Preliminary Geotechnical Investigation Report

ADU Addition

69 Starbuck Drive, Muir Beach, CA

APN: 199-201-03

Prepared for:

Greg Kidd

November 6, 2020

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November 6, 2020

Greg Kidd
Hard YAKA, Inc.
PO Box 1186
Crystal Bay, NV 89402

Re: Preliminary Geotechnical Investigation
69 Starbuck Drive, Muir Beach, CA
APN: 199-201-03
DAC Project No.: 1365-2320 M

As requested, we have performed a preliminary geotechnical investigation for the proposed new ADU to be located at 69 Starbuck Drive, in Muir Beach, California. This letter presents the results of our preliminary field investigation and preliminary engineering evaluation. The preliminary soil and foundation conditions are discussed and recommendations for the geotechnical engineering aspects of the project are presented. Conclusions and recommendations contained herein are based upon our preliminary study and must be verified in the field during construction phase of the project.

Introduction

Site and Project Description

This report presents the results of our preliminary geotechnical investigation for the proposed ADU at 69 Starbuck Drive, in Muir Beach, California. The vicinity map in Figure 1 shows the overall site location. Site coordinates are 37.8634 degrees north latitude and -122.5789 degrees west longitude. It is our understating that the project will consist of construction of a new stand-alone two story ADU on the northeast, downslope side of the existing house as shown on the site plan (Figure 2).

A drawing titled "HARD YAKA ADU" by Studio 300A Architecture, dated October 13, 2020, shows the existing conditions and the location of the proposed improvement project.

Purpose and Scope of Work

The purpose of our preliminary investigation was to determine overall characteristics of foundation soils within the proposed construction area and provide preliminary geotechnical recommendations concerning the proposed project. The scope of work of this study

included a site reconnaissance, review of available pertinent geologic and geotechnical information, and preliminary engineering analysis of the collected data, as well as preparation of this report. The data obtained and the analyses performed were for the purpose of providing preliminary geotechnical design and construction criteria for the new foundations and retaining walls.

This report has been prepared in accordance with generally accepted geotechnical engineering practices, and with our agreement with you for the exclusive use of yourself and your consultants for specific application to the proposed project. In the event that there are any changes in the ownership, nature, design or location of the proposed project, the conclusions and recommendations contained in this report shall not be considered valid unless 1) the project changes are reviewed by our office and 2) conclusions and recommendations presented in this report are modified or verified in writing.

Reliance on this report by another must be at their own risk unless we are consulted on its use or limitations. This study is purely a preliminary geotechnical investigation and it does not include any environmental examination or evaluation of the surface and/or subsurface conditions. We cannot be responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

Preliminary Field Investigation

Site Reconnaissance and Surface Conditions

Figure 1 shows the Vicinity Map of the project area, and Figure 2 shows the Site Plan indicating the existing house location and the proposed new construction. On November 6, 2020, we were present at site to observe existing site conditions and to evaluate the subsurface conditions from a geotechnical engineering standpoint.

The site is an irregularly shaped downslope property with maximum plan dimensions of 330 ft by 150 ft. Based on the site plan provided to us by Studio 300A Architecture, the site generally slopes toward the north and northwest with an overall slope gradient of about 5:1 (horizontal: vertical). Steeper slopes are present to the northwest with slope gradients as steep as 4:1. The site accommodates an existing two-story wood frame building reportedly built in 1984 according to County of Marin Assessor Record Detail.

During our November 6, 2020 site reconnaissance, we observed existing site conditions in consideration of the proposed development and potential geotechnical and soil related

issues relevant to the proposed project. We noted that three rows of retaining walls had been built within the proposed project area, and PVC pipes for the drainage system are located near retaining walls. We also observed sandstones and highly weathered bedrock exposed at about 2 to 3 feet below the level of surface grade, at the edge of the retaining wall footings.

Based on our site observation, we did not notice any evidence of major soil movements (such as land sliding) within the investigation area.

Vegetation consists of redwood, ornamental trees, and other types and sizes of trees, as well as shrubs and weeds. The site is bounded by adjoining properties on the north, and by Starbuck Drive on the east and south, as well as by private road on the west.

Subsurface Conditions

Based on our preliminary investigation, we assume that the near surface soils at the site consist of a 2-3 foot layer of fill and colluvium. Colluvium is derived from the underlying bedrock consisting of sandstone and claystone.

The general classification of the colluvial deposits and residual soils ranges from silty to clayey sand. The clay fraction of these materials has a medium plasticity and should be considered as moderately expansive.

Site Geology and Seismicity

The project site is located on the northeast of Muir Beach, near Redwood Creek. Based on the Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties (2000), prepared by D.L. Jones, M.C. Blake Jr., and R.W. Graymer (see Figure 3), the site is underlain primarily by Franciscan complex (fsr). According to Marin Map Viewer, the relative stability map of the proposed project (Figure 4) locates in an area considered the potential for most landslide. The above referenced relative stability maps, however, were developed based on the overall slope gradients and other geologic features on a larger scale, which would apply to the general site proximity. Site specific geotechnical investigation of the individual project areas would be necessary to provide evaluation of the subsurface conditions and stability of the slopes.

The Bay Area is considered a region of high seismic activity with numerous active and potentially active faults capable of producing significant seismic events. The U.S. Geological Survey (USGS) Working Group on California Earthquake Probabilities has evaluated the probability of one or more earthquakes occurring in the Bay Area and

concluded that there is currently a 63 percent likelihood of a magnitude 6.7 or higher earthquake occurring in the Bay Area by 2037.

The San Andreas and the Hayward faults are the two faults considered to have the highest probabilities of causing a significant seismic event in the Bay Area. These two faults are classified as strike-slip-type faults that have experienced movement within the last 150 years. The San Andreas Fault is a major structural feature in the region and forms a boundary between the North American and Pacific tectonic plates. Other principal faults capable of producing significant Bay Area ground shaking include the Calaveras fault, the Rodgers Creek fault, and the Concord–Green Valley faults. A major seismic event on any of these active faults could cause significant ground shaking and surface fault rupture, as was experienced during earthquakes in recorded history, namely the 1868 Hayward earthquake, the 1906 San Francisco earthquake, and the 1989 Loma Prieta earthquake. The estimated magnitudes (moment) identified in Table1 represent characteristic earthquakes on particular faults. In addition, active blind- and reverse-thrust faults in the region that accommodate compressional movement include the Monte Vista–Shannon and Mount Diablo faults.

Table 1. Active Faults In The Bay Area[1]

Fault	Recency of Movement	Historical Seismicity[2]	Maximum Moment Magnitude Earthquake (Mw)[3]
Hayward	1868 Holocene	M6.8, 1868 Many <M4.5	7.1
San Andreas	1989 Holocene	M7.1, 1989 M8.25, 1906 M7.0, 1838 Many	7.9
Rodgers Creek-Healdsburg	1969 Holocene	M6.7, 1898 M5.6, 5.7, 1969	7.0
Concord–Green Valley	1955 Holocene	Historic active creep	6.9
Marsh Creek-Greenville	1980 Holocene	M5.6 1980	6.9
San Gregorio–Hosgri	Holocene; Late Quaternary	Many M3-6.4	7.3
West Napa	2014 Holocene	M5.2 2000	6.0
Maacama	Holocene	Historic active creep	7.1
Calaveras	1990 Holocene	M5.6-M6.4, 1861 M4 to M4.5 swarms 1970, 1990	6.8

Mt. Diablo Thrust	Quaternary (possibly active)	n/a	6.7

Notes:

1. See footnote 4 of the text for definition of active faults.
2. Richter magnitude (M) and year for recent and/or large events. Richter magnitude scale reflects the maximum amplitude of a particular type of seismic wave.
3. The maximum moment magnitude earthquake (Mw), derived from the joint CGS/USGS Probabilistic Seismic Hazard Assessment for the State of California, 1996. (CGS OFR 96-08 and USGS OFR 96-706).
4. An active fault is defined by the State of California as a fault that has had surface displacement within Holocene time (approximately the last 10,000 years). A potentially active fault is defined as a fault that has shown evidence of surface displacement during the Quaternary (last 1.6 million years), unless direct geologic evidence demonstrates inactivity for all of the Holocene or longer. This definition does not mean that faults lacking evidence of surface displacement are necessarily inactive. "Sufficiently active" is also used to describe a fault if there is some evidence that Holocene displacement occurred on one or more of its segments or branches (Hart, E. W., Fault-Rupture Hazard Zones in California: Alquist-Priolo Special Studies Zones Act of 1972 with Index to Special Studies Zones Maps, California Geological Survey, Special Publication 42, 1990, revised 1997).

Sources: CGS, 1996, Hart, 1997; Jennings, 1997; Peterson, 1996, WGCEP, 2008.

The site is located approximately 2.6 miles from the San Andreas fault trace, 4.2 miles from the San Gregorio fault trace, and 15.0 miles from the Hayward fault trace. These faults are active and pose a high risk of strong ground shaking at the site. Figure 5 shows the locations of these and other faults relative to the project site. It should be assumed the site will probably be subjected to at least one moderate to severe earthquake that will cause strong ground shaking.

Preliminary Conclusions and Recommendations

Based on the results of our preliminary geotechnical study, it is our opinion that the site is feasible for the proposed new construction from a geotechnical engineering standpoint. The conclusions and recommendations presented in this letter, however, should be incorporated in the design of project to help minimize any possible soil and/or foundation related problems. In addition, we need to perform observation of the subsurface conditions during the construction phase of the project to verify or modify our preliminary recommendations presented in this letter.

Primary geotechnical considerations include (1) presence of incompetent near surface soil and colluviums, which are loose/ soft and not suitable for supporting foundations, (2) presence of moderately steep downslope gradients to the southeast, (3) potential for landslide at the proposed project area, and (4) presence clayey subsurface materials with moderate expansion potential.

Foundation Recommendation

As a preliminary recommendation, the proposed ADU should be supported by continuous and isolated spread footings on competent bedrock, or alternatively on prepared subgrade. The foundation excavations should penetrate a minimum depth of 12 inches into competent bedrock. Competent bedrock should be verified by the geotechnical engineer at the time of construction.

Prepared subgrade should be provided by overexcavating a minimum of 18 inches below the proposed ADU footing and backfilling with a layer of engineered fill. The width of the overexcavation trench should be 3 feet wider than the width of the footing. Engineered fill consists of granular non-expansive soil compacted to 95 percent relative compaction. The proposed new foundations should be constructed at a minimum depth of 18 inches below lowest adjacent subgrade. Also, footing subgrade preparation should be observed by the geotechnical engineer.

Lateral load resistance of new ADU components can be developed by friction between the bottom of footings and competent subgrade soils. If additional resistance is required for design against lateral loads, passive pressure against vertical face of footing in bedrock may be used in conjunction with the frictional resistance. The value of passive pressure is provided in Table 2 below.

The allowable bearing pressures on competent bedrock or on prepared subgrade for the footing foundations designed and constructed in accordance with our geotechnical recommendations are presented in Table 2 below. When determining the lateral load resistance for the footing, the resistance of materials overlying bedrock should be ignored.

As a second alternative option, the new ADU could also be supported on a system of drilled piers and grade beams. The recommendations for drilled pier and grade beam are provided later in this report.

Retaining Walls Recommendation

The concrete retaining walls should have a minimum thickness of 8 inches and extend a minimum depth of 18 inches below the lowest adjacent subgrade. Concrete retaining walls should be designed to resist both lateral earth pressures and any additional lateral loads caused by surcharging. The designer should include appropriate surcharge loads if required in retaining wall design. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained or restrained walls, respectively. Lateral load criteria for design of

retaining walls are presented in Table 2 below. In addition, 1 pcf should be added per every 2 degrees of backfill horizontal slope angle.
To prevent hydrostatic pressure buildup, the retaining walls should be provided with permanent backdrains. The above lateral pressures also assume drained conditions. Subdrains should consist of a vertical blanket of Class 2 permeable material, a minimum of 1 foot thick and a 4-inch-diameter perforated pipe (SDR 35). The perforated pipes should have two rows of holes and be placed holes-down. The permeable blanket should extend up to about 1 foot of finished ground surface at the top. Subdrain pipes from behind the walls should be connected to solid collector pipes that outlet to an appropriate discharge point. In lieu of perforated pipes and solid collector pipes, the retaining walls may be provided with weep holes. Weep holes should be located no more than 1 foot above grade in front of the wall and be at least 3 inches in diameter and no more than 5 feet apart on center.

Excavation for the retaining wall should conform to applicable state and federal industrial worker safety requirements. Where the excavation is more than 5 feet deep, the excavation wall may need to be sloped and/or shored.

The excavation for the retaining wall should be backfilled with properly compacted engineered fill, up to design finish subgrade. Soil backfill should be placed in level lifts about 6 to 8 inches in loose thickness, moisture conditioned, and mechanically compacted to at least 90% relative compaction. If backfill is not well compacted, considerable settlement may be anticipated. Retaining wall backfill should be placed after concrete has attained sufficient strength to resist the active pressures. The structural engineer should determine the required age of concrete before backfilling operation.

Table 2 - Recommended Geotechnical Design Parameters

Spread Footing Foundation	Allowable bearing pressure: Dead Load Allowable bearing pressure: Dead + Live Load Allowable bearing pressure: All Load [3] Allowable passive pressure resistance in competent bedrock[4] Friction coefficient b/w bottom of footing and bedrock	2000 psf[1] 3000 psf[1] 4000 psf[1] 400 pcf[2] 0.3
Drilled Pier Foundation	Minimum pier diameter Minimum embedment into competent bedrock Allowable skin friction in competent bedrock: compression Allowable skin friction in competent bedrock: tension Allowable passive pressure resistance in competent bedrock[4] Applied downslope creep load in overburden (4 feet) materials[4]	18 inch 6 feet 900 psf[1] 700 psf[1] 400 pcf[2] 65 pcf[2]

Retaining Walls	Active soil pressure for level backfill: unrestrained Active soil pressure for sloping backfill (25°)[5]: unrestrained Active soil pressure for level backfill: restrained Active soil pressure for sloping backfill (25°)[5]: restrained Allowable passive pressure resistance in competent bedrock	35 pcf[2] 41 pcf[2] 50 pcf[2] 56 pcf[2] 400 pcf[2]
[1] psf = pounds per square foot, [2] pcf = pounds per cubic foot, [3] including wind and seismic, [4] applied over two pier diameter, [5] angle measured with horizontal		

Grade Beam

If drilled pier and grade beam system is selected, all foundation systems need to be interconnected by using reinforced concrete grade beams with minimum cross-sectional dimensions of 12-inch square. In addition, as a minimum, grade beams should be reinforced with two #4 bars, top and bottom. The actual dimensions and reinforcement of the grade beams should be determined by the structural engineer.

Drilled Pier

Cast-in-Place concrete drilled piers shall derive their load bearing capacity from skin friction within the competent bedrock underlying colluvium. Competent bedrock is expected to be encountered at depths of about 2 to 3 feet within the proposed project areas of the site.

Drilled piers should have a minimum 18-inch diameter and should penetrate a minimum depth of about 6 feet into competent bedrock. In addition, the piers should have a minimum overall depth of about 9 feet below lowest adjacent subgrade. Minimum center-to-center spacing of piers should be three times pier diameter. Lateral load criteria for design of drilled pier is presented in Table 2 above.

As a minimum, concrete piers should be reinforced with 6#5 longitudinal and #3 shear ties spaced at 12-inch on centers. However, the actual design of the piers should be performed by the structural engineer and reviewed by the geotechnical engineer. Drilling of the piers shall be observed by a representative of our firm for verification or modification of the pier depths.

Additional Design and Construction Consideration

If a utility trench or another footing is located adjacent to a proposed foundation, the bottom of the foundation should be situated below an imaginary line drawn from the

bottom corner of the adjacent trench or footing, projected upward at a 30 degree angle with horizontal.

The drilling contractor should be aware of the presence of intervals of potentially "hard rock" conditions where rock coring may be required. In addition, pier excavations may extend below the water table and water may be entering the holes. Under such conditions, we recommend that the concrete be placed in the bottom of the hole using tremie methods. Alternatively, if the water can be pumped from the hole without causing instability in the pier shaft walls, concrete may be placed in the dry hole without the use of a tremie pipe.

The rebar cages should be secured against lateral movement during placement of concrete in the pier holes by installing dobies or spaces. Concrete for the piers should be designed with a high slump equal to or greater than 6 inches to facilitate construction and help minimize the potential for development of air or water filled voids in the pier excavation. Concrete should be placed in all piers the same day that their excavations are completed.

Slab-on-grade Recommendations

Concrete slab-on-grade structures should be supported on prepared subgrade. In areas where competent bedrock is exposed, the subgrade should be cleaned and made smooth and even. A 4-inch layer of compacted class 2 aggregate base should be provided below the slab. The concrete slab-on-grade should have a minimum thickness of 4 inches and as minimum be reinforced with a biaxial grid of #4 bars at 18-inch on centers. The design of the slab should be done by the project structural engineer.

For interior slab-on-grade, if migration of moisture through the slab is undesirable, a moisture barrier or capillary break should be provided between the slab and subgrade. We recommend that the moisture barrier consist of 4 inches of free draining gravel (drain rock) covered with an impermeable membrane (10-mil visqueen or equivalent). The membrane should be covered with 2 inches of sand for protection against tearing and puncture during construction. The sand should be lightly moistened just prior to placing the concrete. The drain rock should be placed on a properly moisture conditioned and compacted subgrade that has been approved by the geotechnical engineer. Alternatively, a capillary break consisting of 6 inches of free draining gravel (drain rock) could be used.

In lieu of a 10-mil visqueen, we recommend using a heavy duty (Stego wrap or approved equivalent) minimum 15-mil plastic membrane vapor barrier in conformance with the class A requirements outlined in ASTM Test Method E 1745. The membrane should be placed per

ASTM Test Method 1643 over the drain rock. Joints and penetrations should be sealed with the manufacturer-recommended adhesive, pressure-sensitive tape, or both.

Temporary Excavation Shoring

If the continuous and isolated spread footings are selected for support of the new structural loads, excavations on the order of about 4 to 5 feet deep would be anticipated for construction of the new foundations on competent bedrock. In this case, we recommend that the contractor be aware that in no case should slope height, inclination, and excavation depths exceed those specified in local, state, or federal safety regulations. Specifically, the contractor needs to be aware of the current OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926. We understand that these regulations are strictly enforced, and if they are not closely followed, the owner, contractor, and/or his earthwork and utility subcontractors could be liable for substantial penalties.

Alternatively, in lieu of open excavation method which limits the maximum excavation slope gradients, the construction excavations could be supported by temporary shoring to allow vertical cuts. Temporary shoring must be designed by a specialized shoring contractor.

The soil materials overlying competent bedrock may be considered as firm to stiff clay. This is considered to be a Type B material when applying the OSHA regulations. OSHA recommends the excavation on a slope less steep than four horizontal to one vertical (4H:1V) for Type B materials. This criterion can be applied to excavations that are above the groundwater level. Below groundwater level the excavation needs to be supported by properly designed and constructed temporary shoring. It is important to note that the soils to be penetrated by the proposed excavation may vary across the site and may require flatter slopes to remain stable.

The contractor's "responsible person" should establish a minimum lateral distance from the crest of the slope for all vehicles, equipment, and spoil piles. Likewise, the contractor's "responsible person" should establish protective measures for exposed slope faces.

We recommend that the contractor or his specialty subcontractor design temporary construction slopes to conform to the OSHA's "Guidelines for Excavations and Temporary Shoring". The temporary slope inclination should be determined by the contractor or responsible subcontractor based on the soil conditions exposed at the time of construction. We recommend that our office have the opportunity to observe all excavated slopes for conformance with the anticipated soil conditions. This will provide an opportunity to monitor the soil types encountered and to recommend modifying the excavation slopes as necessary. It also offers an opportunity to assess the stability of the excavation slopes

during construction.

Drainage and Erosion Protection Recommendations

All roof gutters and downspouts on the building should be connected to a drainage system that conducts the stormwater runoff to an appropriate discharge point(s) away from the building foundations. In addition, ground surface should be sloped away from building foundations with minimum slope gradients on the order of about 5 percent and for a minimum distance of about 10-feet measured from the building footprint. Impervious surfaces within 10 feet of the foundation shall be sloped a minimum 2 percent away from the foundation.

We recommend perimeter and intermediate subdrains to collect potential groundwater flows, especially from the upslope side of the building. Collected water shall be discharged away from the building area.

The groundwater collected from retaining wall backdrains and other subdrains should also be collected in solid pipes and directed to the designated discharge points. However, under no circumstance should surface runoff flows be directed into the subdrains.

It is our understanding that drainage water would be discharged on the surface within the site boundaries. Regardless of the discharge location, the discharge flows should be dispersed in such a way that protects the natural (unprotected) slope from erosion. This can be achieved by filtration of the surface runoff flows through a catch basin followed by a dissipation/ discharge system. The discharge facility may consist of a horizontal trench, backfilled with coarse gravel (1 to 2 inch in size) enveloped in filter fabric. The drainpipe should be a closed ended 4-inch diameter, horizontal, perforated pipe (SDR 35 or schedule 40) with perforation facing up. The location of the dispersion pipes should be away from building foundations and retaining walls. The dispersion location should also be verified by the geotechnical engineer during the construction phase of the project.

Exterior Flatwork

Exterior concrete flatwork should be supported directly on prepared subgrade. Flatwork placed on expansive clayey soils will be subject to movement due to soil expansion. From a cost-benefit standpoint, exterior concrete flatwork subgrades exposing expansive soil should be moisture conditioned to at least 3 percent above optimum moisture content to a depth of at least 12 inches to induce some soil expansion before placement of concrete, thus reducing the amount of post-construction soil expansion.

Review of Construction Plans and Specifications

We recommend that we review the final design and specifications to check that the earthwork and foundation recommendations presented in this letter have been properly interpreted and incorporated into the design and construction specifications. We can assume no responsibility for misinterpretation of our recommendations if we do not review final project plans and specifications.

Wet-Weather Construction Recommendation

If construction proceeds during or shortly after wet weather conditions, the moisture content of the on-site soils could appreciably increase leading to potential slope stability problems. Consequently, working at the site may become difficult and even hazardous. In addition, construction excavations may become exposed to accumulated standing runoff water, which may adversely impact the project. Wet weather construction recommendations can be provided by the geotechnical engineer in the field at the time of construction, if appropriate.

Seismic Design Parameters

We have obtained site-specific spectral seismic design parameters in accordance with the 2010 ASCE-7 (w/March 2013 errata). These design parameters are for use by the structural engineer in designing the house addition for potential seismic shaking.

Table 3. Seismic design parameters.

Parameter	Value
S_{MS}, for 0.2-second period	1.995g
S_{M1}, for 1.0-second period	1.408g
S_{DS}, for 0.2-second period	1.330g
S_{D1}, for 1.0-second period	0.938g

Based on specific site location, by latitude and longitude, S_S and S_1 are 1.995g and 0.938g, respectively. These values were obtained online from a seismic design tool provided by Structural Engineers Association of California, assuming a Site Class D. Based on subsurface conditions encountered in our boring, we classified the site as Site Class D for seismic design parameters, corresponding to a *Stiff Soil*.

Additional Services

Additional geotechnical engineering services will be needed for design and construction of the project. These include plan review, and responses to plan-check comments, and construction observations by our firm.

Our firm can provide engineering services for the above tasks. In addition, we should be accorded the opportunity to review the final plans and specifications to determine if the recommendations of this report have been implemented in those documents. Results of the review should be summarized in writing.

To a great degree, the performance of the site improvement depends on construction procedures and quality. Therefore, we should perform site visits to observe the contractor's procedures and the foundation soils, together with field testing during excavation. These observations will allow us to check the contractor's work for conformance with the intent of our recommendations and to observe any unanticipated soil conditions that could require modification of our recommendations. In addition, we would appreciate the opportunity to meet with the contractor before the start of construction to discuss the procedures and methods of construction. This can facilitate the performance of the construction operation and reduce possible misunderstandings and construction delays.

Closure and Limitations

Submittal of this letter completes the current scope of our preliminary geotechnical study for the project. By accepting this report, the recipients acknowledge their understanding of conditions described below.

Conclusions and recommendations contained herein are based upon our preliminary study and are contingent upon our verification/modification in writing, pending the results of our field observations during construction phase of the project. For construction observation scheduling, our firm must be notified at least three business days in advance.

The analysis, designs, opinions, and recommendations submitted in this letter are based in part upon the preliminary geotechnical data that was collected, and upon the conditions existing when services were performed. Variations of subsurface conditions from those analyzed or characterized in this report are possible as may become evident during construction. In that event it may be necessary to revisit certain analyses or assumptions.

This report has been prepared for the exclusive use of Greg Kidd and Denise Landucci, and their consultants for specific application to the proposed addition as described herein. Our services consist of professional opinions and conclusions developed in accordance with generally-accepted geotechnical engineering principles and current standards of practice. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided to us pertaining to the proposed construction, and on the results of our preliminary site investigation, as well as our engineering analyses and our professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of project construction.

Changes in the surface and subsurface conditions may occur as a result of natural/environmental changes or human activities. Site conditions and site features described herein are those existing at the time of our field observation and may not necessarily be the same or even comparable at other times. Therefore, the validity of subsurface conditions and our recommendations should be reviewed and confirmed by our firm after a period of 12 months from the date of issuance of this preliminary report.

Our investigation did not include any environmental assessment or investigation of the presence or absence of hazardous, toxic, or corrosive materials in the soil, surface water, ground water, or air-on, below, or around the site, nor did it include an evaluation or investigation of the presence or absence of ecologically sensitive features. In addition, we did not perform any assessment or evaluation of the existing structures either from the environmental standpoint concerning the composition of onsite construction materials or integrity/stability of the facilities and building components.

We appreciate the opportunity of providing you with our engineering services. If you have any questions or require additional information, please do not hesitate to contact us.

Sincerely,
DAC Associates, Inc.

Darius Abolhassani, P.E., G.E. *(Principal)*
C58778, GE2648

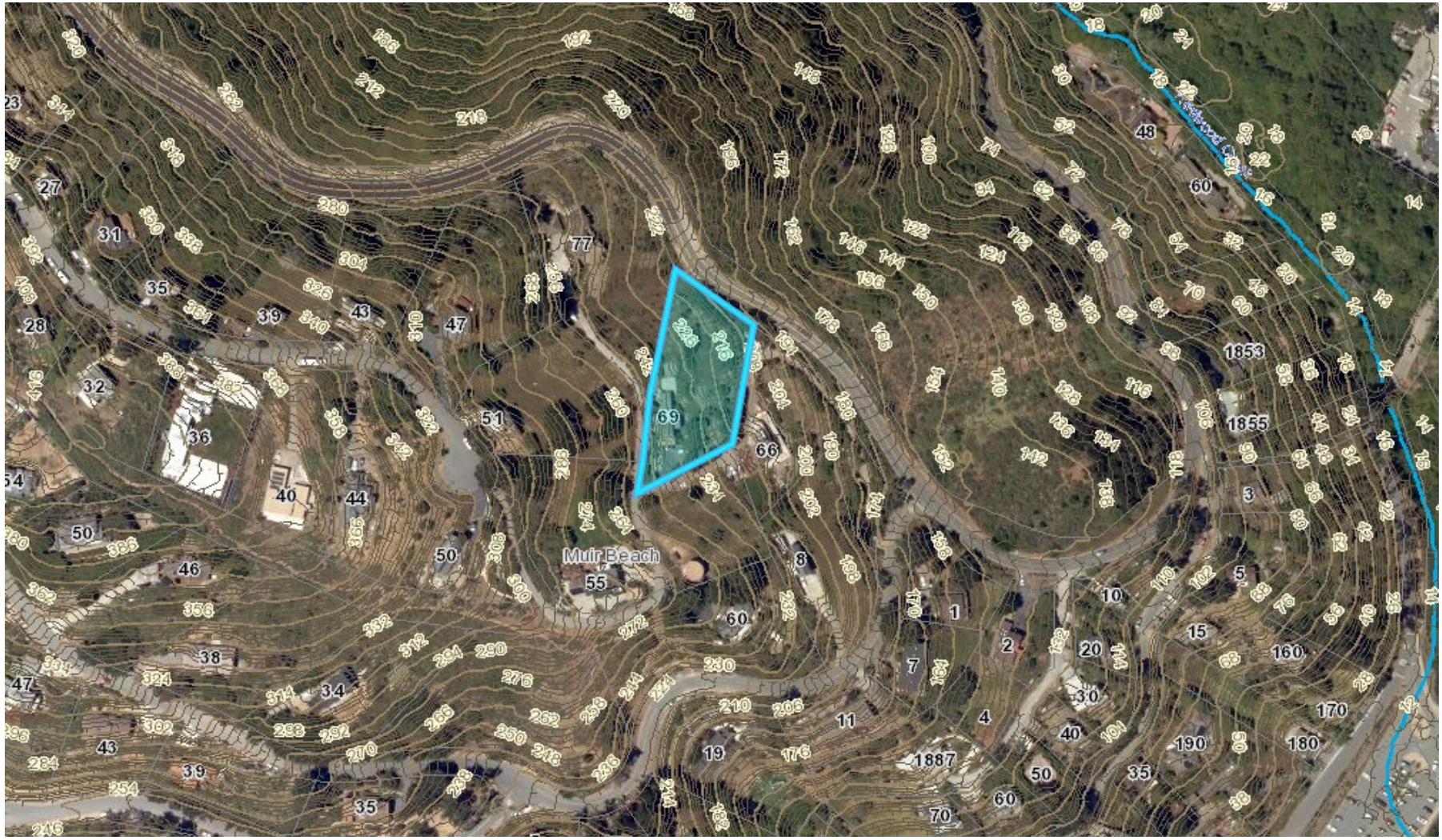
Attachments:
- **Figure 1 – Vicinity Map**
- **Figure 2 – Site Plan**
- **Figure 3 – Geologic Map 2000**
- **Figure 4 – Relative Stability Map**
- **Figure 5 – Fault Map**

References

Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, Scale 1: 75,000
By D.L. Jones, M.C. Blake Jr., and R.W. Graymer, (2000)

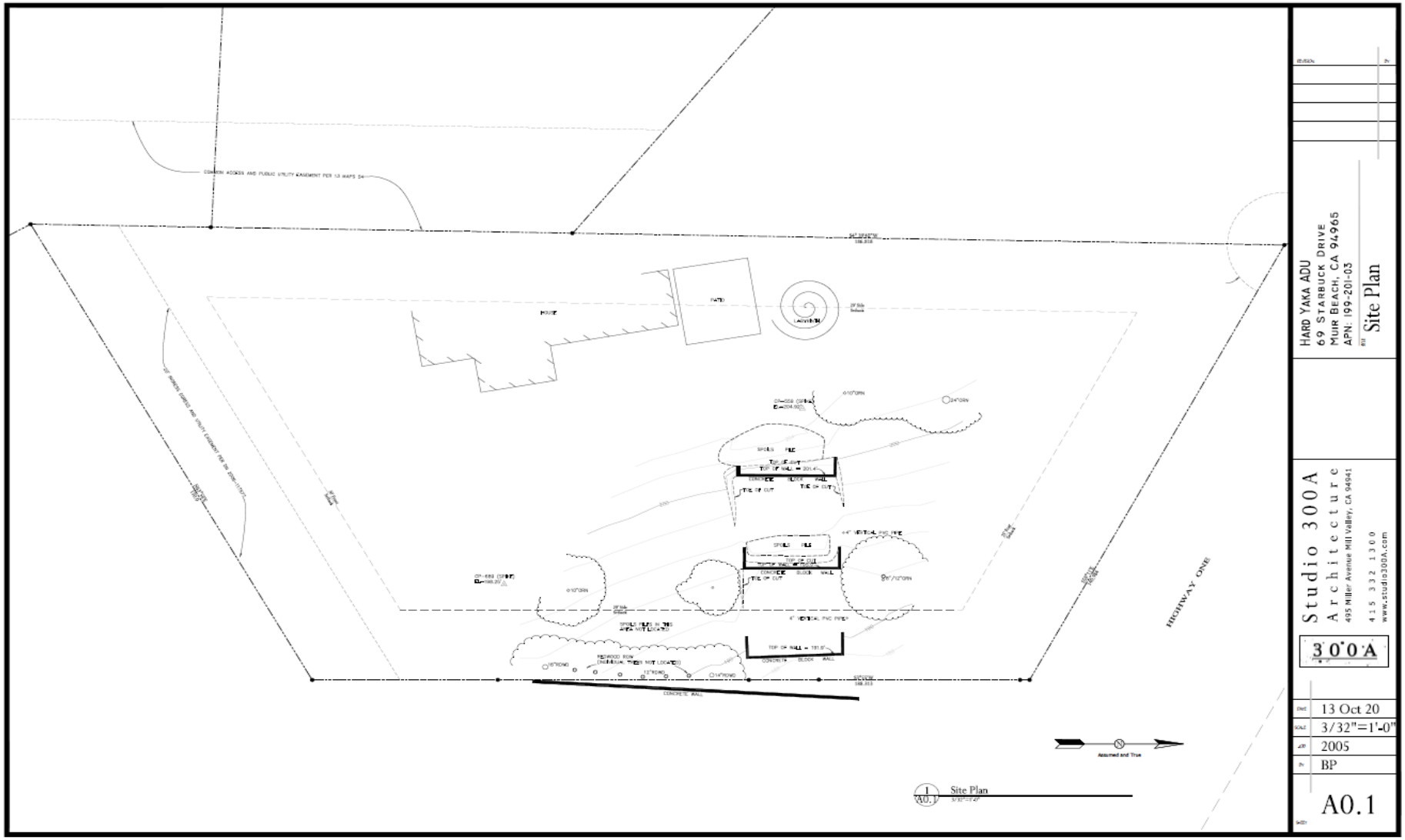
Marin Map Viewer
https://www.marinmap.org/Html5Viewer/Index.html?viewer=smmdataviewer

Structural Engineers Association of California, Seismic Design Tool
https://seismicmaps.org/



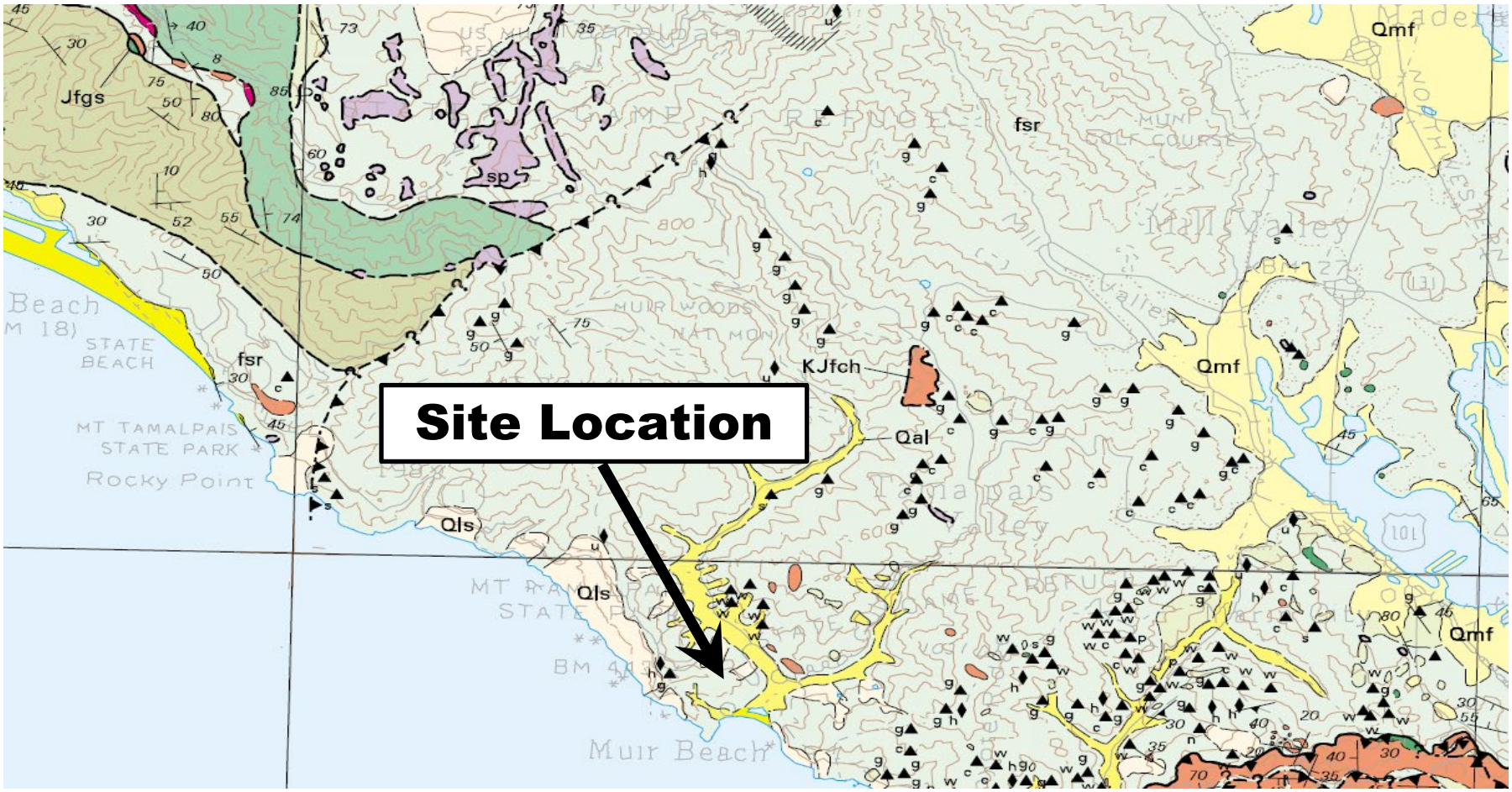
Marin Map Viewer (Layers include Aerial Ortho and contour elevations)

DAC	**Vicinity Map**	Report Date:	**November 2020**	**Figure 1**
	ADU Addition **69 Starbuck Drive, Muir Beach, CA** **APN: 199-201-03**	Reviewed By:	**DA**	
		Proj. Manager:	**DA**	
		Job No.:	1365-2320 M	



DAC	**Site Plan**	Report Date:	**November 2020**
	ADU Addition **69 Starbuck Drive, Muir Beach, CA** **APN: 199-201-03**	Reviewed By:	**DA**
		Proj. Manager:	**DA**
		Job No.:	1365-2320 M

Figure 2



fsr = Franciscan Complex
Qal = Alluvium
Qls = Landslide deposits

N

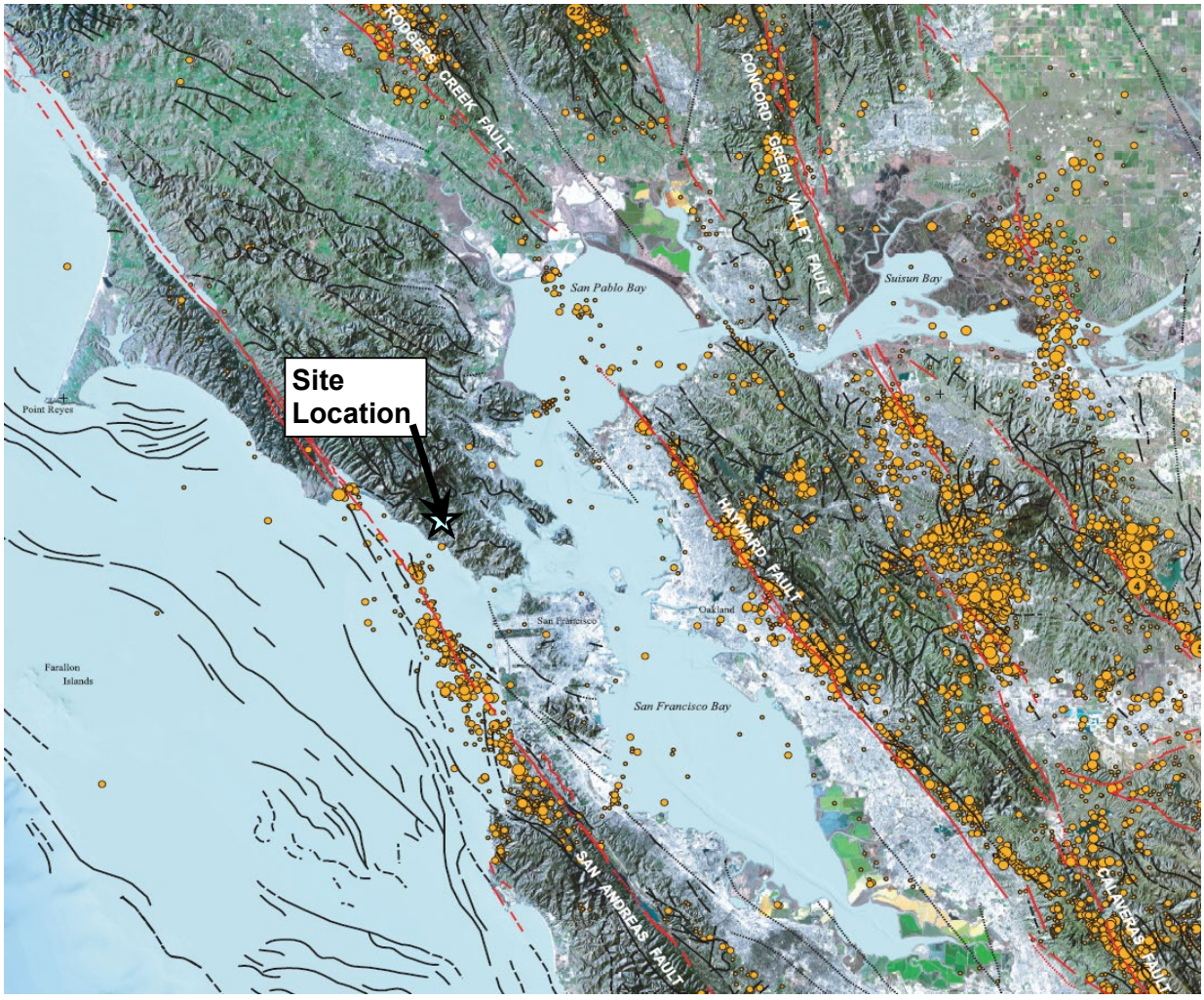
Source:D.L. Jones, M.C. Blake Jr., and R.W. Graymer, "Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California" 2000, Map Scale 1:75,000

DAC	**Geologic Map**	Report Date:	**November 2020**	**Figure 3**
	ADU Addition **69 Starbuck Drive, Muir Beach, CA** **APN: 199-201-03**	Reviewed By:	**DA**	
		Proj. Manager:	**DA**	
		Job No.:	1365-2320 M	

Marin Map Viewer. (Layers include Hazards and Geology, Landslide)

DAC	**Relative Stability Map**	Report Date:	**November 2020**	**Figure 4**
	ADU Addition **69 Starbuck Drive, Muir Beach, CA** **APN: 199-201-03**	Reviewed By:	**DA**	
		Proj. Manager:	**DA**	
		Job No.:	1365-2320 M	

Major Faults in the San Francisco Bay Area



EXPLANATION

EARTHQUAKES

MAGNITUDE (RICHTER)

- 1.5 - 2.0
- 2.1 - 3.0
- 3.1 - 4.0
- 4.1 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0

14 Number refers to earthquake number in table

BATHYMETRY

Sea level

3,650 Meters

FAULTS – Dashed where approximately located; dotted where inferred

Active in last 700,000 years

Active prior to 700,000 years ago

Source: Sleeter, B.M., Calzia J.P., Walter S.R., Wong F.L., and Saucedo G.J., "Earthquakes and Faults in the San Francisco Bay Area (1970-2003), 2004, Map Scale 1:300,000

DAC	**Fault Map**	Report Date:	**November 2020**	**Figure 5**
	ADU Addition **69 Starbuck Drive, Muir Beach, CA** **APN: 199-201-03**	Reviewed By:	**DA**	
		Proj. Manager:	**DA**	
		Job No.:	**1365-2320 M**	