Architecture dated May 17, 2021, a Topographic Map prepared by David Harp & Associates dated January 13, 2021 and information obtained from the County website "MarinMap.org", 3) a site specific field investigation and detailed engineering analysis, and 4) our experience with other projects in the vicinity of the site.

The vacant lot is roughly rectangular in shape with maximum plan dimensions of approximately 62 feet by 162 feet. According to the topographic map, the site slopes down to the east from approximately Elevation 20 Feet[2] (NAVD88) at the street to approximately Elevation 8 Feet at the east property line. A mature Oak tree exists at the midpoint of the lot near the north property line and will remain on the property.

We understand current plans include building a new two-story, single-family residence on the western half of the property. The proposed first floor will be roughly at street grade and the living space of the entire structure will be elevated above predicted base flood elevation of 10 feet per the County and FEMA requirements. As a result, the rear of the structure will include a crawl space under a raised wood floor. At the front of the property, an at-grade concrete driveway and vehicle backup area is planned. Civil drawings include details for drainage through and around the proposed structure with the addition of a bioretention planter and spreader at the rear of the property. Only minor excavations are planned.

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[1] Project North referenced in this report is parallel to Point San Pedro Road at the front of the property.

[2] Elevations are in feet, NAVD88, based on Topographic Map prepared by David Harp & Associates dated January 13, 2021.

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The focus of our investigation was to determine the depth to bedrock and the properties of the underlying soil (and bedrock) 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, bedrock and groundwater conditions at the site
- appropriate foundation type(s) for the proposed residence
- design criteria for the recommended foundations
- estimates of foundation settlement
- below grade wall design criteria
- slab on grades (driveway & vehicle backup area)
- site seismicity and seismic hazards
- site grading, including fill and compaction 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

On March 12, 2021, as part of our on-site field investigation, we logged the conditions exposed in 3 hand-augured borings and performed 5 dynamic cone penetration tests (DCPTs) at the property. In addition, shallow bedrock was mapped across the western portion of the property. The locations of the borings, DCPTs and shallow bedrock mapping are shown on the Site Plan, Figure 2.

The borings, designated as RR-1 through RR-3, were augured to depths ranging from 8- to 10.5- feet below the adjacent ground surface where refusal was met. Changes in soil and bedrock were visually observed and recorded; the Logs of Borings, RR-1 through RR-3 are presented in Appendix A as Figures A-1 through A-3.

In addition to the borings, we mapped the bedrock exposed at shallow depths across the property and presented the results on the Site Plan, Figure 2.

The soil and bedrock encountered in the borings and exposed at shallow depths across the property was classified in accordance with the Classification Chart and Physical Criteria for Rock Descriptions presented as Figures A-4 and A-5, respectively.

The DCPTs, designated as DCPT-1 through DCPT-5, were advanced to depths of approximately 4- to 10- feet below the existing ground surface within the footprint of the proposed residence. The DCPTs were performed by driving a 1.4-inch-diameter, cone-tipped probe into the ground with a 35-pound hammer falling 15 inches. The blows used to drive the probes were converted to Standard Penetration Test N-values for use in correlating the relative density of the soil encountered in the hand auger borings, evaluating seismic hazards and performing foundation analyses. The results of DCPTs are presented in Appendix A on Figures A-6 through A-10.

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.

SITE GEOLOGY & SUBSURFACE CONDITIONS

On the basis of the results of our subsurface investigation at the site, we judge that the property is underlain by two distinctly different subsurface conditions; the western half is underlain by bedrock at shallow depths of 2.5 feet or less and the eastern half by up to 9 feet of soil consisting of soft gravelly clay, clayey gravel and alluvium underlain by bedrock.

As presented on the boring logs and DCPTs, the fill consists of a mix of gravelly clay to clayey gravel mixed with debris and organics. It is soft and represents the transition from bedrock to the San Francisco Bay tidal flats that encompass much of the area.

Material interpreted as alluvium consists of stiff, sandy clay. This material is underlain by sandstone bedrock. The rock as encountered during our field investigation is variably weathered and fractured sandstone. As noted, the top of rock was encountered at shallow depths along the western half of the property and at a depth of about 9 feet at the rear of the proposed residence. The sandstone is typically yellowish- to reddish- brown, moderately to intensely fractured, low hardness to hard, weak to strong, and highly weathered to locally slightly weathered where less fractured. However, it should be noted that Franciscan Complex bedrock typically becomes less fractured, harder, stronger and less weathered with depth.

It should be noted that a large outcrop of hard and strong bedrock was encountered along the north property line near the street (Figure 2). In is unclear from our field investigation if this outcropping is part of the intact bedrock or a large boulder. Our geologist will visit the property to assist with the evaluation once construction activities begin as part of the site grading and foundation installations.

As shown on Figure 3, Map of Regional Geology, the site is located in an area where the transition between bedrock (western half) and fill (eastern half) which is consistent with our findings.

Groundwater was encountered at depths of approximately 4 to 5 feet within the tidal portion of the property corresponding to about Elevation 5 to 6 Feet (NAVD88). We anticipate that tidal waters roughly between Elevation 10 Feet and Elevation 0 Feet (depending on tide levels) likely affect the level of groundwater within the lower portions of the property and therefore we estimate that groundwater levels at approximately Elevation 10 Feet, or marginally higher, could occur locally. Furthermore, shallow perched groundwater may travel seasonally along the fill-bedrock contact from the east towards the west and west into the San Francisco Bay.

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 4. For each of the active faults within about 60 kilometers (km) of the site, the distance from the site and the mean characteristic Moment magnitude[3] [2007 Working Group on California Earthquake Probabilities (WGCEP) and Cao et al. (2003)] are summarized in Table 1.

[3] 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.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
Total Hayward	10	Northeast	7.00
Total Hayward-Rodgers Creek	10	Northeast	7.33
Rodgers Creek	13	Northeast	7.07
N. San Andreas - North Coast	19	West	7.51
N. San Andreas (1906 event)	19	West	8.05
N. San Andreas - Peninsula	23	Southwest	7.23
San Gregorio Connected	26	Southwest	7.50
West Napa	29	Northeast	6.70
Point Reyes	31	West	6.90
Green Valley Connected	36	East	6.80
Mount Diablo Thrust	40	East	6.70
Total Calaveras	45	East	7.03
Great Valley 5, Pittsburg Kirby Hills	54	East	6.70
Greenville Connected	58	East	7.00
Great Valley 4b, Gordon Valley	58	Northeast	6.80

Figure 4 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 117 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 31 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 soil liquefaction[4], lateral spreading[5], differential compaction[6], fault rupture and earthquake-induced landsliding.

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[4] 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.

[5] 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.

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

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 very low. In a seismically active area such as the Bay Area, the remote possibility exists that future faulting could occur in areas where no faults were previously recognized; however, we conclude the risk of fault rupture (surface faulting) and associated secondary ground failure from an unknown fault is very low.

Liquefaction, Lateral Spreading and Differential Compaction

We used the results of the on-site borings and DCPTs to evaluate the potential for liquefaction lateral spreading, and settlement from differential compaction. Groundwater was observed at the site during our field investigation at about Elevation 5 Feet to Elevation 6 Feet (eastern portion of the proposed residence). Groundwater was not encountered along the west side (front) of the property and in that area the groundwater level is likely within the bedrock layers which are sufficiently strong to resist the potential for these phenomenon. It appears saturated soils exist at the lower elevations on the property closest to Bay.

In those lower areas (closest to the Bay), we judge that the soil layers contain a large enough percentage of fines (clay) below the groundwater table or are strong enough and are not susceptible to liquefaction or lateral spreading.

In addition, strong ground shaking can cause unsaturated sand above the groundwater table to densify and settle (referred to as differential compaction). No sand was encountered during our field investigation, and therefore, we judge the potential for differential compaction to occur at the property is very low.

Furthermore, the proposed structure including the rear deck and stairs will be supported on drilled piers embedded into bedrock and if sandy soils are present and do settle as a result of liquefaction, lateral spreading and differential compaction, the risk of impact to the structure will be minimized.

Earthquake-Induced Landsliding

Rice et al's detailed geologic mapping of San Rafael (1976) does not show any landslides or areas of thick surficial soils within the areas underlying or adjacent to 726 Point San Pedro Road. No indications of significant slope instability were noted within or near the limits of the property during our investigation, and no landslides are known to have been previously mapped or otherwise identified within the site.

Given the presence of shallow bedrock (less than 10 feet) under the area of the proposed residence, and provided that the improvements will incorporate the recommended drilled pier foundation system, we believe that any potential earthquake-induced landslide damage would be mitigated. Landslide potential (erosion) on all of the peripheral landscaped slopes should be reduced by managing surface run off from all impermeable surfaces to prevent concentration of runoff on slopes.

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 of the sloping terrain along the western half and the soft soils encountered along the eastern half of the proposed structure, respectively, we judge drilled piers should be used to support the new residence. We anticipate footings, a mat or thickened edge slab can be used for walls or ancillary structures near the street on the uphill side of the property, where at least 7 feet of lateral cover of bedrock can be maintained.

A properly constructed drilled pier and shallow foundation system founded in bedrock 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 Foundation

Drilled, cast-in-place concrete piers should be a minimum of 18 inches in diameter and extend a minimum of 10 feet into bedrock. An 18-inch-diameter pier with 10 foot of embedment into bedrock should be designed for an allowable dead plus live vertical compression capacity of 70 kips. Piers with a diameter of 24-inches with 10 foot embedment into bedrock should be designed for an allowable dead plus live vertical compression capacity of 95 kips. Capacities may be increased by 1/3 for seismic and/or wind loads. For each additional foot of embedment below 10 feet of bedrock, the piers may be designed using an allowable skin friction value of 1,500 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 10 and 20 feet below existing site grades extending towards the Bay from west to east, respectively, across the footprint of the proposed residence and rear deck.

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 a shallow bedrock condition, Type 1 piers and for a soft fill condition Type 2 piers 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 for the two differing pier conditions are presented in Tables 3 and 4. The dividing line where each type of pier should be used is presented on the Site Plan, Figure 2.

TABLE 3 – TYPE 1 PIERS (BEDROCK)

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)	67	250	0
18	Unrestrained (free)	27.5	77	4.5
24	Restrained (fixed)	110	551	0
24	Unrestrained (free)	38.5	114	4.5

TABLE 4 – TYPE 2 PIERS (FILL)

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)	30	157	0
18	Unrestrained (free)	11.5	63	8
24	Restrained (fixed)	55	344	0
24	Unrestrained (free)	20	127	9

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 bedrock embedments are obtained and that the foundation soil/bedrock 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 into the bedrock, the contractor should anticipate variable but potentially hard and slow drilling in the bedrock and the use of core-barrels and rock augers to achieve the recommended pier depths. Drilling may also require casing and/or the use of slurry to prevent caving (flowing 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).

Shallow Foundations – Ancillary Structures

The foundations for ancillary structures where at least 7 feet of lateral cover in the bedrock can be maintained may be supported on shallow footings, mat or a thickened edge slab bearing on bedrock. We recommend footings bearing on bedrock be designed for a maximum dead plus live load bearing pressure of 4,000 pounds per square foot (psf). This value may be increased by 1/3 (5,300 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 and embedded at least 12 inches in undisturbed bedrock. 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 a uniform rectangle pressure of 1,000 psf for the portion of the foundations embedded in undisturbed bedrock (neat cut where concrete is poured directly against the bedrock). If compacted soil backfill against the outside edge of the mat is used (formed construction), passive resistance should be calculated using an equivalent fluid pressure of 250 pcf. Frictional resistance should be computed using a base friction coefficient of 0.40 (bearing on bedrock). 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.

The foundation subgrade should be free of standing water, debris, loose or soft material prior to placement of the waterproofing membrane. 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 bedrock subgrade is suitable to support the design bearing pressures. If loose, overly-saturated, soft or undesirable bedrock is encountered in the excavations, it should be removed, and the over-excavation(s) backfilled with lean or structural concrete.

Lowest Floor Slab

We understand except for the garage portion of the residence, the structure will be raised creating a crawl space below the living space. 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.

As a minimum the subgrade of crawl space should be covered using a thin layer of concrete referred to as a “protective” or “rat” slab.

The garage floor should 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 5.

TABLE 5
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.

Below Grade Walls

Below grade walls associated with the residence should be supported on drilled piers using the appropriate design values presented previously in this report.

Walls should be designed to resist lateral pressures created by the soil, bedrock 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. Research on basement wall pressures was undertaken in a comprehensive research program undertaken at the University of California, Berkeley. This research reached 2 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 soil pressure condition.

Using the procedure outlined in SEAOC 2010 Convention Proceedings for Seismic Earth Pressures on Deep Building Basements, we recommend the pressures presented in Table 6 be used in design for new permanent basement walls and to check the existing basement walls with level backfill.

TABLE 6
Recommended Design Parameters for Walls

Restrained Walls, Drained Condition	
Static	**Dynamic**
At-rest pressures corresponding to an equivalent fluid weight of 60 pounds per cubic foot (pcf)	Greater of either at-rest condition (60 pcf) or active (40 pcf) plus a seismic pressure increment of 25 pcf (equivalent fluid weight, triangular distribution)

Where traffic loads are expected within 10 feet of the walls, an additional surcharge load of 100 psf should be applied to the upper 10 feet of the wall. If the wall will be subjected to potential fire truck loading, the additional surcharge should be 250 psf.

These earth pressures are recommended for walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall 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 ¾-inch drainrock wrapped in filter fabric (Mirafi 140N or equivalent). Prefabricated Hydroduct® Coil 600 (or equivalent) may be used in lieu of a collector pipe. The collector pipes or Hydroduct® Coil 600 should drain to a suitable discharge location. We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use.

Even with drainage paneling 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. If the subgrade is bedrock, compaction is not required. The soil (or bedrock) 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 unless the exposed subgrade consists of bedrock. 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 (unless the subgrade consists of native bedrock). 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. We have reviewed the civil drawings prepared by LTD Engineering and take no exceptions to their design. The drawings show surface runoff directed away from foundations to an acceptable County outlet. In addition, the roof gutters and downspouts are shown to be connected to a closed pipe system and outlet away from the structure. Under no circumstances should outlets drain onto the adjacent slopes.

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 bedrock should conform to OSHA Type A soil 3/4 to 1 (horizontal to vertical). Steeper slopes may be made but only after review in the field by a representative from our office. Temporary slope cuts made in the fill should be no steeper than 2 to 1 (H:V), 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 fill soil.

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.

Permanent slope cuts should start at least 3 feet away from the property line. If temporary slope cuts are planned within 3 feet of the property line, a sequenced manner of construction may be required. We should be advised prior to construction if temporary cut slopes are planned near the property line so we can be on site and advise as necessary. 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 properties.

Excavations and slope cuts should be performed under the part-time observation of an engineer from our office who can assess their stability and so that we can recommend changes to the excavation plan in a timely manner, as necessary. Steeper slopes may be permitted on a case-by-case basis.

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 1.500g and 0.600g, respectively.
- Site Class C
- Site Coefficients; Fa=1.2, Fv=1.4
- MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.800g and 0.840g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.200g and 0.560g, respectively.

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, backfill and compaction, and the excavations. 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.

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

1693.1.1rpt

Distribution: John Wright & Ruth Kiskaddon
Bill Engelhardt – Engelhardt Architecture
Glenn Dearth – LTD Engineering, Inc.

Attachments: References

Figures
Figure 1 – Site Location Map
Figure 2 – Site Plan
Figure 3 – Map of Regional Geology
Figure 4 – Map of Major Faults and Earthquake Epicenters in the San Francisco Bay Area

Appendix A
Figure A-1 – Log of Boring, RR-1
Figure A-2 – Log of Boring, RR-2
Figure A-3 – Log of Boring, RR-3
Figure A-4 – Classification Chart
Figure A-5 – Physical Properties Criteria for Rock Descriptions
Figure A-6 – Dynamic Cone Penetration Test Results, DCPT-1
Figure A-7 – Dynamic Cone Penetration Test Results, DCPT-2
Figure A-8 – Dynamic Cone Penetration Test Results, DCPT-3
Figure A-9 – Dynamic Cone Penetration Test Results, DCPT-4
Figure A-10 – Dynamic Cone Penetration Test Results, DCPT-5

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FIGURES

Base map: The Thomas Guide
Marin County

N

0 1/4 1/2 mile

Approximate Scale

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SITE LOCATION MAP
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	07/09/21
FIGURE	1

POSSIBLE BOULDER OR SANDSTONE/GRAYWACKE OUTCROPPING

SANDSTONE AT 0.5 FEET

SANDSTONE AT 2.5 FEET

SANDSTONE OUTCROPPING

POINT SAN PEDRO ROAD

(E) WOOD FENCE

(E) DECK

(E) 24" LIVE OAK

APPROX. DRIP-LINE OF TREE TRIM TO CLEAR BUILDING SEE TP-1 AND ARBORIST REPORT

APPROX. PROP. LINE

GATE (SEE LANDSCAPE DRAWINGS)

(N) DRIVEWAY (SEE LANDSCAPE AND CIVIL DWGS.)

9' x 19' PARKING SPACE

UPPER FLOOR ROOF EDGE

LINE OF WALL BELOW

PARKING IN GARAGE BELOW

TYPE 1

TYPE 2

SEE A2.3 FOR ROOF ELEVATIONS

LOWER ROOF EDGE

NEW TWO STORY SINGLE SINGLE FAMILY RESIDENCE

EXT. LIGHTING IN SOFFIT (SEE A2.1)

FLOOD ZONE "AE" EL 10 FEET (SHADED AREA)

(N) FENCES (SEE LANDSCAPE DWGS.)

LEVEL SPREADER (SEE CIVIL AND LANDSCAPE DWGS.)

100' WETLAND/BAY BUFFER

EDGE OF WETLAND EDGE OF BAY

P1 P2 P3

RR-1 RR-2 RR-3

DCPT-1 DCPT-2 DCPT-3 DCPT-4 DCPT-5

Project North

True North

0 8 16

Scale (feet)

Reference: Base map from a drawing titled "Site Plan", Sheet A1.1 by Engelhardt Architecture, dated May 17, 2021.

EXPLANATION

RR-1	Approximate location of boring performed by Rollo & Ridley Inc. on March 12, 2021
DCPT-1	Approximate location of Dynamic Cone Penetrometer Test (DCPT) performed by Rollo & Ridley Inc. on March 12, 2021
P1	Approximate location of probe performed on March 12, 2021
	Approximate location of property line
10	Approximate surface contour in feet NAVD88 based on Topographic Map prepared by David Harp & Associates dated January 13, 2021
	Approximate outline of proposed residence including rear deck and stairs
TYPE 1	Approximate dividing line of pile type for lateral load analysis; see report text

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SITE PLAN
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	08/20/21
FIGURE	2

Base map: Google Earth with U.S. Geological Survey (USGS), Marin County, 2021.

EXPLANATION

af	**Artificial Fill**
Qhym	**Mud deposits (late Holocene)**
Qha	**Alluvium (Holocene)**
Qpa	**Alluvium (Pleistocene)**
fsr	**Franciscan Complex melange (Eocene, Paleocene, and (or) Late Cretaceous)**
Kfs	**Franciscan Complex sedimentary rocks (Cretaceous)**

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

N

0 1,000 2,000 Feet

Approximate Scale

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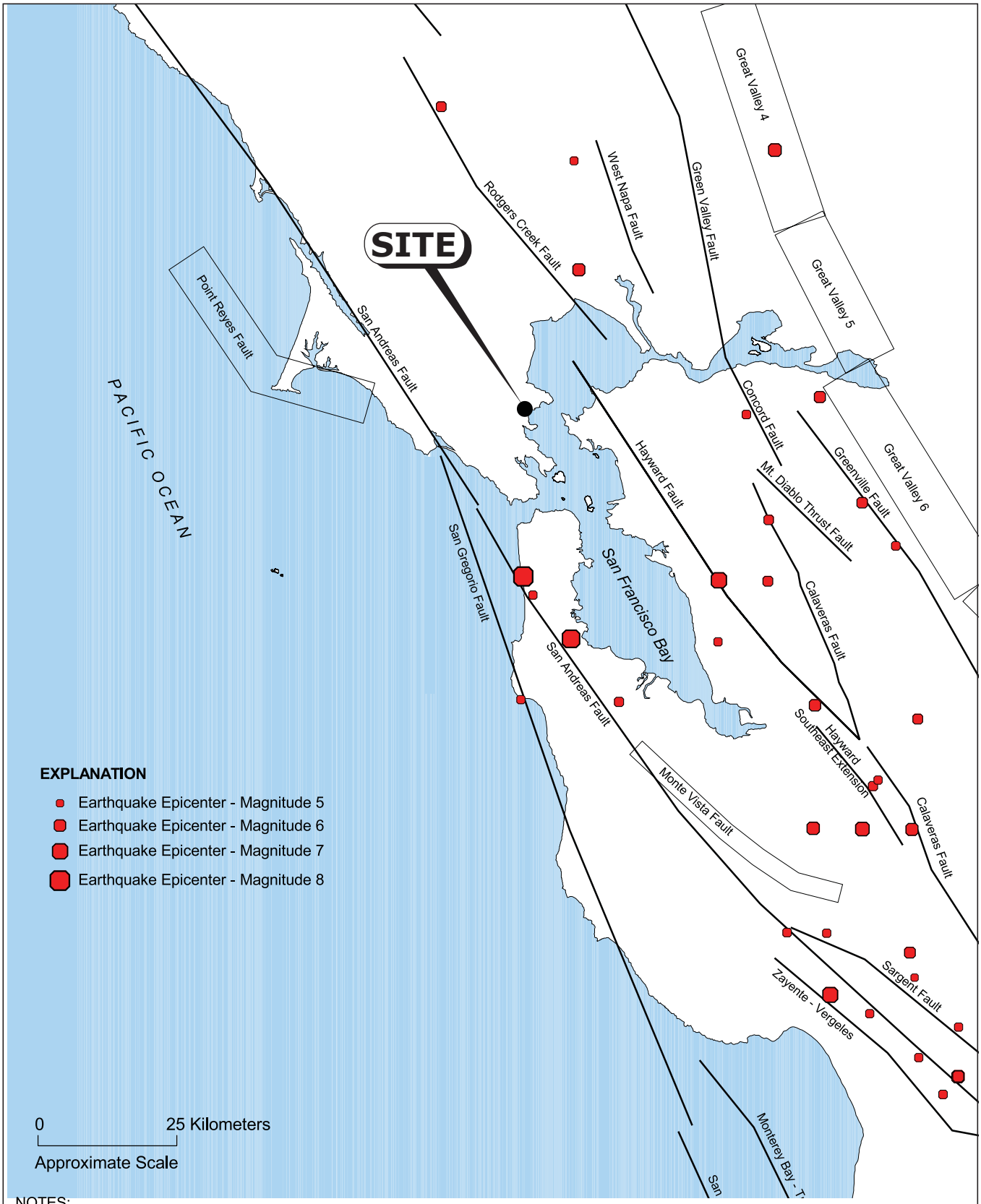
MAP OF REGIONAL GEOLOGY
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	07/09/21
FIGURE	3



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
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	07/09/21
FIGURE	4

APPENDIX A
Logs of Borings, Classification Charts & Dynamic Cone Penetration Test Results (DCPTs)

PROJECT:	**WRIGHT RESIDENCE** **726 POINT SAN PEDRO ROAD** San Rafael, California	**Log of Boring RR-1** PAGE 1 OF 1

Boring location: See Site Plan, Figure 2		Logged by: F. Rollo
Date started: 03/12/2021	Date finished: 03/12/2021	
Drilling method: Hand Auger		
Hammer weight/drop: N/A	Hammer type: N/A	LABORATORY TEST DATA
Sampler: Grab		

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: ~10 feet [1]		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						GRAVELLY CLAY (CL) mottled brown to gray to black, soft[2] to medium stiff, moist	FILL						
1						contains sand							
2													
3					CL								
4						▽ (03/12/2021)							
5													
6													
7						SANDY CLAY (CL) yellow-brown, stiff, wet	ALLUVIUM						
8					CL								
9						bedrock fragments at bottom of hole							
10													
11													
12													

Boring terminated at a depth of 9 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater encountered at a depth of 4 feet during drilling.

[1] Elevation in feet, NAVD88 datum based on a Topographic Survey prepared by Daivd Harp & Associates dated 1-13-2021.
[2] Soil stiffness based on data collected in DCPT-1, see Figure A-6.

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Project No.: 1693.1

Figure: **A-1**

PROJECT:	**WRIGHT RESIDENCE** **726 POINT SAN PEDRO ROAD** San Rafael, California	**Log of Boring RR-2** PAGE 1 OF 1

Boring location: See Site Plan, Figure 2		Logged by: F. Rollo
Date started: 03/12/2021	Date finished: 03/12/2021	
Drilling method: Hand Auger		
Hammer weight/drop: N/A	Hammer type: N/A	LABORATORY TEST DATA
Sampler: Grab		

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 9.5 feet [1]		Type of Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						GRAVELLY CLAY (CL) mottled brown and gray, soft[2] to medium stiff, moist							
1						contains sand							
					CL			-200			56	17.9	
2													
3							FILL						
						CLAY (CL) brown to black, soft, wet (03/12/2021)							
4													
5													
					CL								
6						rubble and plastic pipe fragments in clay							
7													
						SANDY CLAY (CL) yellow-brown, stiff, wet							
8							ALLUVIUM						
					CL								
9													
						SANDSTONE brown, intensly to occassionally fractured, moderately hard to hard, weak to strong, moderate to little weathering	FRANCISCAN COMPLEX						
10													
11													
12													

Boring terminated at a depth of 10.5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater encountered at a depth of 4 feet during drilling.

[1] Elevation in feet, NAVD88 datum based on a Topographic Survey prepared by Daivd Harp & Associates dated 1-13-2021.
[2] Soil stiffness based on data collected in DCPT-2, see Figure A-7.

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Project No.: 1693.1

Figure: **A-2**

PROJECT:	WRIGHT RESIDENCE 726 POINT SAN PEDRO ROAD San Rafael, California	Log of Boring RR-3 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2		Logged by: F. Rollo
Date started: 03/12/2021	Date finished: 03/12/2021	
Drilling method: Hand Auger		
Hammer weight/drop: N/A	Hammer type: N/A	LABORATORY TEST DATA
Sampler: Grab		

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: ~9.8 feet[1]		Type of Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAYEY GRAVEL (GC) mottled brown and gray to black, loose[2], moist							
1					GC		FILL						
2													
						contains sand							
3								-200			31	16.7	
						SANDY CLAY (CL) yellow-brown, stiff, wet							
4													
5					CL	∇ (03/12/2021)	ALLUVIUM						
6													
						SANDSTONE brown, intensly to occassionally fractured, moderately hard to hard, weak to strong, moderate to little weathering							
7							FRANCISCAN COMPLEX						
8													
9													
10													
11													
12													

Boring terminated at a depth of 8 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater encountered at a depth of 5 feet during drilling.

[1] Elevation in feet, NAVD88 datum based on a Topographic Survey prepared by Daivd Harp & Associates dated 1-13-2021.
[2] Soil density based on data collected in DCPT-3, see Figure A-8.

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Project No.: 1693.1

Figure: **A-3**

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
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No. 1693.1
DATE 07/09/21
FIGURE **A-4**

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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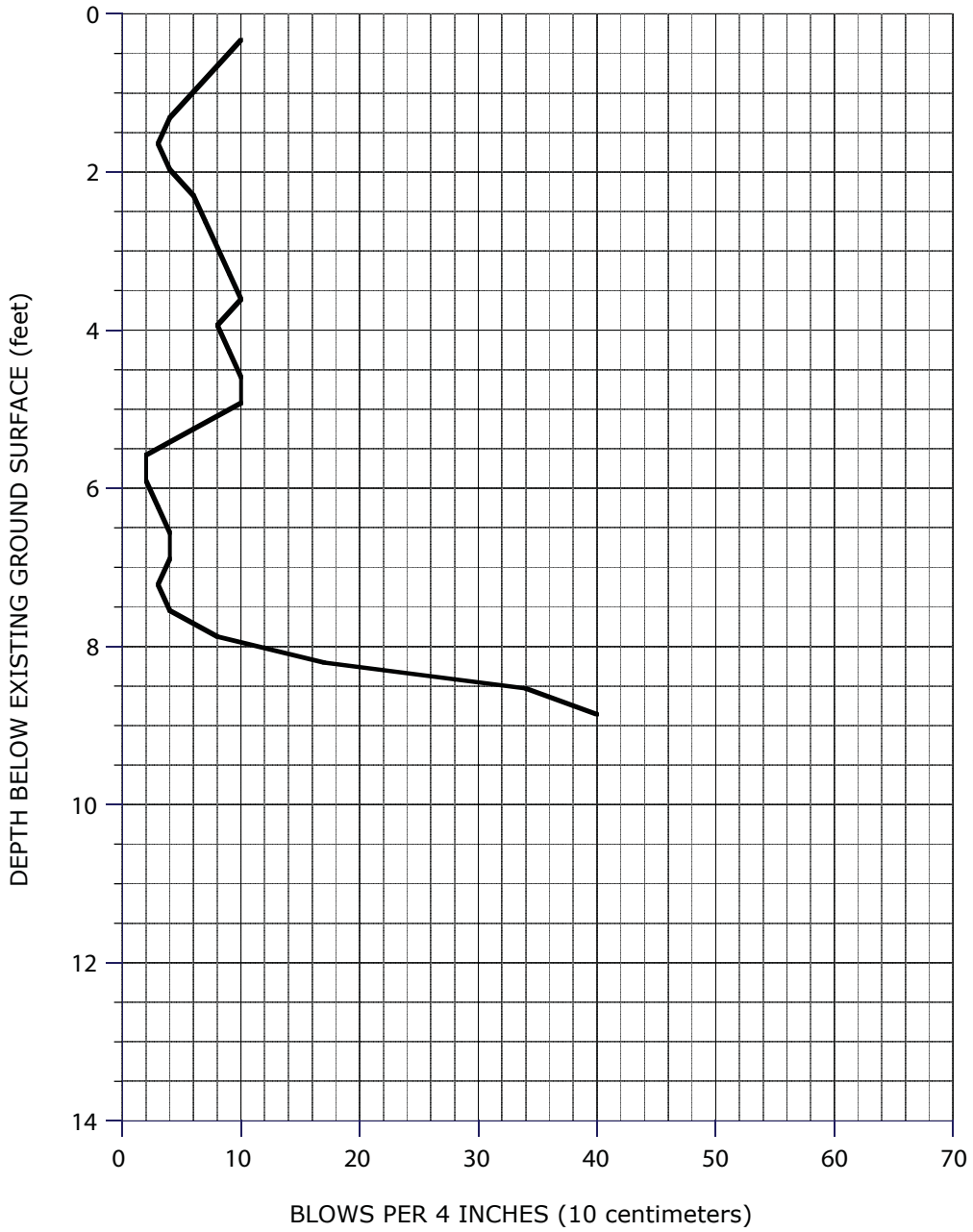
PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No. 1693.1
DATE 07/09/21
FIGURE **A-5**



Notes:
1. DCPT-1 was performed adjacent to RR-1 at the rear of the proposed residence, see Site Plan, Figure 2.
2. Zero depth corresponds to approximately Elevation 10 Feet (NAVD 88).

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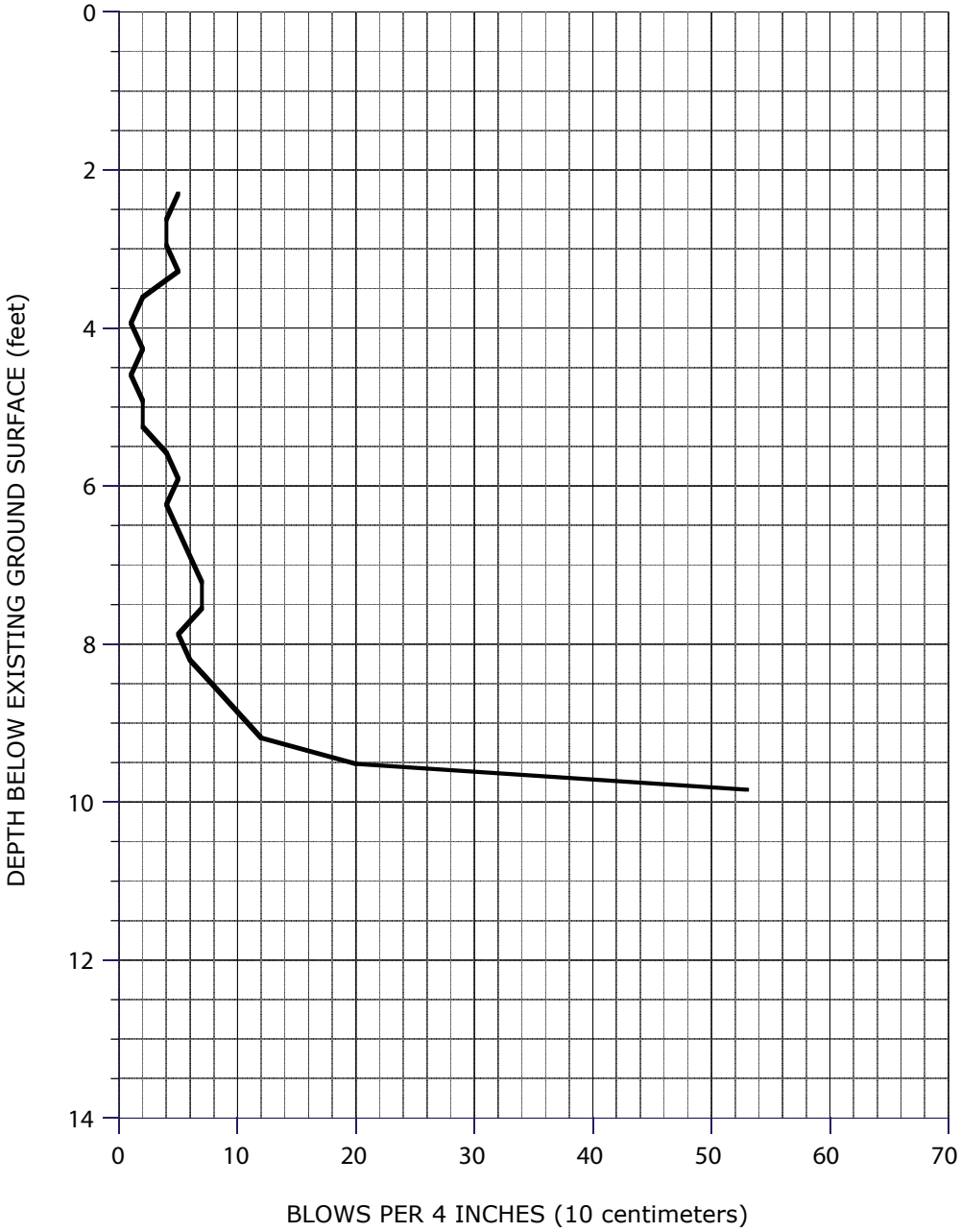
DYNAMIC CONE PENETROMETER TEST RESULTS, DCPT-1
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	07/13/21
FIGURE	**A-6**



Notes:

1. DCPT-2 was performed adjacent to RR-2 at the rear of the proposed residence, see Site Plan, Figure 2.
2. Zero depth corresponds to approximately Elevation 9.5 Feet (NAVD 88).

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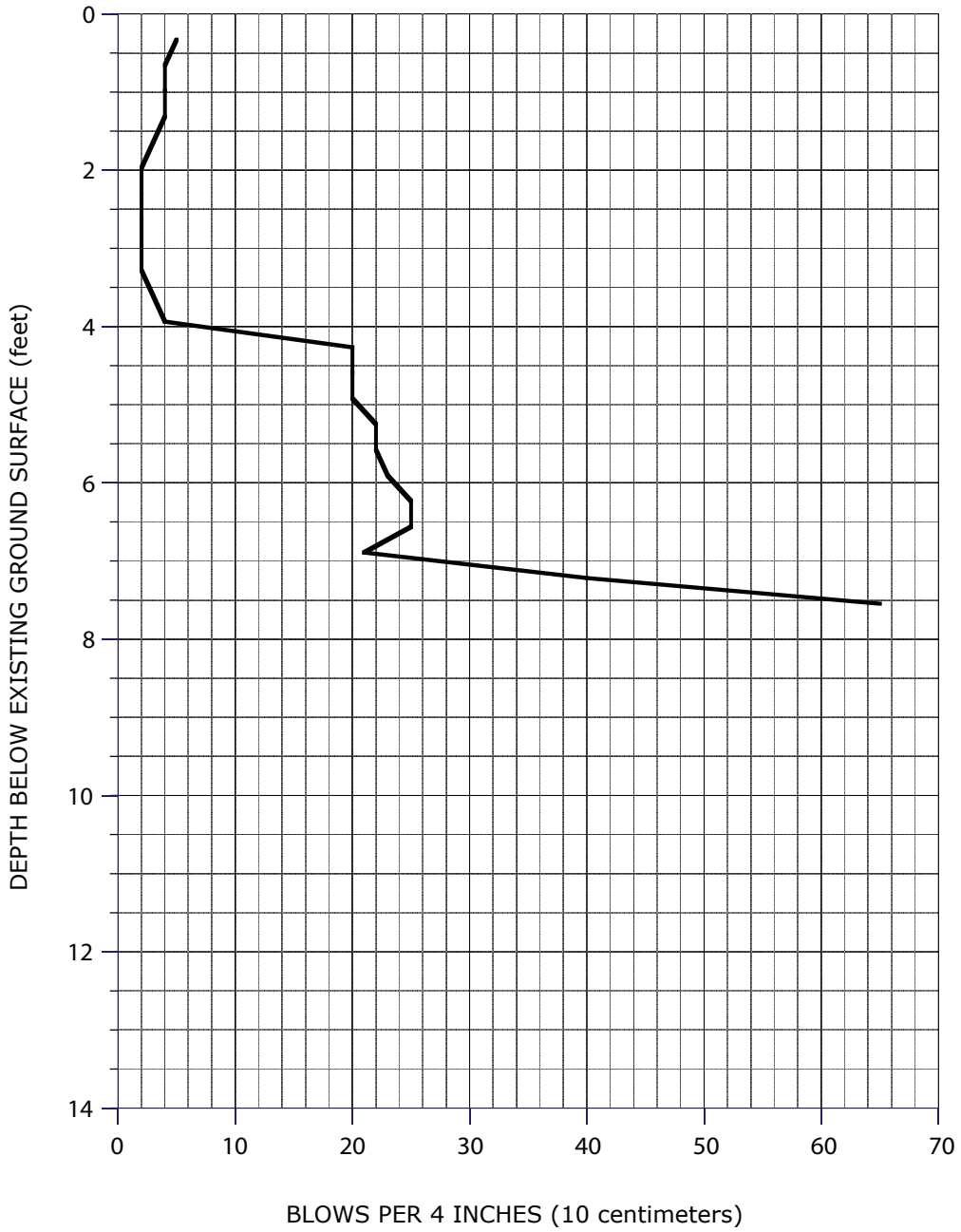
DYNAMIC CONE PENETROMETER TEST RESULTS, DCPT-2

WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	07/13/21
FIGURE	**A-7**



Notes:
1. DCPT-3 was performed adjacent to RR-3 near the middle of the proposed residence, see Site Plan, Figure 2.
2. Zero depth corresponds to approximately Elevation 9.8 Feet (NAVD 88).

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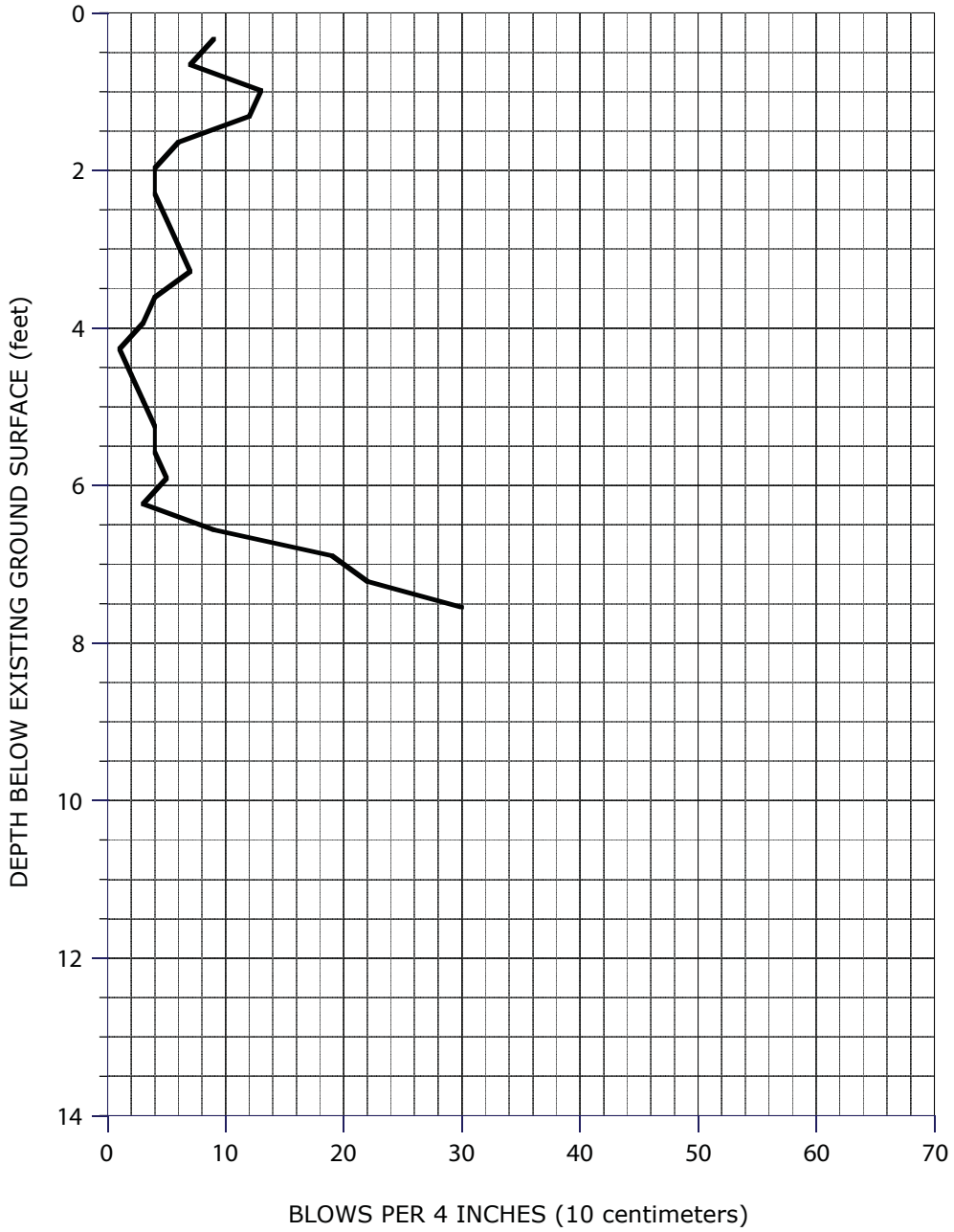
DYNAMIC CONE PENETROMETER TEST RESULTS, DCPT-3
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	07/13/21
FIGURE	**A-8**



Notes:
1. DCPT-4 was performed near the rear of the proposed residence, see Site Plan, Figure 2.
2. Zero depth corresponds to approximately Elevation 10 Feet (NAVD 88).

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DYNAMIC CONE PENETROMETER TEST RESULTS, DCPT-4
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

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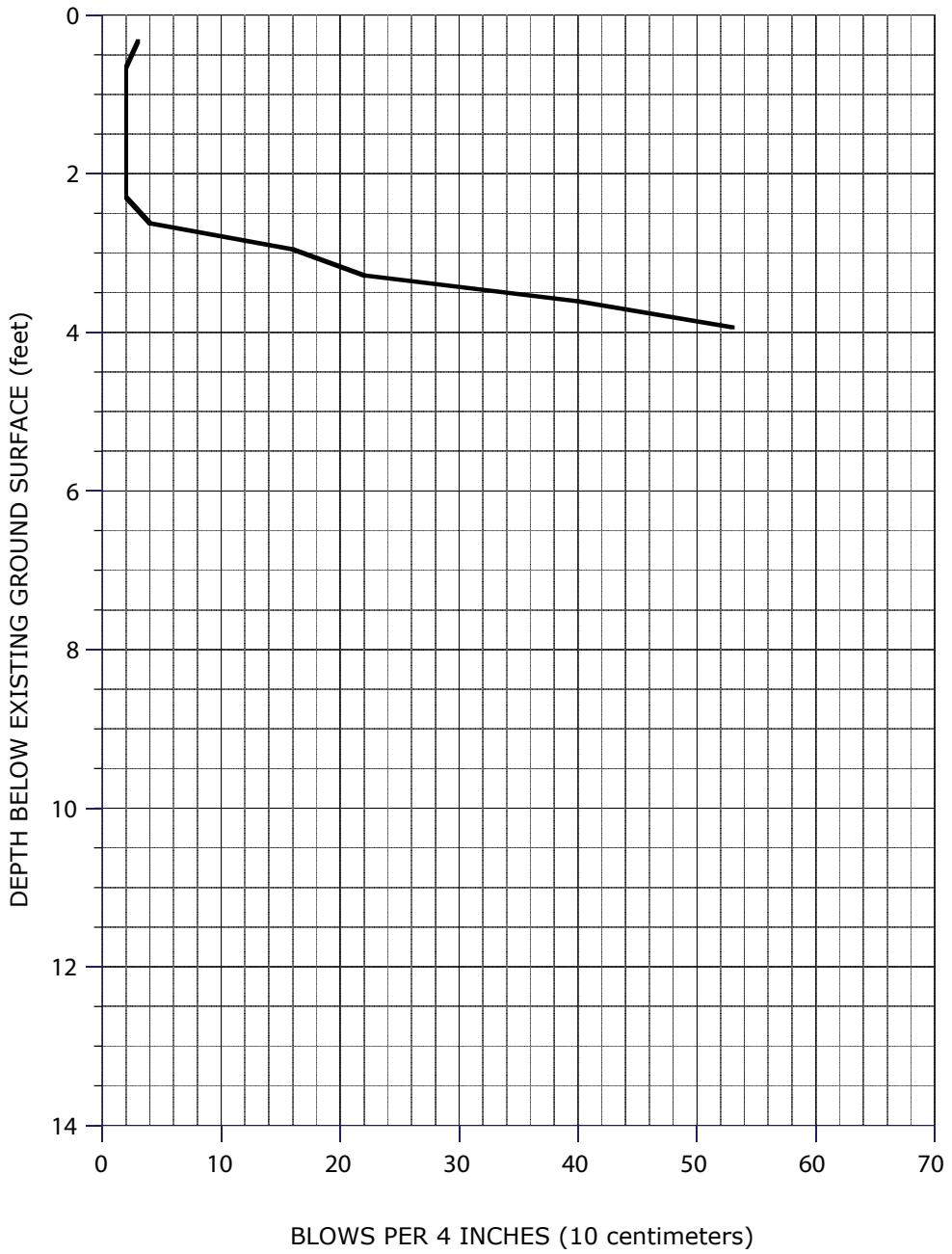
FIGURE	**A-9**



Notes:
1. DCPT-5 was performed near the front area of the proposed residence, see Site Plan, Figure 2.
2. Zero depth corresponds to approximately Elevation 10.5 Feet (NAVD 88).

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GEOTECHNICAL ENGINEERS & SCIENTISTS

DYNAMIC CONE PENETROMETER TEST RESULTS, DCPT-5
WRIGHT RESIDENCE
726 POINT SAN PEDRO ROAD
San Rafael, California

PROJECT No.	1693.1
DATE	07/13/21
FIGURE	**A-10**