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Revised Geotechnical Investigation Report

Altman Residence

63 Tamalpais Road

Fairfax, CA

APN: 001-123-03

Prepared for:

Stephen Altman

June 21, 2022

Geotechnical, Structural, Civil Engineering & Construction Support
7 Mt. Lassen Drive, Suite A-129, San Rafael, CA 94903
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ATTACHMENT B

DAC
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June 21, 2022

Mr. Stephen Altman
63 Tamalpais Road
Fairfax, CA 94930

Re: Revised Geotechnical Investigation
Altman Residence
63 Tamalpais Road, Fairfax, CA
APN: 001-123-03
DAC Project No.: 1428-3321G

Dear Mr. Altman:

As requested, we have performed a geotechnical investigation for the proposed new residence to be located at the above address, in Fairfax, California. This revised report presents the results of our review of readily available geologic and geotechnical information pertaining to immediate site proximity as well as our exploratory work performed at the site. The soil and foundation conditions are discussed and recommendations for excavation and earthwork operation, foundation and retaining wall design and construction, as well as geotechnical drainage of the project are presented. Conclusions and recommendations contained herein are based on applicable standards of our profession at the time this report was prepared. Copies of this letter report are furnished only to provide the factual data that were gathered and summarized.

INTRODUCTION

Site and Project Description

This report presents the results of our geotechnical investigation for the proposed new residence at 63 Tamalpais Road, in Fairfax, California. The vicinity map in Figure 1 shows the overall site location. Site coordinates are 37.9892 degrees north latitude and -122.5970 degrees west longitude. The purpose of our investigation was to evaluate the foundation soils and provide geotechnical recommendations concerning the proposed project.

A drawing titled *'Altman Residence, 63 Tamalpais Road, Fairfax, CA 94930'* by Kappe Architects, dated June 11, 2021, shows the location of the proposed project. Based on our review of the architectural plans, it is our understanding that the project will consists of construction of a new two-story building over a below grade garage at the street level. The development of the project involves cut slopes up to about 21 feet in height supported by retaining walls.

Purpose and Scope of Work

The purpose of our geotechnical investigation was to determine overall characteristics of foundation soils within the proposed construction area and provide geotechnical recommendations concerning the proposed project. Our scope of work was as follows:

1. Drill two exploratory boreholes using portable (minuteman) drilling or a track mounted drill equipment with 4-inch diameter solid stem augers and/or using a hand auger if appropriate, within the immediate proximity of proposed construction to maximum depths of about 14 to 16 feet below grade or to competent subgrade, whichever is encountered first.

2. Perform limited geotechnical field and laboratory tests on selected samples of the soils obtained from the test borings as deemed necessary.

3. Develop geotechnical conclusions and recommendations and design parameters for the foundations and retaining walls, including allowable soil bearing pressures for footings, friction resistance for drilled concrete piers, active and passive soil pressures, as well as seismic design parameters.

4. Provide recommendations for excavation and earthwork operations, as well as geotechnical drainage as applicable to the proposed construction.

5. Prepare a geotechnical engineering report summarizing our findings, conclusions, and recommendations.

This report has been prepared in accordance with generally accepted geotechnical engineering practices, and with our agreement with you for exclusive use of yourself and your consultants for specific application to the proposed project. In the event there are any changes in the ownership, nature, design or location of the proposed development, 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 others must be at their own risk unless we are consulted on its use or limitations. This study is purely a geotechnical investigation and it does not include any environmental examination or evaluation of the surface and/or subsurface conditions. We cannot be responsible for impacts of any changes in engineering and 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.

FINDINGS

Site Reconnaissance and Surface Conditions

Figure 1 shows the vicinity map of the project area, and Figure 2 shows the site plan indicating the proposed project. On July 16, 2021, we were present at the site to observe existing site conditions, drill two exploratory borings, collect soil samples, and perform field tests for evaluation of soil properties from a geotechnical engineering standpoint.

The general site parcel is irregularly shaped, located on an uphill sloped terrain with maximum plan dimensions of about 60 feet by 97 feet. Based on available topographic information, the site generally slopes up towards the south and southwest with an overall slope gradient of about 2.6:1 (horizontal: vertical). Steeper slopes are present to the south, with slope gradients as steep as 1.2:1.

During our July 16, 2021, site reconnaissance, we observed the existing site conditions in consideration to potential geotechnical and soil related issues relevant to the proposed project. We also noted that two parking decks supported by a post and pier system had been built on the uphill slope on the south side of the property at Tamalpais Road level.

During our site reconnaissance, we noticed evidence of minor soil movements within the investigation area. On the north side of the property, within 10 feet of Tamalpais Road, a small but steep, sudden drop of 3-4 feet suggests potential slippage of overburden layer towards Fairfax Creek.

Vegetation consists of oak, bay and different types and sizes of trees, as well as shrubs and weeds. The site is bounded by adjoining properties on east and west, by Tamalpais Road on the north and the south.

Subsurface Conditions

On July 16, 2021, we drilled, logged, and sampled two exploratory borings using a portable rig (minuteman) in the areas of the proposed project to evaluate subsurface soil conditions and estimate depth of weathered bedrock. Figure 2 shows the approximate boring locations on the site plan. Boring BG-1 was drilled in close proximity to the proposed rear patio, near the existing parking deck. Boring BG-2 was located at the north side of the proposed house, close to the property line.

Our test boring BG-1 encountered a 1-foot layer of fill followed by a 2-foot layer of colluvium. Competent weathered bedrock was encountered at about 3 feet below the surface grade, and drilling of boring was terminated at 10-1/2 feet. In our boring BG-2, the subgrade consisted of a 3-foot layer of fill followed by a 3-foot layer of colluvium overlying highly weathered bedrock. Drilling was terminated at 10-1/2 feet.

The general classification of the colluvium ranges from silty to clayey sand. The clay fraction of these materials has a medium plasticity and should be considered as moderately expansive. The logs of our borings are presented in Appendix A.

Site Geology and Seismicity

Based on the Geologic Map of the Upper Ross Valley and the Western Part of the San Rafael Area

Marin County (1976), prepared by Smith, Strand, and Rice, (see Figure 3), the site is underlain primarily by colluvium (Qc) over Franciscan sandstone and shale bedrock. According to Interpretation of the Relative Stability of Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area Marin County by the same authors, the site is located in an area classified as Zone 1 bordering Zone 4 (see Figure 4). Zones 1 through 4 have been designated with 1 corresponding to the most stable and 4 least stable. However, the above referenced relative stability map was developed based on the overall slope gradients and other geologic features on a larger scale, which would apply to the general site proximity.

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 6.7 miles from the San Andreas fault trace, 7.8 miles from the San Gregorio fault trace, and 11.2 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.

Conclusions and Recommendations

Based on the results of our geotechnical study, it is our opinion that the site is feasible for the proposed project from a geotechnical engineering standpoint. The conclusions and recommendations presented in this letter, however, should be incorporated into design and construction of the project to help minimize any potential soil and/or foundation related problems.

Primary geotechnical considerations to take into account in design and construction of the proposed project are the presence of incompetent near surface fill and colluvium, which are not suitable for supporting foundations, as well as presence of relatively steep site slopes and potential instability of unsupported and over steepened site slopes. Discussion of these important issues and other design considerations as well as recommendations for addressing them, are provided in detail below.

Foundation and Retaining Wall Recommendations

We recommend that the proposed house should be supported by a combination of continuous spread footings embedded into competent bedrock and/or drilled piers. A drilled pier and grade beam system should be used for structural support in areas where the overburden will not be removed. Recommendations for both systems are presented below.

Continuous Spread Footing

Within the areas where construction excavation would remove overburden materials, continuous spread footings may be used for support of retaining walls and building foundations. Such spread footings should be level on the bottom and should be embedded a minimum of 18 inches into competent. Allowable bearing pressure for competent bedrock could be considered as 3500 pounds per square foot (psf) for dead plus live loads and it may be increased by 1/3 under transient loads such as wind and seismic.

For lateral resistance, a passive equivalent fluid pressure of 400 pounds per cubic foot (pcf) can be considered to act on the portion of footing extending into competent bedrock or against properly compacted engineered fill. The top one foot of soil, however, should be ignored in determination of passive resistance, unless the grade is covered with a structural slab or pavement. In addition, a friction coefficient equal to 0.3 could also be considered to act between the bottom of footings and competent subgrade.

Concrete footings should have a minimum width and depth of 12 inches and be located at a minimum depth of about 18 inches below lowest adjacent subgrade or 18 inches into competent bedrock. Continuous perimeter footings should be reinforced with a minimum of 2 #4 longitudinal rebars. For the grade beam, we should have top and bottom with #3 ties. However, the actual design of the footings and grade beams should be performed by the structural engineer.

Drilled Pier

In areas of the site where construction excavation is not expected to be deep enough to expose competent bedrock, drilled piers should be used for structural support. Cast-in-Place concrete drilled piers shall derive their load bearing capacity in skin friction in competent bedrock. Competent bedrock is expected to be encountered at depths of about 6 feet below the surface grade.

Drilled piers should have a minimum 18-inch diameter and should penetrate a minimum depth of about 6 feet into competent bedrock. The piers should have a minimum overall depth of about 12 feet below lowest adjacent surface grade. The actual depth of piers should be determined by our firm during the construction period when we are present to observe pier drilling. The allowable skin friction for dead plus live loads in competent subgrade shall be taken as 1000 pounds per square foot (psf) in compression, and 800 psf in tension. These values may be increased by 1/3 under transient loads such as wind and seismic.

As a minimum, concrete piers should be reinforced with 4 #5 longitudinal and #3 shear ties spaced at 12-inch on centers. The actual design of the piers and grade beams, however, should be performed by the structural engineer.

Piers should be designed to resist structural loads as well as a soil creep pressure equivalent to a fluid pressure of 65 pounds per cubic foot (pcf) applied against two pier diameters. The lateral load capacity of piers installed per the above recommendations would be developed by passive soil pressure within the competent subgrade materials. The allowable passive soil pressure as referenced in this paragraph could be considered as an equivalent fluid pressure of 400 pcf acting against two pier diameters. The vertical and horizontal resistance of fill should be ignored.

The drilling contractor should be aware of 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.

Retaining Wall Recommendations

Unrestrained retaining walls should be designed to resist an active pressure equivalent to a fluid pressure of 40 pcf for level backfill. Restrained retaining walls should be designed to resist an earth pressure equivalent to a fluid pressure of 55 pcf for level backfill. The pressure due to compaction equipment should be considered as an additional surcharge load on the retaining wall. For sloped backfill add a 1 pcf for every 2-degree slope angle.

In addition to the lateral earth pressure, vertical uniform surcharge loads (qsur) in pounds per square foot (psf) behind retaining wall should be considered in development of lateral pressure. The minimum design surcharge load should be 0.35*qsur in psf with rectangular distribution on retaining wall. The pressure due to compaction should be considered as an additional vertical surcharge load of qsur = 100psf. Other construction surcharge pressures are dependent on contractor's operations, such as placement of cranes and storage of materials, and should be determined by the contractor.

In addition, for retaining walls supporting more than 6 ft of backfill, a seismic load should be also considered in development of the lateral pressure. The minimum design seismic load should be 20*H in psf with rectangular distribution, where H is the retained height in feet. However, the factor of safety against sliding and/or overturning under seismic conditions can be reduced to a minimum of 1.1.

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. Backfill behind the retaining walls should consist of soil placed in level lifts about 8 inches in loose thickness, moisture conditioned to about the optimum moisture content, and mechanically compacted to at least 85% relative compaction for landscape area or 95% relative compaction for building areas. Relative compaction refers to the in-place dry density of soil expressed as a percentage of maximum dry density of the same soil, as determined by ASTM Test Method D1557, latest version. In lieu of compacted backfill, the subdrain material may take up the entire space behind the retaining wall. The top of the wall should be provided with a concrete-lined V- or U-ditch.

If the continuous spread footings are selected to support the retaining walls, excavations on the order of about 6 feet deep would be anticipated for construction of the new foundations on competent native soils below the fill. 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 specialty shoring contractor.

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 actual design of the retaining walls and foundations should be done by the structural engineer. Please note that the resistance of materials overlying competent bedrock should be ignored in consideration of vertical and lateral load capacity of drilled pier and footing foundations.

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 Shoring Recommendations

The soil materials overlying competent bedrock may be considered as medium dense clayey sand. 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.

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 specialty shoring contractor.

Drainage and Erosion Protection Recommendations

All roof gutters and downspouts on the buildings 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, the ground surface should be sloped away from building foundations with minimum slope gradients of about 5% for a minimum distance of about 10 ft from the building footprint. Impervious surfaces within 10 ft of the foundation should be sloped a minimum 2% away from the foundation. Under no circumstance should surface runoff be directed into subdrains.
The groundwater collected from retaining wall backdrains and other subdrains should be collected in solid pipes and directed to the designated discharge points. Under no circumstance, however, should surface runoff flows be directed into the subdrains.

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 with minimum width of 12 inches and a maximum depth of about 18 inches, backfilled with coarse gravel (1 to 2 inch in size) enveloped in filter fabric. The drainpipe should be a closed ended 6-inch diameter 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.

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 Recommendations

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 and 2016 ASCE-7. These design parameters are for use by the structural engineer in designing the house addition for potential seismic shaking.

Table 2. Seismic design parameters (ASCE 7-10).

Parameter	Value
S_S, for 0.2-second period	1.500g
S_{MS}, for 0.2-second period	1.500g
S_1, for 0.2-second period	0.633g
S_{M1}, for 1.0-second period	0.949g
S_{DS}, for 0.2-second period	1.000g
S_{D1}, for 1.0-second period	0.633g

Table 3. Seismic design parameters (ASCE 7-16).

Parameter	Value
S_S, for 0.2-second period	1.500g
S_{MS}, for 0.2-second period	1.500g
S_1, for 0.2-second period	0.600g
S_{M1}, for 1.0-second period	NA
S_{DS}, for 0.2-second period	1.000g
S_{D1}, for 1.0-second period	NA

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 provide on-site soil observations of 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 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 geotechnical investigation including our exploratory work performed at the site. 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 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 Stephen Altman, and his 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 field exploration, 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.

Our boring logs only represent near surface conditions at the specific locations and on the dates they were excavated. It is not warranted that they are representative of such conditions elsewhere or at other times. Also, the locations of the test pits were determined in the field by reference to existing features, and should be considered approximate only.

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 exploration 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 month from the date of issuance of this 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 or 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

DARYOUSH DARIUS ABOLHASSANI
No. 2648

Attachments:
- **References**
- **Figure 1 — Vicinity Map**
- **Figure 2 — Site Plan**
- **Figure 3 — Geologic Map**
- **Figure 4 — Relative Stability Map**
- **Figure 5 — Regional Fault Map**

Appendix A —Boring Logs

References

County of Marin Assessor Record Detail
https://apps.marincounty.org/TaxRollSearch/Record?P=176-033-07

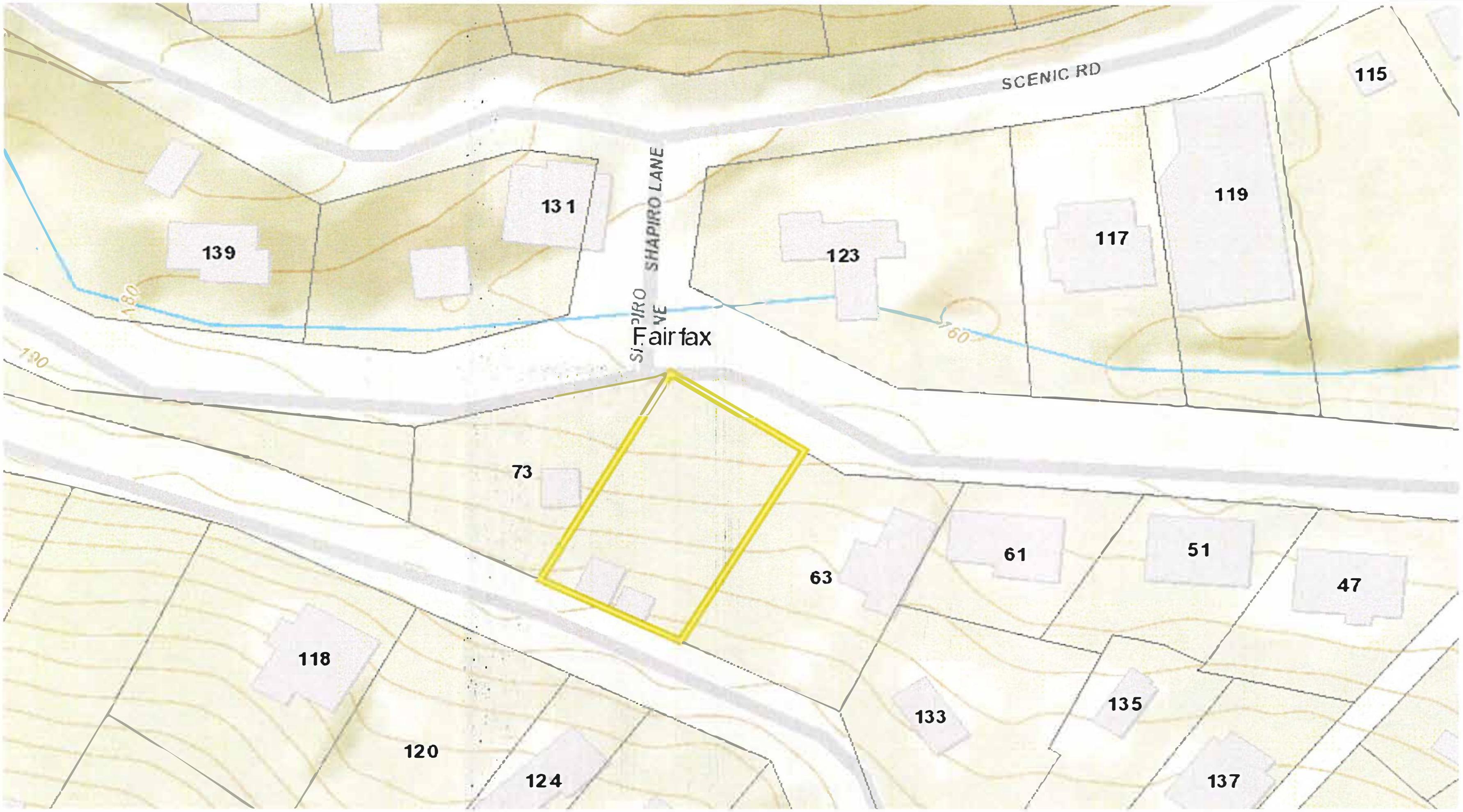
Geology of the Upper Ross Valley and the Western Part of the San Rafael Area Marin County, California, Scale 1: 12,000
By Rudolph G. Strand, Salem J. Rice and Theodore C. Smith, (1976)

Interpretation of the Relative Stability of Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area Marin County, California, Scale 1: 12,000
By Rudolph G. Strand, Salem J. Rice and Theodore C. Smith, (1976)

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

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

Figures



Marin Map Viewer

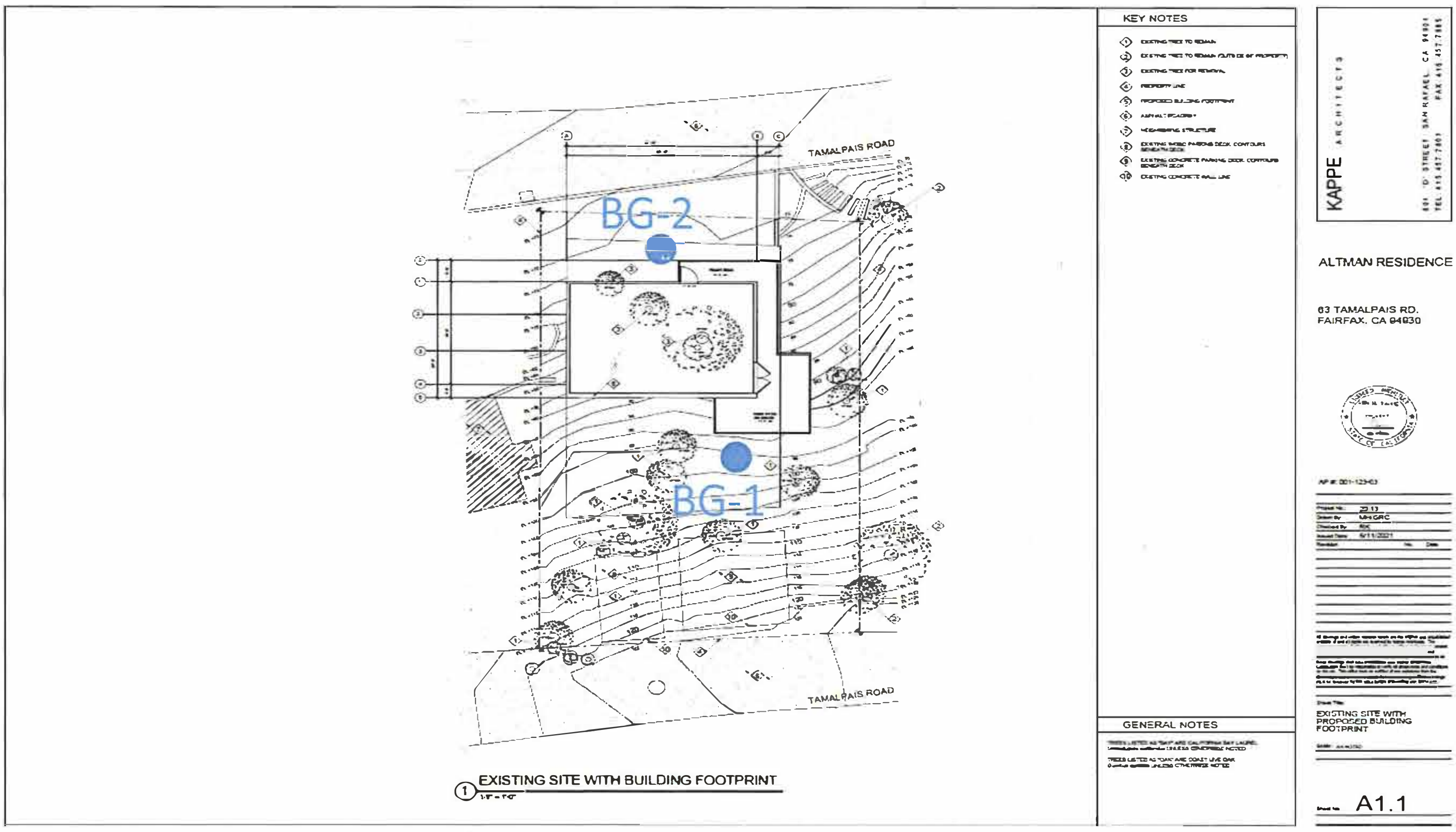
DAC

Vicinity Map

Altman Residence
63 Tamalpais Rd, Fairfax, CA
APN: 001-123-03

Report Date:	June 2022
Reviewed By:	DA
Proj. Manager:	DA
Job No.:	1428-3321G

Figure 1



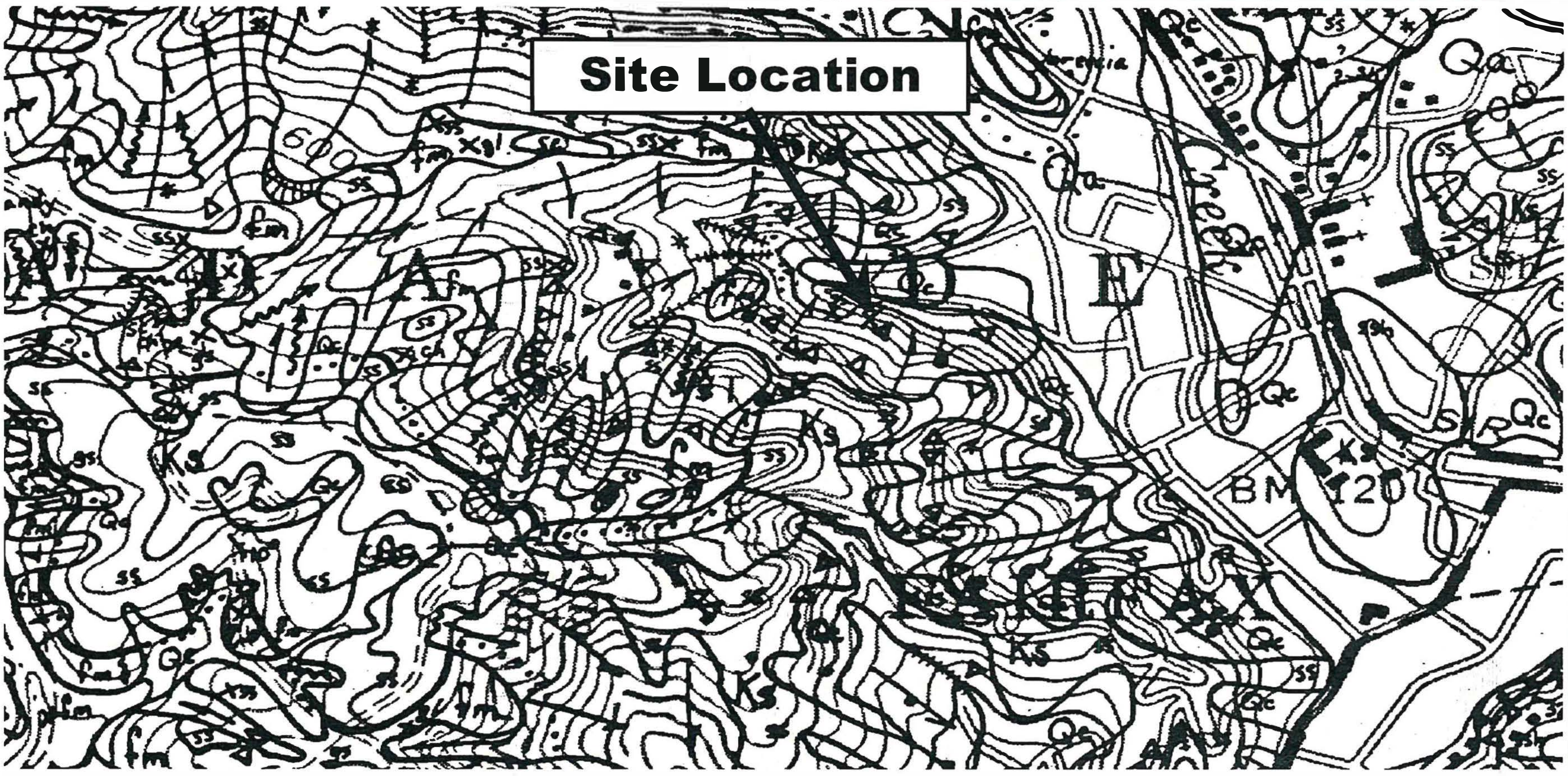
BG = Boring Location

Site Plan

Altman Residence
63 Tamalpais Rd, Fairfax, CA
APN: 001-123-03

Report Date: June 2022
Reviewed By: DA
Proj. Manager: DA
Job No.: 1428-3321G

Figure 2



Qc = Colluvium
fm = Franciscan Melange

Source: Rice, S.J., Strand, R.G., and Smith, T.C., "Geology of the Upper Ross Valley and the Western Part of the San Rafael Area Marin County, California"
Plate 1B, 1976, Map Scale 1:12,000

Geologic Map

Altman Residence
63 Tamalpais Rd, Fairfax, CA
APN: 001-123-03

Report Date:	**June 2022**
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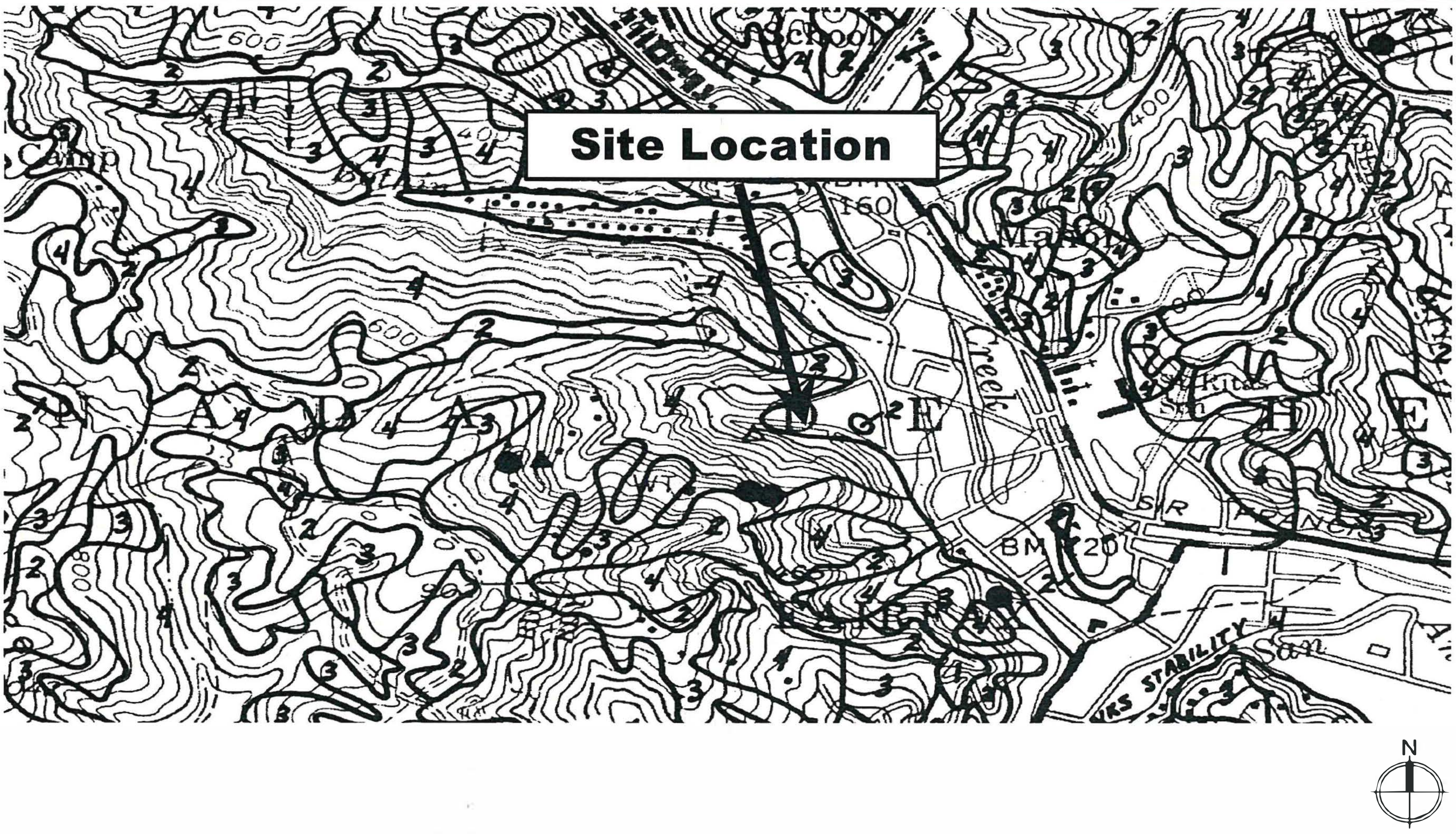
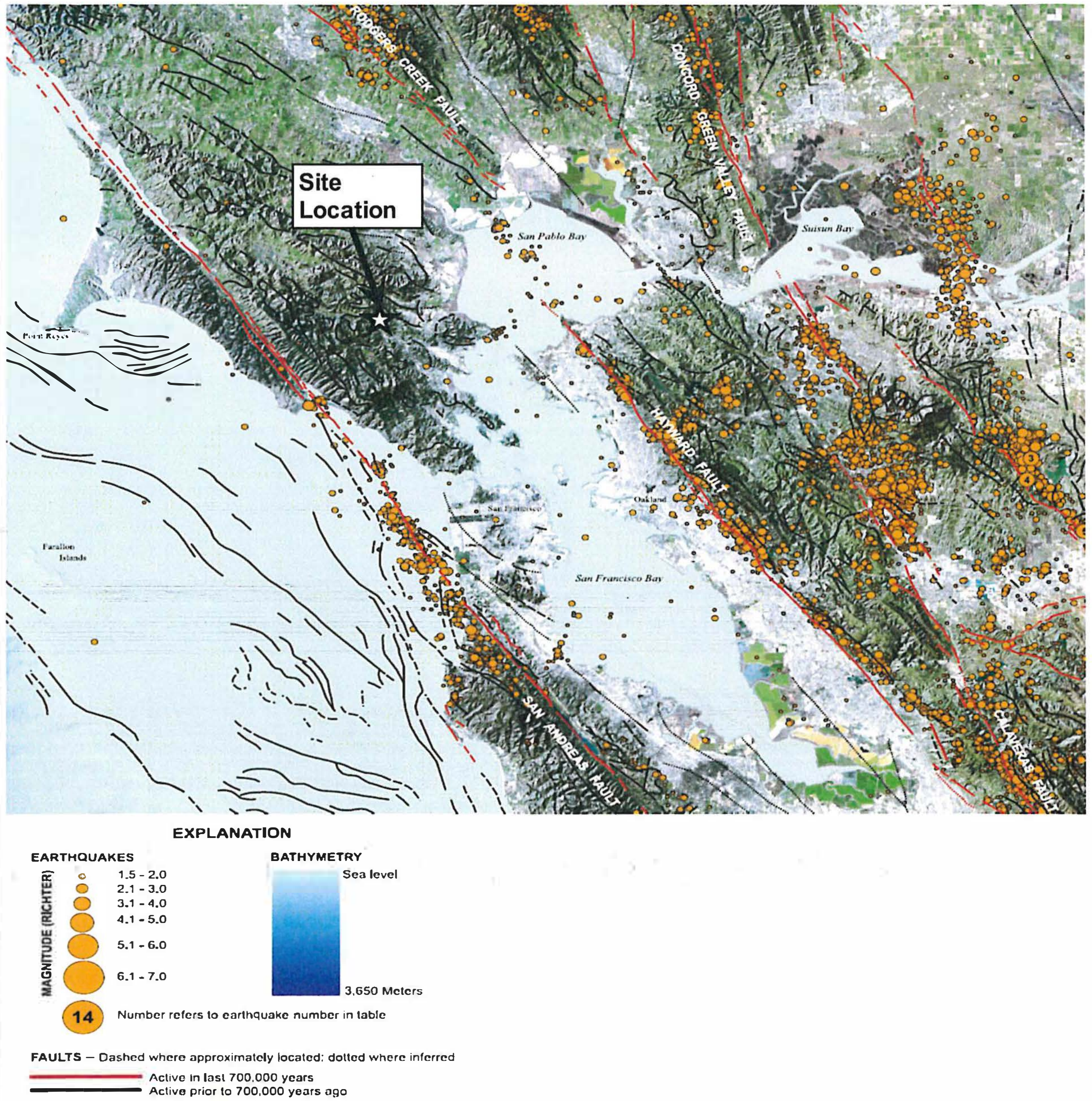
Figure 3



Source: Rice, S.J., Strand, R.G., and Smith, T.C., "Relative Stability of Slopes in the the Upper Ross Valley and the Western Part of the San Rafael Area Marin County, California" Plate 2B, 1976, Map Scale 1:12,000

DAC	**Relative Stability Map**	Report Date:	June 2022	**Figure 4**
	Altman Residence **63 Tamalpais Rd, Fairfax, CA** **APN: 001-123-03**	Reviewed By:	DA	
		Proj. Manager:	DA	
		Job No.:	1428-3321G	

Major Faults in the San Francisco Bay Area



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:	June 2022	Figure 5
	Altman Residence 63 Tamalpais Rd, Fairfax, CA APN: 001-123-03	Reviewed By:	DA	
		Proj. Manager:	DA	
		Job No.:	1428-3321G	

Appendix A

Material Symbols

▨	Topsoil	▨	Loose sand
▨	Fill	▨	Dense sand
▨	Asphalt	▨	Gravel
▤	High-plasticity clay (CH)	▨	Cobble
▤	Low-plasticity clay (CL)	▦	Rock or concrete
▥	Silt	▥	Colluvium / Alluvium / Residual Soil

Sample Types

⧄	Standard penetration test (SPT)	B	Bulk sample (bag)
☒	Modified California sample (MC)	S	Shelby tube sample (push)

Grain Sizes

U.S. standard series seive: 200 40 10 4

Clear square seive openings: 3/4" 3" 12"

Silt and clay	Sand			Gravel		Cobble	Boulder
	Fine	Medium	Coarse	Fine	Coarse		

Relative Density

Sand and gravel	Blows/foot*
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	> 50

Consistency

Silt and Clay	Blows/foot	Strength (tsf)**
Very Soft	0-2	0-¼
Soft	2-4	¼-½
Firm	4-8	½-1
Stiff	8-16	1-2
Very Stiff	16-32	2-4
Hard	> 32	> 4

* Number of blows from 140-pound hammer falling 30 in., driving 2-in. O.D. (1⅜-in. I.D.) split-spoon sampler 12 in. into soil.

** Unconfined compressive strength.

Terminology and abbreviations

W_n	Natural moisture content	LL	Liquid limit	C_u	Undrained shear strength
γ_d	Dry density	PL	Plastic limit	u_c	Unconf. comp. strength
O_c	Organic content	PI	Plasticity index	P_p	Pocket pen. reading (tsf)
				t_v	Torvane reading (tsf)

DAC

Key to Boring Logs

Altman Residence
63 Tamalpais Rd, Fairfax, CA
APN: 001-123-03

Report Date:	**June 2022**
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Job No.:	**1428-3321G**

Sheet A-0

Project 1428-3321G	63 Tamalpais Rd, Fairfax, CA			Driller: DeNovo
Date: July 16, 2021	Drill Rig: Minuteman	Portable	Hammer: 140 pound	Borehole diam.: 4 in.
Groundwater was not encountered	Boring Log BG-1			Logged by DL

Boring Log BG-1

	Sample type	Blow count	Test results	Graphic log	Material descriptions	Depth (ft)
				▩	**Topsoil**	
				▥	**Colluvium——Silty Sand:** brown; medium grain; loose; with some dark brown and reddish fine gravel fragments	
1	MC	8		▥		1
2		13 16		▥		2
3	MC	13		▦	**Weathered Bedrock——Sandstone:** brown; medium dense; breaks down into sandy silt	3
4		19 36		▦		4
5				▦	(high drilling resistance)	5
6	SPT	14		▦		6
7		14 14		▦		7
8				▦		8
9	SPT	21		▦		9
10		35 26		▦		10
11					Bottom of boring at 10 1/2 ft below the ground surface.	11
12						12
13						13
14						14
15						15
16						16

DAC	**Altman Residence** **63 Tamalpais Rd, Fairfax, CA** **APN: 001-123-03**	Report Date:	**June 2022**	**Sheet**
		Reviewed By:	**DA**	
		Proj. Manager:	**DA**	**A-1**
		Project No.:	**1428-3321G**	

Project 1428-3321G	63 Tamalpais Rd, Fairfax, CA		Driller: DeNovo
Date: July 16, 2021	Drill Rig: Minuteman	Portable	Hammer: 140 pound	Borehole diam.: 4 in.
Groundwater was not encountered			Logged by DL

Boring Log BG-2

Depth (ft)	Sample type	Blow count	Test results	Graphic log	Material descriptions	Depth (ft)
					Fill —— Silty Clay with Gravel: brown; coarse grain; with some dark brown and rusty-colored coarse gravel	
1	SPT	13				1
2						2
3	SPT	14 20 14			**Colluvium —— Silty Clay with Gravel:** brown; fine grain; with some brown coarse gravel	3
4						4
5						5
6	SPT	12 26 40			**Weathered Bedrock —— Shale:** brown; medium dense; breaks down into silty clay; with occassional white, sandy clay deposits	6
7						7
8						8
9	SPT	21 25 38				9
10					**Bedrock —— Shale:** dark gray; medium dense	10
11					Bottom of boring at 10 1/2 ft below the ground surface.	11
12						12
13						13
14						14
15						15
16						16

DAC

Altman Residence
63 Tamalpais Rd, Fairfax, CA
APN: 001-123-03

Report Date:	June 2022
Reviewed By:	DA
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Project No.:	1428-3321G

Sheet

A-2

DAC
Darius Abolhassani Consultant & Associates, Inc.
7 Mt. Lassen Dr, Suite A-129, San Rafael, CA 94903
(415) 499-1919 *Email: darius@dacassociates.net*

August 2, 2022

Mr. Stephen Altman
63 Tamalpais Road
Fairfax, CA 94930

Re: Temporary Excavation Shoring
Altman Residence
67 Tamalpais Road, Fairfax, CA
APN: 001-123-03
DAC Project No.: 1505-3622S

Dear Mr. Altman:

As requested, this letter provides preliminary description of temporary excavation shoring for development of the proposed new residence to be located at the above address, in Fairfax, California. Previously we had performed a geotechnical investigation for the proposed project and our geotechnical findings and recommendations were presented in a report titled '*Geotechnical Investigation, Altman Residence,63 Tamalpais Road, Fairfax, CA, APN: 001-123-03, DAC Project No.: 1428-3321G*', dated July 28, 2021. We had also submitted an updated report titled '*Revised Geotechnical Investigation, Altman Residence,63 Tamalpais Road, Fairfax, CA, APN: 001-123-03, DAC Project No.: 1428-3321G*", dated June 21, 2022.

A drawing titled *'Altman Residence, 67 Tamalpais Road, Fairfax, CA 94930'* by Kappe Architects, dated June 11, 2021, shows the location of the proposed project. Based on our review of the architectural plans, it is our understanding that the project will consists of construction of a new two-story building over a below grade garage at the street level. The development of the project involves cut slopes up to about 21 feet in height supported by retaining walls.

The general site parcel is irregularly shaped, located on an uphill sloped terrain with maximum plan dimensions of about 60 feet by 97 feet. Based on available topographic information, the site generally slopes up towards the south and southwest with an overall slope gradient of about 2.6:1 (horizontal: vertical). Steeper slopes are present to the south, with slope gradients as steep as 1.2:1.

Based on the results of our geotechnical investigation, the subsurface conditions at the site consist of a 1- to 3-ft layer of fill and slope debris over a 2- to 3-ft layer of colluvium overlying Franciscan sandstone and shale bedrock. The near surface soils are variable, consisting of coarse-grained slope debris to clayey colluvium. The overall CalOSHA classification of overburden materials would be Type B and C soils, which would need to have a safe unsupported cut slope of 4:1 (horizontal: vertical).

From our geotechnical borehole investigation, the average equivalent SPT blow counts in the top 5 to 7 feet depth of bedrock was found to be 40 to 60, which indicates it may be considered a 'stable rock' below depths of about 10 feet. Based on CalOSHA, 'stable rock' can be *"excavated* *with vertical sides and remain intact while exposed. It is usually identified by a rock name such as granite or sandstone…".*

However, our geotechnical investigation did not include the study of rock mass characteristics. In addition, due to the proximity of the proposed construction footprint to the property lines, there will not be sufficient space to allow for safe unsupported slope cut within the overburden materials. For this reason, a construction excavation shoring program has been recommended as follows.

The proposed temporary shoring would consist of a top-down excavation supported by a soil nail and shotcrete retaining wall system. Soil nails would be 1-inch diameter galvanized steel rods installed in 4-inch diameter drill holes at 6ft centers. The embedment of soils nails should be a minimum of 10 ft installed at a 15-degree inclination with horizontal. The actual embedment of soils nails will be determined in the design phase of the project. Shotcrete would be 6-inch thick reinforced with a single curtain biaxial grid of #4 bars at 12-inch on centers. Vertical and horizontal strip drains should be installed between the face of the slope and the shotcrete layer.

For areas where the required embedment of soil nails would exceed the available site space within the property lines (such as the west wall), a drilled concrete pier with steel soldier pile and wood lagging shoring system would be specified. In this case, the piers will be spaced at 6ft centers, drilled to appropriate depths followed by installation of steel soldier piles. Concrete would be poured to fill the pier up to the final excavation depth. Similarly, excavation would progress from the top and wood laggings installed as the excavation proceeds. The actual design of the soldier pile and lagging shoring system will also be performed in the upcoming design stage of the project. After the excavation is completed, the permanent retaining wall will be formed and built according to plans.

We trust the above description of the proposed excavation shoring fulfills the requirements of Town of Fairfax as presented in their plan review comments. If there are any questions or requirement for providing additional information, please do not hesitate to contact us.

Sincerely,

DAC Associates, Inc.

No. 58778

Darius Abolhassani, P.E., G.E.
Principal

HEI

ALTMAN RESIDENCE GRADING AND DRAINAGE PROJECT REPORT AT 67 TAMALPAIS ROAD, FAIRFAX, CA

July 27, 2022

Prepared for:

Kappe Architects

Prepared by:

Harrison Engineering Inc.
1987 Bonifacio St.
Concord, CA
(925) 691-0450

This report was prepared under the Direction of the following licensed persons:

Randell Harrison, PE
Harrison Engineering Inc.
July 27, 2022

REGISTERED PROFESSIONAL ENGINEER
RANDELL T. HARRISON
No. 46646
CIVIL
STATE OF CALIFORNIA

This drainage and design report has been prepared to evaluate the hydrology, hydraulics and peak flow mitigation at the site of the Altman Residence located at 67 Tamalpais Road in Fairfax.

I. Project Background

The current site at 67 Tamalpais Road in Fairfax is current undeveloped, other than two parking structures on the south side of the property, which are located in an easement for automobile parking purposes only. The parking structures and roadway on the south side of the property drain south, away from the site. There is only minor sheet flow entering the site from the adjacent property to the east.

The existing site slopes are approximately 2:1, sloping downward to the north.

II. Project Description

Drainage

HEI evaluated pre and post development storm water flow from the parcel at 67 Tamalpais Road, identified storm water conveyance system constraints, and recommended improvements. HEI also prepared the Grading and Drainage Plan for the site, which included the preliminary design of storm water detention vault to mitigate for the increase in impervious area on the site.

HEI evaluated the 100-year (one percent chance of occurence) storm event for the site to determine peak flows and corresponding volumes for runoff for both pre- and post-development site conditions.

Grading

Site grading is predominantly being achieved with retaining walls to create the building pad, rear patio, driveway, and on-site parking areas. Drainage is being incorporated to control concentrated flows coming down the slopes and keep storm water away the building foundation drainage system.

It is our understanding that a soil nail retaining wall will be used to stabilize the primary foundation excavation. Additional H-Pile and Timber Lagging retaining walls will be used elsewhere to stabilize slopes on the project site.

The site excavation is expected to utilize a large excavator and dump trucks for the majority of the earth moving for the project. Exact means and methods will be determined by the contractor that constructs the project.

Excess excavated material will be hauled and dumped at a commercial site, properly licensed and environmentally cleared to receive the site spoils. We anticipate the contractor will use the Marin Resource Recovery Center in San Rafael to dispose of soil and vegetation from the site.

III. Hydrology Analysis

A. Analysis Method

The Rational Method was used to calculate the 100-year peak flows and runoff volumes within the project site draining towards the northern section of Tamalpais Road adjacent to the Altman Residence. A storm duration of 30 minutes was assumed to develop hydrographs and corresponding runoff volumes for the site. Total watershed area is less than one square mile.

The "Modified Rational Hydrograph" method was used to create hydrographs for the purpose of calculating volumes of increased stormwater runoff to determine detention vault sizes.

B. *Drainage Areas*
A combination of topographic survey, Google Earth topographic data, and field observation were used to delineate the boundaries of the watershed areas. See attached watershed map exhibit (Appendix).

C. *Existing Drainage Features*
Survey data for the project site was collected by ILS Associates, Inc. The nearest point of collection for stormwater is a drainage inlet at the northwest corner of the property frontage.

D. *Time of Concentration*
The time of concentration for each sub-area of the watershed is less than 10 minutes. Therefore, a minimum time of concentration of 10 minutes was used in the rational method calculations.

E. *Runoff Coefficient*
Runoff coefficient of 0.525 was used for the majority of drainage areas on the project site, which consisted primarily of steep woodland with high potential for infiltration and negligible surface depressions. Runoff coefficient of 0.9 was used for the drainage areas of all roofs and the carports at the south end of the project site.

F. *Rainfall Data*
The rainfall data was obtained from the NOAA's Precipitation Frequency Data Server gage list. The data was used to determine a ten-minute 100-year storm intensity of 3.744 inches/hour.

Peak flow rates for each location within the project area were calculated using the rational method. Peak flow calculations for each drainage area are shown in Appendix A.

Runoff volumes for each location within the project area were calculated using modified rational method hydrographs. Modified rational method hydrographs relate the peak flow rate and storm duration to determine runoff volume for each drainage area. Runoff volume calculations for each drainage area are shown in Appendix A.

IV. Design Recommendations

Due to the approximately 300 gallons of increased runoff, HEI recommends the introduction of two 150 gallon (minimum) rainwater storage vaults, at the northwest corner and adjacent to the east side of the proposed building, to accommodate runoff increases due to the increase in impervious area (roof area only). 12" square drainage inlets will be used at pipe junctions and to capture surface drainage. All inlets and rainwater storage vaults shall be connected by 4" PVC SDR 26 pipe, ultimately directing runoff to either the existing inlet at the northwest corner of the proposed driveway or to the roadside gutter, which will maintain existing drainage patterns.

Each rainwater storage vault is anticipated to be a buried 250 gallon plastic vault with and orifice control outlet. The orifice outlet size is 0.25-inches. Due to the small orifice size, we highly recommend that the vaults have two sets of filter screens (one at the inlet to the vault and a second prior to the orifice. Also, the orifice should be a screwed on cap fitting accessible from the nearest downstream drainage inlet for periodic cleaning of the filters.

V. Abbreviations

cfs	Cubic feet per second
fps	Feet per second
LF	Linear feet
R/W	Right of Way

VI. Appendices

A. NOAA Atlas 14 Point Frequency Estimates for Fairfax, CA
B. Watershed Delineation Map with Hydrology Calculations
C. Modified Rational Method Hydrograph Volume Calculations
D. Stormwater Storage Vault Orifice Calculations

NOAA Atlas 14, Volume 6, Version 2 BON TEMPE DAM

Station ID: 84-0969
Location name: Fairfax, California, USA*
Latitude: 37.957°, Longitude: -122.61°
Elevation:
Elevation (station metadata): 723 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.149 (0.133-0.169)	0.183 (0.163-0.208)	0.231 (0.205-0.263)	0.272 (0.239-0.313)	0.332 (0.280-0.398)	0.382 (0.314-0.469)	0.435 (0.347-0.551)	0.494 (0.381-0.647)	0.579 (0.425-0.797)	0.650 (0.459-0.932)
10-min	0.214 (0.191-0.243)	0.263 (0.234-0.298)	0.331 (0.293-0.377)	0.390 (0.342-0.449)	0.476 (0.401-0.570)	0.547 (0.450-0.673)	0.624 (0.498-0.790)	0.708 (0.546-0.927)	0.830 (0.610-1.14)	0.932 (0.657-1.34)
15-min	0.259 (0.231-0.293)	0.318 (0.283-0.361)	0.400 (0.355-0.456)	0.471 (0.414-0.542)	0.576 (0.485-0.690)	0.662 (0.544-0.813)	0.755 (0.602-0.955)	0.856 (0.661-1.12)	1.00 (0.737-1.38)	1.13 (0.795-1.62)
30-min	0.439 (0.391-0.498)	0.538 (0.479-0.611)	0.678 (0.601-0.772)	0.799 (0.702-0.920)	0.976 (0.823-1.17)	1.12 (0.922-1.38)	1.28 (1.02-1.62)	1.45 (1.12-1.90)	1.70 (1.25-2.34)	1.91 (1.35-2.74)
60-min	0.655 (0.583-0.742)	0.803 (0.714-0.912)	1.01 (0.897-1.15)	1.19 (1.05-1.37)	1.46 (1.23-1.74)	1.67 (1.38-2.06)	1.91 (1.52-2.42)	2.17 (1.67-2.84)	2.54 (1.87-3.49)	2.85 (2.01-4.09)
2-hr	0.961 (0.856-1.09)	1.18 (1.05-1.34)	1.49 (1.32-1.70)	1.76 (1.54-2.02)	2.15 (1.81-2.57)	2.46 (2.02-3.03)	2.80 (2.24-3.55)	3.17 (2.45-4.15)	3.70 (2.72-5.10)	4.14 (2.92-5.94)
3-hr	1.25 (1.11-1.41)	1.53 (1.37-1.74)	1.94 (1.72-2.21)	2.28 (2.00-2.62)	2.78 (2.34-3.33)	3.18 (2.61-3.91)	3.61 (2.88-4.57)	4.07 (3.14-5.33)	4.74 (3.48-6.52)	5.29 (3.73-7.58)
6-hr	1.88 (1.67-2.13)	2.32 (2.07-2.64)	2.93 (2.60-3.34)	3.44 (3.02-3 96)	4.17 (3.51-5.00)	4.75 (3.90-5.84)	5.36 (4.27-6.78)	6.01 (4.63-7.86)	6.92 (5.08-9.52)	7.66 (5.40-11.0)
12-hr	2.67 (2.38-3.03)	3.35 (2.98-3.80)	4.25 (3.77-4.84)	5.00 (4.39-5.75)	6.03 (5.08-7.22)	6.83 (5.61-8.40)	7.66 (6.11-9.70)	8.53 (6.58-11.2)	9.72 (7.14-13.4)	10.7 (7.52-15.3)
24-hr	3.88 (3.49-4.39)	4.91 (4.42-5.57)	6.27 (5.62-7.13)	7.38 (6.57-8.46)	8.89 (7.68-10.5)	10.1 (8.53-12.1)	11.3 (9.32-13.9)	12.5 (10.1-15.8)	14.2 (11.0-18.6)	15.5 (11.6-21.0)
2-day	5.04 (4.54-5.71)	6.35 (5.71-7.21)	8.08 (7.25-9.19)	9.50 (8.46-10.9)	11.4 (9.87-13.5)	12.9 (10.9-15.6)	14.4 (12.0-17.8)	16.0 (12.9-20.2)	18.2 (14.1-23.8)	19.8 (14.9-26.9)
3-day	5.80 (5.22-6.57)	7.29 (6.56-8.27)	9.25 (8.30-10.5)	10.9 (9.67-12.4)	13.0 (11.3-15.4)	14.7 (12.5-17.7)	16.5 (13.6-20.3)	18.2 (14.7-23.1)	20.7 (16.1-27.1)	22.5 (17.0-30.6)
4-day	6.42 (5.78-7.28)	8.07 (7.26-9.16)	10.2 (9.18-11.6)	12.0 (10.7-13.7)	14.4 (12.4-17.0)	16.2 (13.7-19.5)	18.1 (15.0-22.2)	20.0 (16.1-25.2)	22.5 (17.5-29.6)	24.5 (18.5-33.3)
7-day	7.80 (7.03-8.85)	9.86 (8.87-11.2)	12.5 (11.2-14.2)	14.6 (13.0-16.7)	17.4 (15.0-20.5)	19.5 (16.5-23.4)	21.5 (17.8-26.5)	23.6 (19.1-29.9)	26.4 (20.5-34.7)	28.5 (21.5-38.7)
10-day	9.13 (8.22-10.3)	11.6 (10.4-13.1)	14.7 (13.2-16.7)	17.1 (15.2-19.6)	20.3 (17.5-24.0)	22.6 (19.2-27.2)	24.9 (20.7-30.7)	27.2 (22.0-34.4)	30.3 (23.5-39.7)	32.5 (24.5-44.1)
20-day	12.0 (10.8-13.7)	15.4 (13.9-17.5)	19.5 (17.5-22.2)	22.7 (20.2-26.0)	26.7 (23.1-31.6)	29.6 (25.1-35.7)	32.4 (26.9-40.0)	35.2 (28.4-44.5)	38.7 (30.0-50.8)	41.2 (31.0-55.9)
30-day	14.8 (13.3-16.7)	19.0 (17.0-21.5)	24.0 (21.5-27.3)	27.8 (24.8-31.9)	32.6 (28.2-38.5)	36.0 (30.5-43.4)	39.3 (32.5-48.4)	42.4 (34.2-53.6)	46.3 (36.0-60.8)	49.2 (37.0-66.6)
45-day	18.1 (16.3-20.5)	23.2 (20.9-26.4)	29.4 (26.3-33.4)	33.9 (30.2-38.9)	39.6 (34.2-46.8)	43.6 (36.9-52.5)	47.3 (39.2-58.3)	50.8 (41.1-64.3)	55.2 (42.9-72.5)	58.3 (43.9-79.1)
60-day	21.5 (19.3-24.3)	27.5 (24.8-31.2)	34.7 (31.1-39.4)	40.0 (35.6-45.8)	46.5 (40.1-54.9)	50.9 (43.2-61.4)	55.1 (45.7-67.9)	59.1 (47.7-74.6)	63.9 (49.7-83.9)	67.3 (50.6-91.2)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

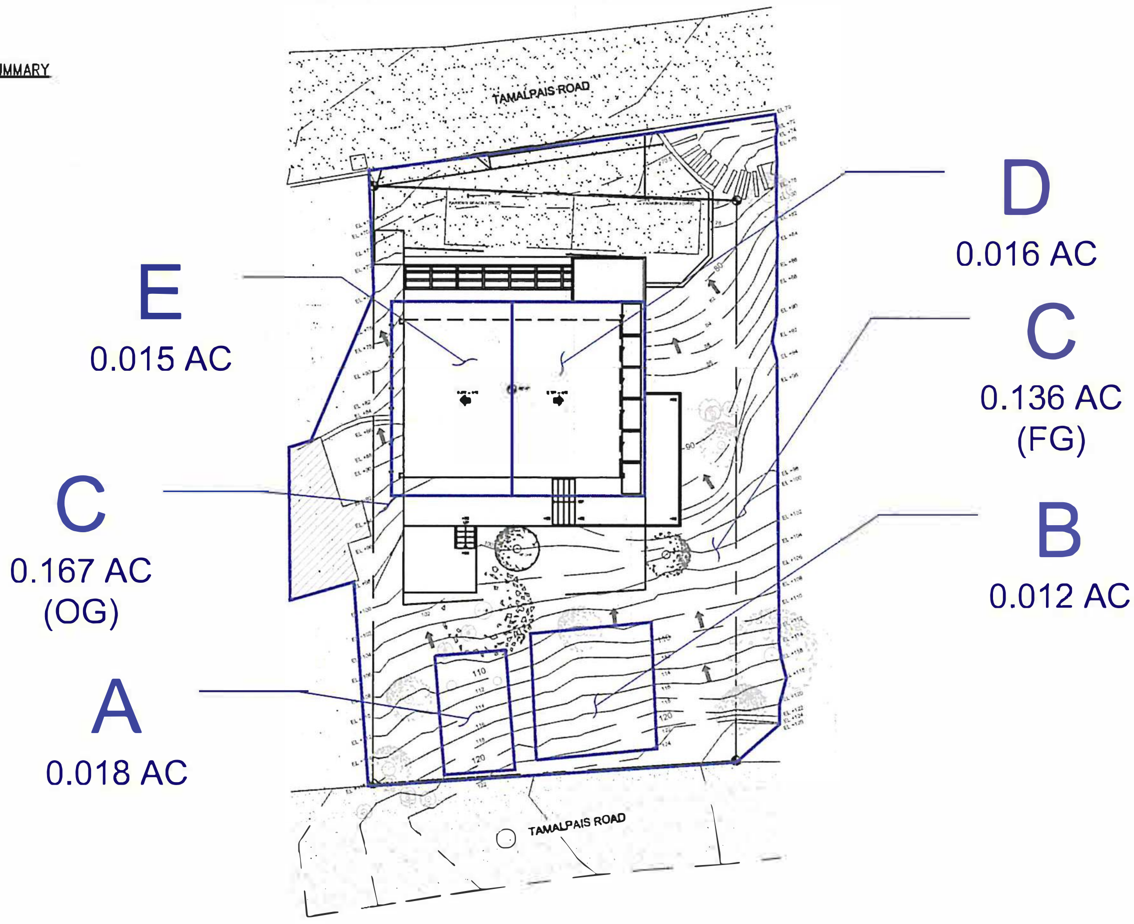
PF graphical

PRE-DEVELOPMENT (OG) HYDROLOGY SUMMARY

WATERSHED A
AREA = 0.018 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.9
Q = AiC = 0.018 x 3.744 x 0.9 = 0.06 (CFS)
V = 0.5 x 0.06 x 1800 x 7.48 = 404.95 (GAL)

WATERSHED B
AREA = 0.012 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.9
Q = AiC = 0.012 x 3.744 x 0.9 = 0.04 (CFS)
V = 0.5 x 0.04 x 1800 x 7.48 = 272.28 (GAL)

WATERSHED C
AREA = 0.167 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.525
Q = AiC = 0.167 x 3.744 x 0.525 = 0.33 (CFS)
V = 0.5 x 0.33 x 1800 x 7.48 = 2209.18 (GAL)



POST-DEVELOPMENT (FG) HYDROLOGY SUMMARY

WATERSHED A
AREA = 0.018 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.9
Q = AiC = 0.018 x 3.744 x 0.9 = 0.06 (CFS)
V = 0.5 x 0.06 x 1800 x 7.48 = 404.95 (GAL)

WATERSHED B
AREA = 0.012 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.9
Q = AiC = 0.012 x 3.744 x 0.9 = 0.04 (CFS)
V = 0.5 x 0.04 x 1800 x 7.48 = 272.28 (GAL)

WATERSHED C
AREA = 0.136 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.525
Q = AiC = 0.136 x 3.744 x 0.525 = 0.27 (CFS)
V = 0.5 x 0.27 x 1800 x 7.48 = 1800.88 (GAL)

WATERSHED D
AREA = 0.016 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.9
Q = AiC = 0.016 x 3.744 x 0.9 = 0.05 (CFS)
V = 0.5 x 0.05 x 1800 x 7.48 = 366.64 (GAL)

WATERSHED E
AREA = 0.015 (AC)
Tc = 10 (MIN)
i = 3.744 (IN/HR)
C = 0.9
Q = AiC = 0.015 x 3.744 x 0.9 = 0.05 (CFS)
V = 0.5 x 0.05 x 1800 x 7.48 = 333.31 (GAL)

VERIFY SCALES
0 1 2 3
3 INCHES ON ORIGINAL PLAN
IF NOT THREE INCHES ON THIS SHEET, ADJUST SCALES ACCORDINGLY

NUMBER	DATE	BY	REVISION

SCALE: 1"=20'
DATE: 07/20/2022
DESIGNED BY: RTH
DRAWN BY: ESM
CHECKED BY: RTH

HEI **HARRISON ENGINEERING INC.**
1987 BONIFACIO STREET * CONCORD, CA 94520
PHONE (925) 691-0450

H&H STUDY
ALTMAN RESIDENCE

WATERSHED DELINEATION MAP

SHEET
1 OF 1

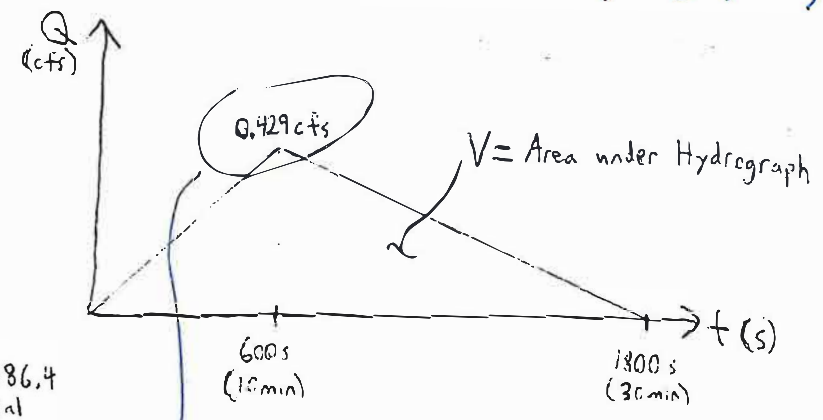
Altman Residence OG Runoff Volume

(Using Modified Rational Method Hydrographs)

$Q_{p,total} = 0.429$ cfs
(from rational method calculation)

$t = 30 \text{min} = 1800 \text{s}$
(assumed storm duration)

$V_{OG} = \frac{1}{2}(1800 \text{s})\underset{\text{cfs}}{(0.429)} = \underset{\text{ft3}}{386.1} = \underset{\text{gal}}{2886.4}$

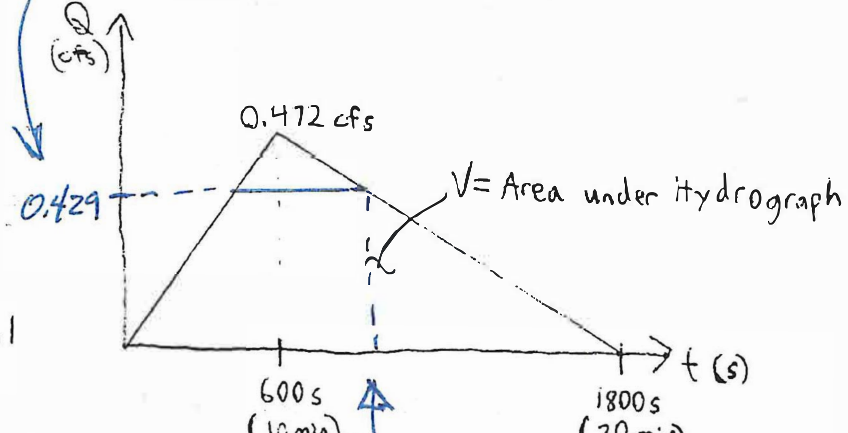


Altman Residence FG Runoff Volume

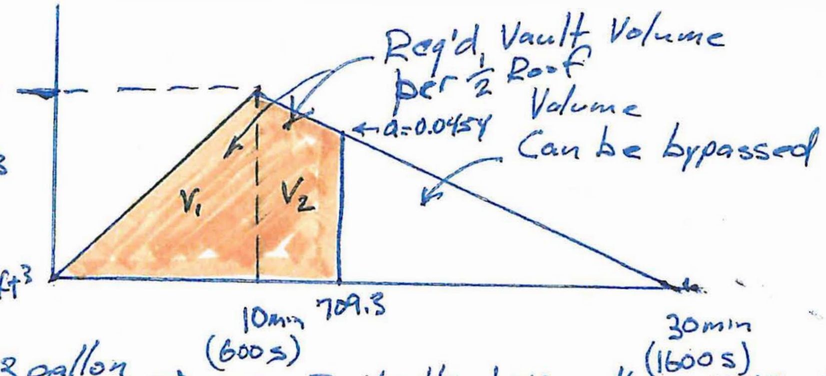
$Q_{p,total} = 0.472$ cfs
(from rational method calculation)

$t = 30 \text{min} = 1800 \text{s}$
(assumed storm duration)

$V_{FG} = \frac{1}{2}(1800 \text{s})\underset{\text{cfs}}{(0.472)} = \underset{\text{ft3}}{424.8} = \underset{\text{gal}}{3178.1}$



Capture Roof Runoff Until this time (709.3 seconds)



Roof Area D
Q=0.05cfs

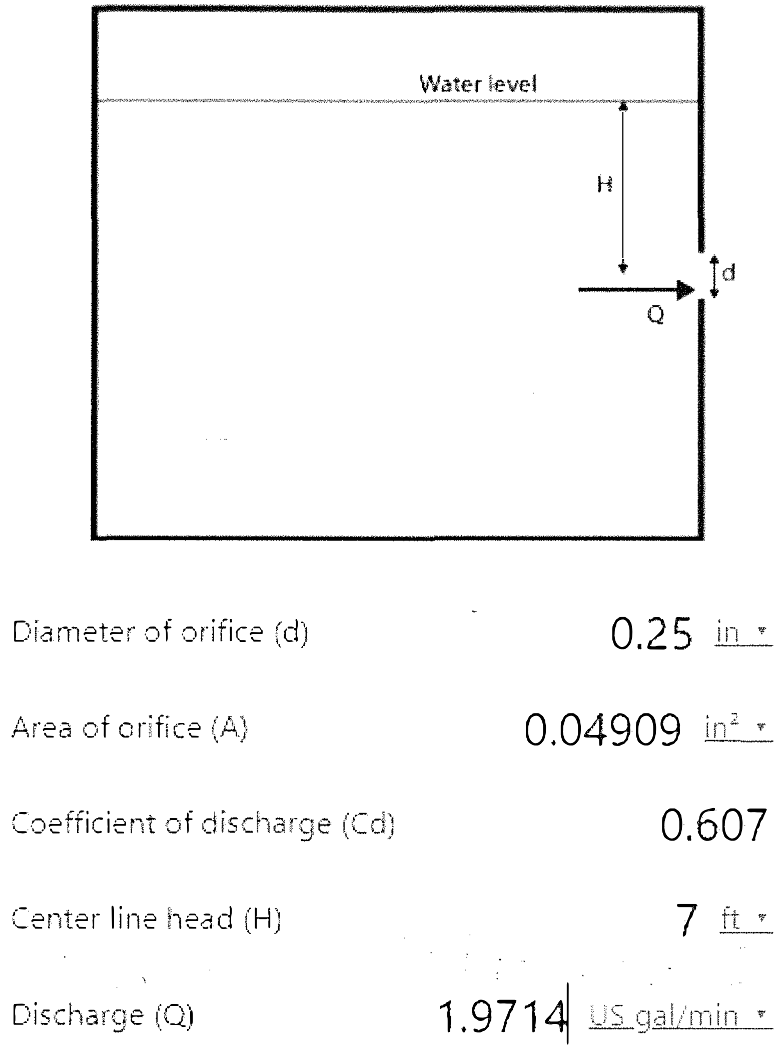
$V_1 = 600 \text{s} \times 0.05 \frac{\text{ft}^3}{\text{s}} \times \frac{1}{2} = 15 \text{ ft}^3$

$V_2 = 109.3 \text{s} \times \left(\frac{0.05 + 0.454}{2}\right) = 5.2 \text{ ft}^3$

$V_1 + V_2 = 20.2 \text{ ft}^3 \Rightarrow$ 151.3 gallon → use 2 Vaults Larger than 151.3gal.

Altman Residents -Vault Orifice Discharge Calculations

West side vault, minimum storage of 20.2 cubic feet (150 gallons).
Orifice size of 1/4-inch; Head =7.0 feet. Actual Storage 250 gallons.



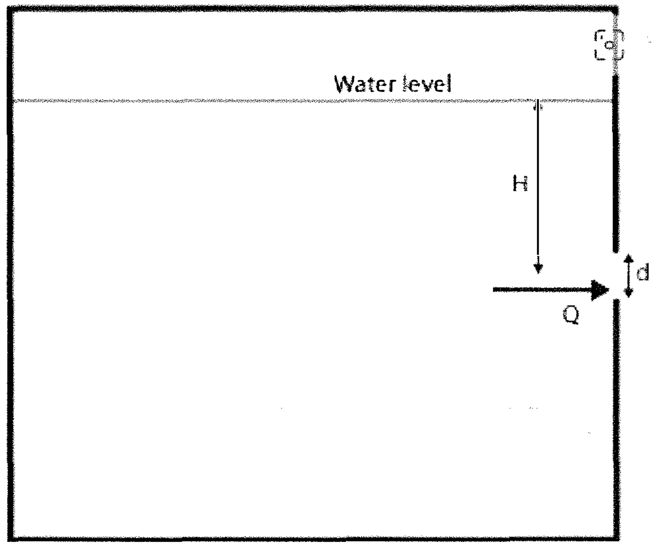
Diameter of orifice (d)	0.25	in
Area of orifice (A)	0.04909	in^2
Coefficient of discharge (Cd)	0.607	
Center line head (H)	7	ft
Discharge (Q)	1.9714	US gal/min

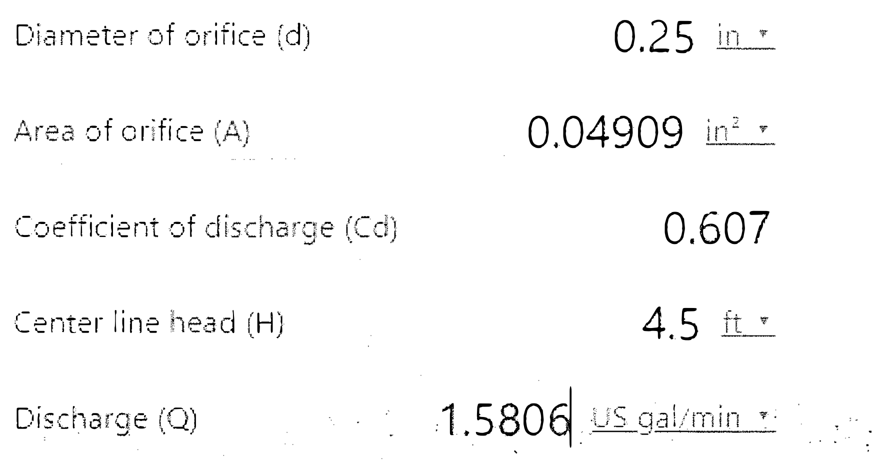
Equivalent discharge of 0.004392 CFS

150 gallons will drain out in 76 minutes.

Altman Residents -Vault Orifice Discharge Calculations

East side vault, minimum storage of 20.2 cubic feet (150 gallons). Orifice size of 1/4-inch; Head =4.5' feet. Actual Storage 250 gallons.



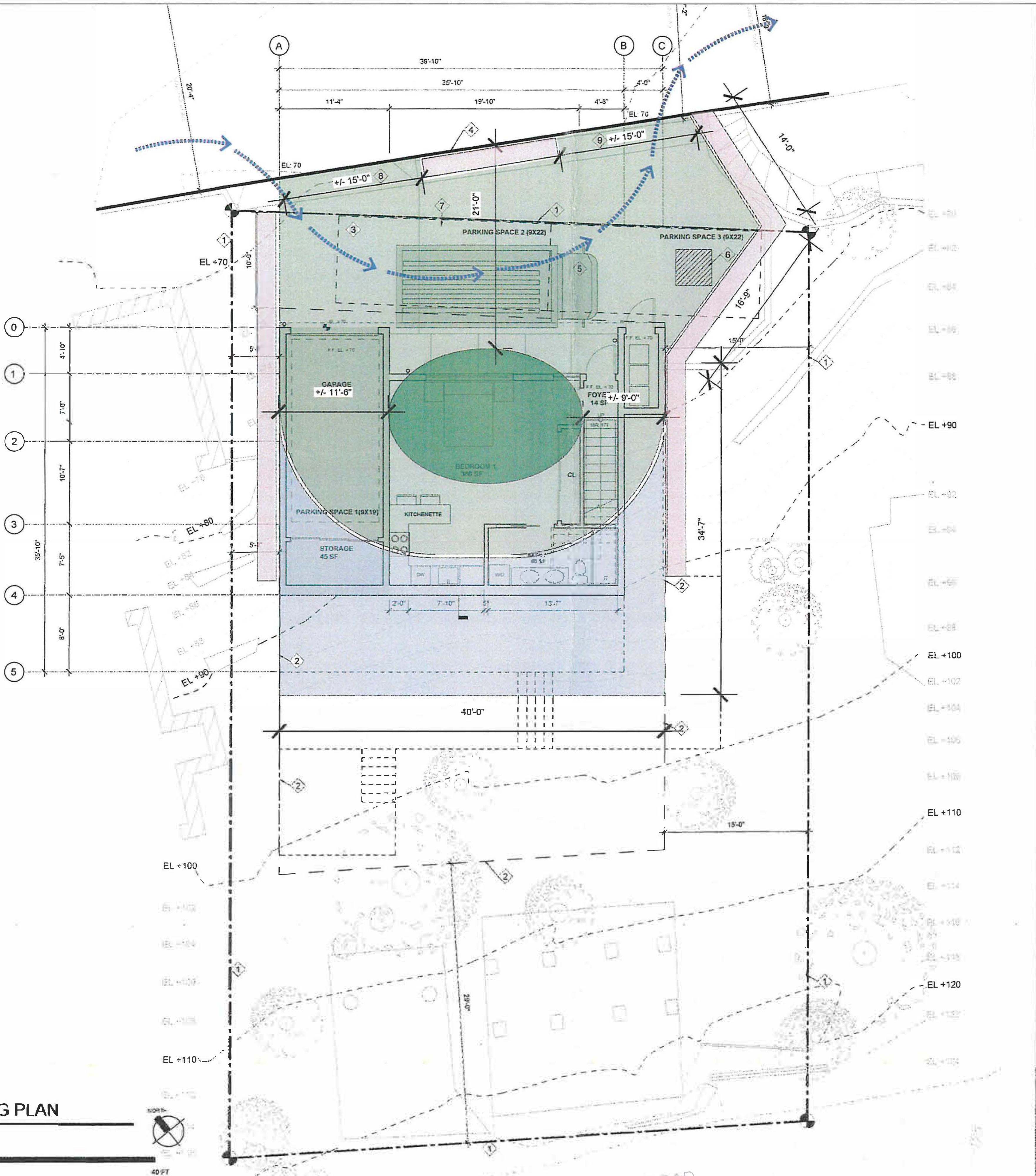


Diameter of orifice (d)	0.25	in
Area of orifice (A)	0.04909	in²
Coefficient of discharge (Cd)	0.607	
Center line head (H)	4.5	ft
Discharge (Q)	1.5806	US gal/min

Equivalent discharge of 0.0035216 CFS

150 gallons will drain out in 95 minutes.

Calculations using www.Omnicalculator.com



CONSTRUCTION STAGING PLAN

1

3/16" = 1'-0"

0 1 2 3 4 5 10 20 40 FT

KEY NOTES

1	PROPERTY LINE
2	SETBACK LINES
3	WORKING ELEVATION 74
4	EXISTING LINE OF CURB
5	DUMP TRUCK (8.5'W X 21'L), TYP.
6	PORTA POTTY LOCATION
7	GRAVEL AREA
8	+/- 15'-0" CLEARANCE FOR TRUCK, EQUIPMENT ENTRANCE
9	+/- 15'-0" CLEARANCE FOR EXIT
🟩	STAGING AREA FOR EARTH MOVER DIGGING FOR THE RETAINING WALL
🟩	STAGING AREA FOR DIRT TO BE OFF-HAULED
☐	AREA OF EARTH TO BE REMOVED FOR RETAINING WALL STRUCTURE
☐	WADDLES ALONG PROPERTY PERIMETER, TYP.
↗	TRUCK MOVEMENT ON AND OFF SITE

GENERAL NOTES

KAPPE ARCHITECTS

801 'D' STREET SAN RAFAEL, CA 94901
TEL: 415.457.7801 FAX: 415.457.7885

ALTMAN RESIDENCE

67 TAMALPAIS RD.
FAIRFAX, CA 94930

LICENSED ARCHITECT
RON H. KAPPE
C-18457
REN 6-30-23

AP #: 001-123-03

Project No.:	20.13	
Drawn By:	MH	
Checked By:	RK	
Issued Date:	8/31/2021	
Revision	No.	Date
PLANCHECK COMMENTS	1	3/28/2022
PLANCHECK COMMENTS	2	8/8/2022
PLANCHECK COMMENTS	3	10/11/2022

Sheet Title:

CONSTRUCTION STAGING PLAN

Scale AS NOTED

3

Sheet No. A2.0.4

KAPPE ARCHITECTS
801 D Street, San Rafael, CA 94901 T: 415.457.7801

Subject: ALTMAN- Task List for Resubmittal to Fairfax Planning 9-30-22

The letter states that they have paused the project until the geotechnical engineer has completed the subsurface and laboratory tests data, and has provided commentary on the project's exposure to risks associated with slope instability and surface runoff infiltration.

Response items required:

1. Architectural, Site

Architectural Scope- Apply for encroachment permit
Requiring a detailed construction management plan- this is usually done by the contractor but Kappe Architects will develop such a plan. They state that this is because structural drawings are not yet provided. They estimate that the off-haul will be more than 850 cubic yards. We did factor in for foundations already

2. Civil Scope

C2.0 need to show existing utility locations, and all new utility connections (water, sewer, electrical and gas) with pipe size labeled.

3. Geotechnical, Site

A. Need laboratory testing for the project- also states that not all comments from the first plan review were answered.

   Response: **We have determined that there would be no need for any laboratory testing to confirm our professional judgement about the soil conditions and our geotechnical recommendations.**

B. Geotechnical report page 3: Geotechnical engineer must comment on potential for slope instability to impact the proposed development and provide mitigation recommendations if warranted.

   Response: **As communicated with Scott Stevens of Miller Pacific during our recent phone conversation, the proposed project is expected to improve the site slope stability and mitigate any local slippages and zones of surficial soil creep. In addition, the excavation for construction of the proposed house would be protected by temporary shoring. The temporary shoring would be specifically designed to prevent slippage of the overburden soils and potential slope stability issues associated with construction excavations. Therefore, from a geotechnical engineering standpoint, we do not anticipate any negative impact on the stability of the subject site and/or the neighboring properties due to development of the project.**

C. Subsurface exploration must extend at minimum- to the proposed garage foundation elevation - not just 10'. Also, requirement to develop design criteria for temporary shoring, retaining wall and foundation elements.