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THOMPSON
WILDLAND MANAGEMENT
Environmental Management & Conservation Services
International Society of Arboriculture Certified Arborist # WE-7468A
Department of Pesticide Regulation Qualified Applicator Lic. #QL50949 B
Environmental & Arborist Assessments, Protection, Restoration, Monitoring & Reporting
Wildland Fire Property Protection, Fuel Reduction & Vegetation Management
Invasive Weed Control, and Habitat Restoration & Management
Soil Erosion & Sedimentation Control
Resource Ecologist

September 25, 2017

To: Mr. Chris Adamski
Emerson Development Group, Inc
24576 Portolo Avenue
Carmel, CA. 93923
APN: 009-463-012-000

Subject: Biological assessment for 26307 Isabella Avenue in Carmel

A biological assessment was recently conducted for the property located at 26307 Isabella Avenue in Carmel (APN: 009-463-012) in preparation for the proposed home developed project. This undeveloped, but previously impacted and disturbed lot is situated in a woodland residential community of Carmel. The property assessment involved performing a ground level visual inspection of the subject parcel to record and document biological resources, vegetation types and habitat characteristics, determine the presence or absence of biological resources that have protection status under federal and state laws (e.g., *California Environmental Quality Act* [CEQA] and *California Endangered Species Act* [CESA]), and provide resource protection and mitigation recommendations that may be necessary in preparation for the proposed property development project.

This biological evaluation was conducted by performing a thorough walk through and visual assessment of the subject property, and reviewing property development plans and maps (refer to the *Exhibit A. Site Plans* for property features and characteristics). Where possible the characteristics and conditions described in this report are depicted in the accompanying photographs located at the end of the report (refer to *Figures 1 & 2*).

I. SITE CHARACTERISTICS & BIOLOGICAL RESOURCES

The Monterey Peninsula supports a diversity of biological and cultural resources, including special status species, sensitive habitat and protected conservation values. The subject property at 26307 Isabella Avenue is less than .25 acres in size and is located in a urban woodland residential community of Carmel (refer to attached photos, *Figures 1 &*

2). This undeveloped and ruderal property has been previously impacted and disturbed. Homes in this community are located in relatively close proximity to one another and natural open space is generally absent in this particular area.

Based on a thorough assessment and evaluation of this previously disturbed and impacted property it is clearly evident that the subject parcel does not support protected special status species and/or sensitive habitat. There are no known occurrences of special status species, sensitive habitat or other protected resources on the subject property and none were observed during the field assessment.

This mixed woodland environment is significantly influenced by seasonally temperate coastal environmental conditions. Native tree species occurring in this coastal area of Carmel are dominated by mid to upper canopy Monterey Cypress (*Cupressus macrocarpa*), Monterey Pine (*Pinus radiata*) and mid to lower canopy Coast Live Oak (*Quercus agrifolia*). Mature and aging Monterey Cypress and Pine are the most visible and conspicuous upper canopy trees in the area with crown classes primarily ranging from co-dominant to dominant. On this particular disturbed and impacted lot, there are no upper canopy trees occurring on the property, however there is a relatively small and immature Coast Live Oak tree. This native oak as well as a introduced Cork Oak (*Quercus suber*) will be retained and protected. This property has little canopy cover; however, it should be noted that larger and more conspicuous cypress, pines and oaks are occurring on nearby adjacent properties.

Trees located on this lot primarily consist of lower growing non-native and introduced ornamental species that appear to have been planted on the property several years ago. Introduced ornamental tree species include Pittosporum (*Pittosporum undulatum*), Bradford Pear (*Pyrus calleryana*) and Cork Oak. Lower growing shrubs and vegetation occurring on this ruderal property include exotic and introduced Hawthorn Bush (*Crataegus* sp.), English Ivy (*Hedera helix*), Panic Veldt Grass (*Ehrharta erecta*) and a few species of exotic annual grasses (e.g., Ripgut Brome [*Bromus diandrus*]). The only native plant species observed on this previously disturbed and impacted property include a few immature Coast Live Oak trees, a Toyon (*Heteromeles arbutifolia*) bush, and a few small patches of Bracken Fern (*Pteridium aquilinum*).

Special status flora and fauna, sensitive habitat, and actively nesting birds that have protection status were not observed and are not known to occur on the subject property. Vegetation density, cover and diversity is lacking in most areas of the property due to the site being previously graded and impacted. Additionally, natural recruitment and regeneration of indigenous tree species is deficient on the subject property.

Prior to site disturbance, this undeveloped lot was primarily composed of introduced ornamental vegetation, forbs and exotic weeds according to communications with the property owner and a neighbor. Per the assessment, it is highly unlikely that this ruderal property has supported any ecologically significant or valuable habitat in recent years.

Soils on this relatively flat parcel appear to be stable and sufficient for supporting healthy flora and property development activities. Wind direction is predominantly out of the southwest. As previously noted, special status animal species, sensitive habitat and nesting birds that have protection status were not observed during the property evaluation. However, a nesting bird assessment should be conducted if construction activities begin during the nesting season, which in Monterey County may begin as early as February and continue through August. Additionally, per the project plans, no development or soil disturbance is occurring on steep slopes with high erosion potential (e.g., slopes with 25% or steeper grade). Consequently, erosion and sedimentation concerns should be minimal.

II. RECOMMENDATIONS

In the interest of protecting and minimizing impacts to biological resources the following resource protection measures and best management practices (BMP's) should be implemented:

1) Prior to construction activities beginning, install resource protection measures to clearly identify and delineate the construction zone and to prevent unnecessary construction site expansion and disturbance to surrounding areas. Resource protection BMP's include appropriate erosion and sedimentation control measures, tree protection measures, and high visibility exclusionary fencing that clearly identifies the construction zone and building envelope. Resource protection measures should be properly maintained for the duration of the project.
2) More specifically, install protective exclusionary fencing along the outer perimeter of the construction site or property line and around trees that will be retained and protected. This high visibility exclusionary fencing will assist in protecting resources from construction related impacts and encroachment.
3) In the landscape plan consideration should be given to utilizing plants that are native to mixed woodland habitat. Plants selected for landscaping operations should be drought tolerant, relatively fire resistant, non-invasive to wildland areas, and well adapted to this particular environment.
4) As previously stated, nesting birds were not observed during the site assessment, however the nesting season in Monterey County may begin as early as February and continue through August. Consequently, if construction activities begin during this nesting period an additional nesting assessment should be conducted within two weeks of construction activities commencing.

III. CONCLUSION

In conclusion, biological resources that are protected under federal and state laws (e.g., CESA and CEQA) were not observed during the assessment of the property located at 26307 Isabella Avenue. Consequently, protected special status species and sensitive habitat will not be impacted by proposed property development activities. Implementation of resource protection measures provided in this report will aid in sustaining existing resources on the property as well as protecting off-site resources, and will assist in satisfying *Monterey County Resource Management Agency* permit conditions.

Thank you and please let me know if you have any questions or need additional information.

Best regards,

________________________________ __________________

Rob Thompson Date
Resource Ecologist
ISA Certified Arborist

Thompson Wildland Management (TWM)
57 Via Del Rey
Monterey, CA. 93940
Office (831) 372-3796; Cell (831) 277-1419
Email: thompsonwrm@gmail.com ; Website: www.wildlandmanagement.com

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Figure 1. Sensitive and protected biological resources are not occurring on previously disturbed and impacted property located at 26307 Isabella Ave in Carmel. Resources are limited to a few native tree and plant species, as well as some non-native introduced plant species.

Figure 2. Mature mid to upper canopy roadside Monterey Cypress trees are bordering property.

HARO, KASUNICH AND ASSOCIATES, INC.

CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

Project No. M11382
18 December 2017

LIB180355

Chris Adamski
Emerson Development Group, Inc.
3345 7th Avenue
Carmel, California 93923

Subject: Geotechnical Investigation

Reference: New Residence
26307 Isabella Avenue
Carmel, Monterey County, California
APN 009-463-012

Dear Mr. Adamski,

This letter summarizes our geotechnical investigation for the proposed new residence at the referenced site. This letter includes our findings and geotechnical recommendations. Our work on this site was performed in conjunction with two other new residences proposed on adjacent lots, APN 009-463-017 & APN 009-463-09, also owned by you. Haro, Kasunich & Associates (HKA) worked in synch with project Engineering Geologist, Craig Harwood. His fault study report (dated November 22, 2017; received November 27, 2017) was prepared independently by his firm. Specifically, HKA was on-site during his subsurface drilling operation and co-logged the bore holes on each respective site. This geotechnical report is specific to the referenced lot only.

Introduction

The site is located on Isabella Avenue, 2 lots southeast of 16th Avenue in Carmel, California. Refer to Figure Number 1 of Appendix A for a site vicinity map. The undeveloped lot slopes gently down from the street with approximately 8 feet of fall across the site. Surface flow trends downward through the proposed building envelope to Valley View Avenue. Vegetation consists of mostly bare ground, few small trees and some ground cover.

Based on review of preliminary architectural sheets prepared by project Architect, Tom Meaney, revision dated August 30, 2017 (received date November 18, 2017) we understand a new two-story single family residence with attached garage is proposed. A basement is proposed requiring about up to 14-foot deep excavations. Retaining walls, patio terraces, grading and drainage improvements are is also proposed.

Purpose and Scope

The purpose of our work was to explore surface and subsurface soil conditions in the vicinity of the proposed residence and to develop geotechnical recommendations.

The specific scope of our services included the following:

A. Site visit, file and document review and project administration.

B. Co-logging of soils encountered during the subsurface field exploration facilitated by the project geologist. One machine drilled boring was advanced on this site (APN 009-463-012) and two other borings were drilled on adjacent sites (APN 009-463-017 & 003) using truck mounted equipment.

C. Engineering analysis and evaluation of the resulting field data. Based on our findings and review of the geology report, we developed geotechnical recommendations for foundations, retaining walls, slabs-on-grade, subgrade preparation beneath flatwork, site drainage, and include California Building Code seismic criteria.

D. Preparation of this report presenting the results of the investigation and generated geotechnical recommendations.

Field Exploration

Subsurface conditions were investigated on 9 October 2017. The boreholes were co-logged with project Engineering Geologist, Craig Harwood. The approximate locations of the test bore holes are indicated on the boring site maps, Figures 2 & 3, in Appendix A. The test borings were advanced using 6-inch diameter continuous flight-auger equipment mounted on a truck.

Representative soil samples were obtained from the exploratory borings at selected depths, or at major strata changes. These samples were recovered using the 3.0 inch O.D. Modified California Sampler (L) or the Standard Terzaghi Sampler (T).

The penetration resistance blow counts noted on the boring logs were obtained as the sampler was dynamically driven into the in situ soil. The process was performed by dropping a 140-pound hammer a 30-inch free fall distance and driving the sampler 6 to 18 inches and recording the number of blows for each 6-inch penetration interval. The blows recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches.

The soils encountered in the borings were continuously logged in the field and described in accordance with the Unified Soil Classification System (ASTM D2486). The Logs of the Borings are included in the Appendix of this report. The Boring Logs denote subsurface conditions at the locations and time observed, and it is not warranted that they are representative of subsurface conditions at other locations or times.

Subsurface Soil

The site is underlain by approximately 5 to 9 feet of loose surficial dark brown silty sand soil. This surficial soil overlays a medium dense poorly graded sand, Coastal Terrace Deposit. In Boring 2, located in the southwest corner of the site, weathered Basaltic bedrock was encountered at a depth of 27.75 feet to depth explored, 31 feet. The adjacent sites also encountered Basaltic bedrock at depths of 35.5 feet and 33.75 feet.

Perched Groundwater was encountered in the Boring B-3 drilled on this parcel at 29 feet below grade. Water levels can fluctuate.

Concurrent Geologic Study

Project Engineering Geologist, Craig Harwood, conducted a Geologic investigation for the site and prepared a geologic report dated 22 November 2017 (received 11/27/17). The Purpose of his geologic investigation was to evaluate and define the geologic conditions and identify potential geologic hazards associated with the project site. Specifically, determining the trend of the nearby Cypress Point Fault and determine its proximity to the project site. Based on the results of his geologic investigation, we understand the Cypress Point Fault does not cross the property but is located about 35 feet away to the southwest of the subject parcel as shown in the Geologic Evaluation report dated 22 November 2017. He suggests the proposed construction is sufficiently setback from the fault line. For a more in depth discussion of the site geology and associated geologic hazards refer to the aforementioned Geologic Evaluation report.

CBC Considerations

It is anticipated significant seismic shaking will occur at the site during the lifetime of the project.

Our field work indicates that the predominant Site Class is "D" as defined in the current CBC. The seismic site factors, as defined in Chapter 16 of the current CBC section 1613.5.3, considered applicable for the project's latitude and longitude can be generated from the printed graphs in the current CBC or the interactive USGS Design Maps web site tool.

Due to the lack of ground water at the potential for seismically induced liquefaction is low.

Settlement

We anticipate that approximately 1 inch of total settlement and 1 inch of differential settlement may be experienced by the new structure.

Conclusions

- The site is conducive to the proposed development if the recommendations contained in this report are followed.

- The upper 5 to 9 feet of dark brown loose silty sand soil is not adequate for shallow foundation or slab support in their present condition. It is anticipated the bottom of excavation for the basement of the main part of the house (about 12 to 14 feet deep) will encountered firm native soils that are adequate for conventional footing support. However, the proposed landscape flatwork, terraces, garage, entry, site walls, master wing excavation (0 to 2 feet) and basement den excavation (3 to 5 feet deep) will likely encounter loose soil at the exposed grade. This differing condition can lend itself to excessive diffential settlement. The potential for differential settlement can be decreased.

- To mitigate the potential for differential settlement of the areas mentioned above, we offer two options.

  Option A) Sub-excavate the top 4* feet of loose soil in the shallowly excavated or shallowly supported areas and in exterior flatwork areas, scarify the bottom of the excavations 12 inches and recompact to 92%. These excavations should then be brought back up to finished graded with re-compacted engineered fill (refer to fill requirements in the Site Grading section of this report). Shallow footings may be then be used.
  * Actual depth to be determined at the time of construction by the geotechcail engineer.

  OR

  Option B) Helical piers that penetrate through the 5 to 9 foot thick upper zone of loose soil and embed in the firm sand at depth, may be used to support the new shallowly supported improvements, master wing and den basement and hardscape improvements.

- A deepened footing option is not recommended due to the potential for cave-in of sidewalls of the deep trench excavations in loose sand, but can be determined in the field at the time of construction.

- In either case, the bottom of the basement excavation of the main house should be scarified 12 inches and compacted to 92% relative compaction, as determined by the geotechnical engineer in the field at the time of construction.

- Drainage improvements should include positive gradients away from structures on all sides; roof and surface runoff control; and discharge away from the home.

- We should be afforded a chance to check all rough excavations prior to installing form boards or steal rebar to confirm anticipated soil conditions.

- We should be afforded a chance check all rough excavations prior to installing form boards or steal rebar to confirm anticipated soil conditions.

<u>Site Grading</u>

1. The geotechnical engineer should be notified <u>at least four (4)</u> <u>working days</u> prior to any site clearing or grading so that the work in the field can be coordinated with the grading contractor and arrangements for testing and observation can be made. The recommendations of this report are based on the assumption that HKA will perform the required testing and observation during grading and construction. It is the owner's responsibility to make the necessary arrangements for these required services.

2. Where referenced in this report, Percent Relative Compaction and Optimum Moisture Content shall be based on ASTM Test Designation D1557-curent.

3. Areas to be graded should be cleared of all obstructions including loose fill, building/concrete debris, trees not designated to remain, or other unsuitable material. Existing depressions or voids created during site clearing should be backfilled with engineered fill.

4. Cleared areas should then be stripped of organic-laden topsoil. Actual depth of stripping should be determined in the field by the HKA. Stripping's should be wasted off-site or stockpiled for use in landscaped areas if desired.

5. Stripped areas should be sub-excavated to the prescribed depth as discussed above in Option A, as determined in the field.

6. Temporary slopes may be cut back at a gradient of 2:1 (horizontal:vertical) in the loose sand. Otherwise temporary shoring will be necessary. The contractor should be aware of local and federal excavation safety laws in regards to cut slope heights. Top-down constructed temporary shoring will be necessary where laying back slope is not feasible.

7. The exposed base of subexcavated areas and other areas to receive engineered fill (hardscape and slab areas at a minimum) should be scarified to a depth of 12 inches, moisture conditioned, and compacted to 92 percent relative compaction.

8. Engineered fill should be placed in thin lifts not exceeding 8 inches in loose thickness; moisture conditioned, and compacted to at least 92 percent relative compaction. The upper 6 inches of pavement section subgrades should be compacted to at least 95 percent relative compaction. The aggregate base below pavements should likewise be compacted to at least 95 percent relative compaction.

9. If grading is performed during or shortly after the rainy season, the grading contractor may encounter compaction difficulty, such as pumping or bringing free water to the surface, in the upper surface silty sands. If compaction cannot be achieved after adjusting the soil moisture content, it may be necessary to over-excavate the subgrade soil and replace it with angular crushed rock to stabilize the subgrade. We estimate that the depth of over-excavation would be approximately 24 inches under these adverse conditions.

10. If properly moisture conditioned, except for organic-rich soil, the on-site soils generally appear suitable for use as engineered fill. Import soils utilized as engineered fill at the project site should:
    a. Be free of wood, organic debris and other deleterious materials;

b. Not contain rocks or clods greater than 2.5 inches in any dimension;
c. Not contain more than 25 percent of fines passing the #200 sieve;
d. Have a Plasticity Index less than 18;
e. Be approved by HKA. Contractor should submit to the geotechnical engineer samples of import material or utility trench backfill for compliance testing a minimum of 4 days before it is delivered

11. After the earthwork operations have been completed and the geotechnical engineer has finished his observation of the work, no further earthwork operations shall be performed except with the approval of and under the observation of the geotechnical engineer.

Temporary Shoring

12. The basement excavation may be 10 to 15 feet deep. Deep excavations potentially can create an instability problem and should be shored. The contractor or the specialty subcontractor is to be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. A registered civil or structural engineer in the State of California should design and stamp plans for shoring.

Spread Footing Foundation System

13. Where firm native soil is anticipated at footing grade, as in the main deep living room basement area, conventional shallow spread foundations may be used. The base of the footing excavations may need to be tamped with a jumping jack to increase soil density and reduce potential differential settlement. To be confirmed on site during earthwork by the geotechnical engineer. Footings excavations should at least 18 inches deep below lowest adjacent frim native grade and at least 18 inches wide. Actual footing dimensions should be determined in accordance with anticipated use and applicable design standards. Footings should be reinforced as required by the structural designer based on the actual loads transmitted to the foundation.

14. Where loose soils are anticipated at footing grade, as in the garage, entry, den, master and exterior landscape areas, (actual condition of exposed soil must be verified by the geotechcail engineer) conventional shallow footings may be embedded at least 18 inches into a mat of certified engineered fill as described in **Option A** above. The mat of engineered fill should extend a minimum 5 horizontal feet beyond the outer edge of the foundation and slab elements in each direction. The mat of engineered fill should be prepared in accordance with the section of this report titled “Site Grading”.

15. Temporary shoring will likely be needed to prevent caving of the sides walls if deep footing trench excavations are utilized.

16. Foundation drains and waterproofing should be used around the exterior perimeter of master wing.

17. Foundations designed in accordance with the above may be designed for an allowable soil bearing pressure of 1,800 psf for dead plus live loads. This value may be increased by one-third to include short-term seismic and wind loads.

18. Lateral loads on spread footings may be designed for a passive resistance acting along the face of the footings. Where footings are poured neat against engineered fill consisting of native soils, an equivalent fluid pressure (EFP) of 250 pcf acting along the face of the footings is considered applicable. Lateral load resistance for structures supported on spread footings may be developed in friction between the foundation bottom and the supporting subgrade. A coefficient of friction value of 0.30 is appropriate.

19. Footings located adjacent to other footings or utility trenches should have their bearing surfaces founded below an imaginary 2:1 plane projected upward from the bottom edge of the adjacent footings or utility trenches.

20. The foundation trenches must be kept moist and be thoroughly cleaned of all slough or loose material prior to pouring concrete.

21. All footing excavations should be thoroughly cleaned and observed by HKA prior to placing forms and steel. Observation of foundation excavations allows anticipated soil conditions to be correlated to those inferred from our investigation and to verify that the footings are in accordance with our recommendations.

Helix Pier Option (Not including main Living Room Basement Level)

22. As an alternative to shallow footings embedded in the certified engineered fill mat, helical piers may be used to support the portion of the house beyond the deep main living rom basement (i.e. garage, entry, den, master any exterior hardscape). It is anticipated the deep living room basement excavation will encounter adequate soil capable of supporting convention footings (to be confirmed at the time of construction).

23. The structural engineer specifies the load demand for each pier. Structural layout, design, hardware specifications and details of the helical pier foundation system should follow manufacture's recommendations as outlined by A.B. Chance, or equivalent supplier.

24. All helix anchors should be protected with galvanized coating.

25. Piers should be spaced at least 5 diameters apart, or at least 5 feet, whichever is greater. The diameter of the largest helix plate is used to determine the spacing.

26. Piers must extend **at least** 7 feet into firm material, and at least 7 feet below proposed main house deep living room basement elevation 43 feet, as determined

by the soil engineer during construction. The piers must also be as deep as necessary to achieve the specified minimum load. Whichever is deeper.

27. The need for battered piers is up to the structural engineer’s need to accommodate lateral forces.

28. The specialty helical pier installer contractor typically selects the appropriate helical configuration. Pull out and compression proof pre-testing of helical anchors should be conducted several weeks prior to the start of the job in order for the contractor to select appropriate helical plate configurations that can accommodate the specified load in a reasonable length. This test also sets a site specific calibration so that the pressure gage “installation torque vs capacity” method can be used during installation to check compliance to specified load. Testing to be confirmed by the soil engineer during construction.

29. **It is recommended that <u>at least</u> one vertical test anchor be installed prior to full scale production in order to verify both design loads and installation torque requirements. This testing should be performed under the observation of the Geotechnical Consultant.**

30. Rotational resistance encountered by an anchor when being screwed into the soil is defined as installation torque. Monitoring of installation torque during installation is required. Installation torque should not exceed the anchor rating.

31. Production piers must both a) be installed deep enough to achieve at least 2 times the specified load and b) reach the minimum embedment depth.

32. To determine the compliance of installed helical piles to 2 times the specified load, installation pressure gage readings, taken during installation, are converted to capacity using the “pressure vs. torque” charts specific to each drive head, supplied by the contractor. Torque values are then converted to load using an estimated conversion factor of 10 for small square shaft dimensions subject to verification in the field. Readings to be confirmed by the soil engineer during construction.

33. All anchor installation must be observed and approved by the Geotechnical Consultant. Any anchors installed without the full knowledge and observation of the Geotechnical Consultant will render the recommendations of this report invalid.

34. The installation contractor should be required to show proof of certification to install the specified manufacturers’ helical pile or tieback material of such is required by the manufacturer or the specification. It is recommended that beyond initial certification, the Installation Contractor re-certifications are consistent with manufacturers’ recommendations.

35. All gage and pump equipment must have current calibration with the previous 6 months of construction.

36. Alternatively, if pier holes can be kept from caving in, skin-friction reinforced concrete-cast-in-place piers may be used. The structural engineer determines depth based on a value of 375 psf of skin friction (this value may be increased by 1/3 to account for seismic and short term loading) for piers with a diameter of at least 18” for that portion of the pier embedded in approved firm native soil. Neglect loose upper soil, topsoil in calculating total capacity of pier. Spacing should not be closer than 3 pier diameters. At a minimum, piers must extend to similar depths as described above for helical piers.

Retaining Wall Lateral Pressures

37. Foundations for retaining/bearing walls should follow the criteria in the previous sections of this report.

38. To account for seismic loading, a horizontal line load surcharge equal to $10H^2$ pounds per linear foot of wall may be assumed to act at 0.6H above the base of the wall (where H is the height of each terraced wall).

39. Retaining walls should be designed to resist both lateral earth pressures and any additional surcharge loads. For design of retaining walls up to 15 feet high and fully drained, the following design criteria may be used:

    i. Active earth pressure for walls **allowed to yield** is that exerted by an equivalent fluid weighing 40 pcf for a level backslope gradient and 55 pcf for a 2:1 (horizontal to vertical) backslope gradient. This assumes a fully drained condition.

    ii. Where walls are restrained from moving at the top, design for a uniform rectangular distribution equivalent to 28H psf per foot for a level backslope, and 39H psf per foot for a 2:1 backslope, where H is the height of the wall. Alternatively, restrained walls may be designed for an ‘at rest’ lateral earth pressure equivalent to a fluid weighing 60 pcf for level backfills.

    iii. In addition, the walls should be designed for any adjacent live or dead loads that exert a force on the wall.

    iv. The above lateral pressure values assume that the walls are fully drained to prevent hydrostatic pressure behind the walls. Drainage materials behind the wall should consist of Class 1, Type A permeable material complying with Section 68 of Caltrans Standard Specifications, latest edition or approved equivalent.

    v. The drainage material should be at least 12 inches thick and extend from the base of the wall to within 12 inches of the top of the backfill.

40. **Wall backdrains should be capped at the surface** with clayey material to prevent infiltration of surface runoff into the backdrains. A layer of filter fabric (Mirafi 140N or equivalent) should separate the subdrain material from the overlying soil cap.

41. Retaining walls should be thoroughly waterproofed their full height, especially at the cold joint at the base of the wall if living space.

42. The base of the drain column should be made to be an impermeable channel. The heel of the foundation should be waterproofed to allow water to build up and enter drainpipe. A perforated rigid drain pipe should be placed (holes down) on the heel of the footing and be tied to a suitable solid rigid drain outlet/sump. The cold joint at the heel should be plugged with a wedge of concrete or poured with rubber gasket type plug, or equivalent system of discharge.

43. We defer moisture proofing and water proofing recommendations to interior wall and floor covering manufacturer's suggested specifications and/or a moisture/water-proofing expert.

Concrete Slabs-on-Grade and Flat Work

44. Exterior and building floor slabs should not be supported on loose topsoil. They should be supported on firm native or engineered fill or designed to span across perimeter and/or continuous foundations. The exposed base of main house living room basement floor slab should be scarified at least 12 inches and recompacted to 92%. The shallowly supported flatwork (i.e. in the garage, patios and den basement slabs) should be situated on a re-compacted earth mat as described above in Option A or supported on helical piers Option B.

45. Slabs can be expected to suffer some cracking and movement. However, thickened exterior edges, a well-prepared compacted subgrade as described above; including pre-moistening prior to pouring concrete; adequately spaced expansion and control joints and good workmanship should minimize cracking and movement.

46. Hardscape patio improvements will behave best when supported on firm, non-organic topsoil or engineered fill as described in Option A; or alternatively they may be allowed to experience settlement while supported on loose soil. Otherwise they may be supported on helical piers.

47. Avoid placing slabs half on fill and half on native. Where necessary provide uniform substrate for slabs by either providing support entirely in cut or by providing support entirely in engineered fill.

48. Loose soil exposed under proposed flatwork should be removed to its full extent and replace with engineered fill. Depth of unsuitable soil shall be determined in the field.

49. Slab on grade floors may include a perforated drain pipe manifold system trenched into the subgrade below the capillary break; and be connected to an independent separate solid rigid pipe to daylight.

50. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab.

51. Where floor dampness must be minimized or where floor coverings will be installed, concrete slabs-on-grade should be constructed on a capillary break layer at least 4 inches thick and covered with a membrane vapor barrier. Capillary break material should be free draining, clean gravel or rock, such as 3/4-inch gravel. The gravel should be washed to remove fines and dust prior to placement on the slab subgrade. The vapor barrier should be a high quality membrane, such as Moistop by Fortifiber Corporation. A layer of sand about 2 inches thick should be placed between the vapor barrier and the floor slab to protect the membrane and to aid in curing concrete. The sand should be lightly moistened prior to placing concrete.

52. We defer moisture proofing recommendations to floor covering manufacturer's suggested specifications and/or a moisture proofing expert.

53. Exterior slab reinforcement should **not** be tied to the building foundations.

Utility Trenches

54. Trenches must be properly shored and braced during construction or laid back at an appropriate angle to prevent sloughing and caving at sidewalls. The project plans and specifications should direct the attention of the contractor to all CAL OSHA and local safety requirements and codes dealing with excavations and trenches.

55. Utility trenches that are parallel to the sides of buildings should be placed so that they do not extend below an imaginary line sloping down and away at a 2:1 (horizontal to vertical) slope from the bottom outside edge of all footings. The structural design professional should coordinate this requirement with the utility layout plans for the project.

56. Trenches should be backfilled with granular-type material and uniformly compacted by mechanical means to the relative compaction as required by county specifications, but not less than 95 percent under paved areas and 90 percent elsewhere. The relative compaction will be based on the maximum dry density obtained from a laboratory compaction curve run in accordance with ASTM Procedure #01557-91.

57. We recommend placing a concrete plug in the trench where it meets foundations to prevent water intrusion under the structure. Provide an evaluation pipe to drain collected water. Care should be taken not to damage utility lines.

58. Trenches should be capped with about 1½ feet of relatively impermeable soil.

Flexible Pavements

59. To have pavers or asphaltic concrete, aggregate base and subbase sections perform to their greatest efficiency, it is important that the following items be considered:

60. Grading should not be performed during inclement weather.

61. Remove unsuitable material, sub-excavate to specified grade, scarify exposed subgrade, moisture condition the subgrade and compact to a relative compaction of 95 percent at about 2 percent over optimum moisture content and tested by HKA.

62. Any fill material should be placed in thin lifts as engineered fill compacted to 95% at about 2 percent over optimum moisture content and tested by HKA.

63. Provide sufficient gradient to prevent ponding of water.

64. Base rock section should meet Caltrans Standard Specifications for Class II Aggregate Base, and be angular in shape.

65. Compact all engineered fill (if any) and baserock and subbase rock sections to a relative dry density of 95 percent. Contact HKA 4 days prior to earthwork so that the compaction curves samples of subgrade and baserock materials may be secured and tested in laboratory so that results are ready when field testing of compaction starts.

66. Place the asphaltic concrete in two lifts or as per Caltrans section 39-6.01. Place during periods of fair weather when the free air temperature is within prescribed limits per Caltrans specifications.

67. Provide a routine maintenance program.

Site Drainage

68. Roof and surface drainage must be controlled and discharged away from structures in a way so as not to allow ponding/infiltration adjacent to the foundation (especially at the basement walls) or cause erosion.

69. Roof drainage should include gutters and downspouts connected to a solid storm drain system that discharges collected water away from house and improvements in a dispersed way so as not to cause erosion.

70. Do not discharge roof or surface water into subdrains and vice versa. Subdrains must be discharged separately.

71. Surface drainage should include provisions for positive gradients so that surface runoff is not permitted to pond adjacent to foundations, flatwork and pavements. Surface drainage should be graded away from the building foundations, flatwork and directed to suitable locations. Positive gradients include 5% for 10' on hard compacted bare ground or 2% for hardscaped surfaces; or where grading is impractical 'area drains' or provisions that promote no infiltration near the foundations should be included in design.

72. Refer to slab-on-grade section for sub-slab drainage recommendations.

73. Basement, site and deep crawl-space walls must be drained their full height. Waterproofing of the drains will be of key importance. Refer to Retaining Wall Section of this report.

74. The migration of water or spread of extensive root systems below foundations, slabs, or pavements may cause undesirable differential movements and subsequent damage to these structures. Landscaping should be planned accordingly.

75. **Avoid planting and irrigation above backfill of basement walls.**

Plan Review, Construction Observation and Testing

76. Haro, Kasunich and Associates should be provided an opportunity to review project plans prior to construction to evaluate if our recommendations have been properly interpreted and implemented.

77. We should also provide foundation excavation observations and earthwork observations and testing during construction. This allows us to confirm anticipated soil conditions and evaluate conformance with our recommendations and project plans.

78. If we do not review the plans and provide observation and testing services during the earthwork phase of the project, we assume no responsibility for misinterpretation of our recommendations.

If you have any questions concerning the data or conclusions presented in this report, please call our office. We are pleased to be of service on this project.

Respectfully Submitted,

HARO, KASUNICH AND ASSOCIATES, INC.

Andrew Kasunich E.I.T.
Staff Engineer

Vicki Odello
C.E. 52651

VO/vo

Attachments

Copies: 1 electronic copy to Chris Adamski at cadamski@emersondevgroup.com
1 to Courtney Adamski at cadamski@carmelrealtycompany.com
1 to Craig Harwood kirnig@cruzio.com

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field. The conclusions and recommendations contained herein are professional opinions derived in accordance with current standards of professional practice. No other warranty expressed or implied is made.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by a geotechnical engineer.

APPENDIX A

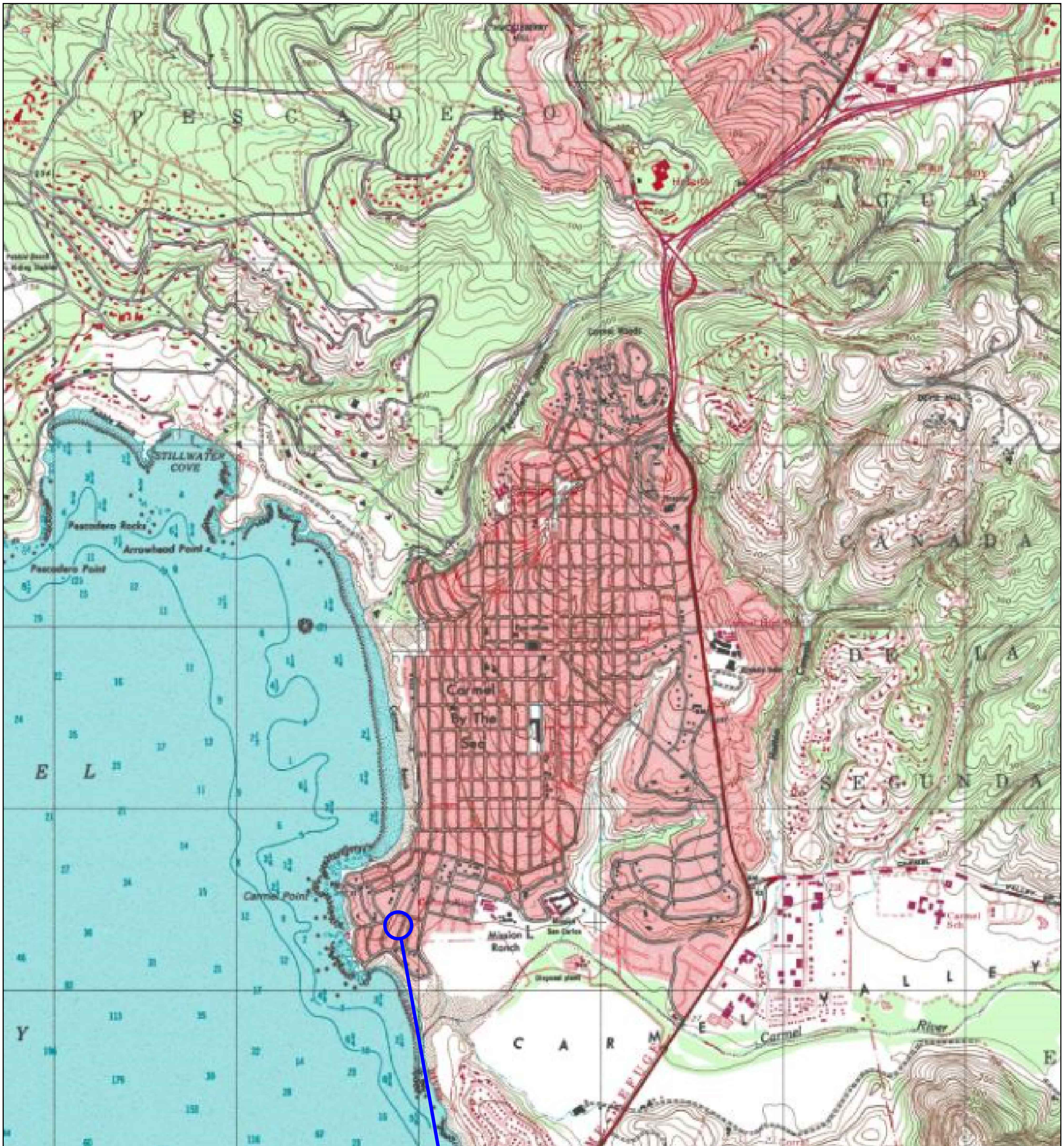
<u>Site Vicinity Map – Figure 1</u>

<u>Regional Boring Site Map – Figure 2</u>

<u>Site Specific Boring Site Map – Figure 3</u>

<u>Key to Logs – Figure 4</u>

<u>Logs of Test Borings – Figure 5 to 9</u>



NOTES:

SITE LOCATION

SITE VICINITY MAP
26307 Isabella Avenue APN 009-463-012
Carmel-By-The-Sea, California

SCALE: NO SCALE	
DRAWN BY: AK	
DATE: DEC 2017	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
REVISED:	
JOB NO. M11382	

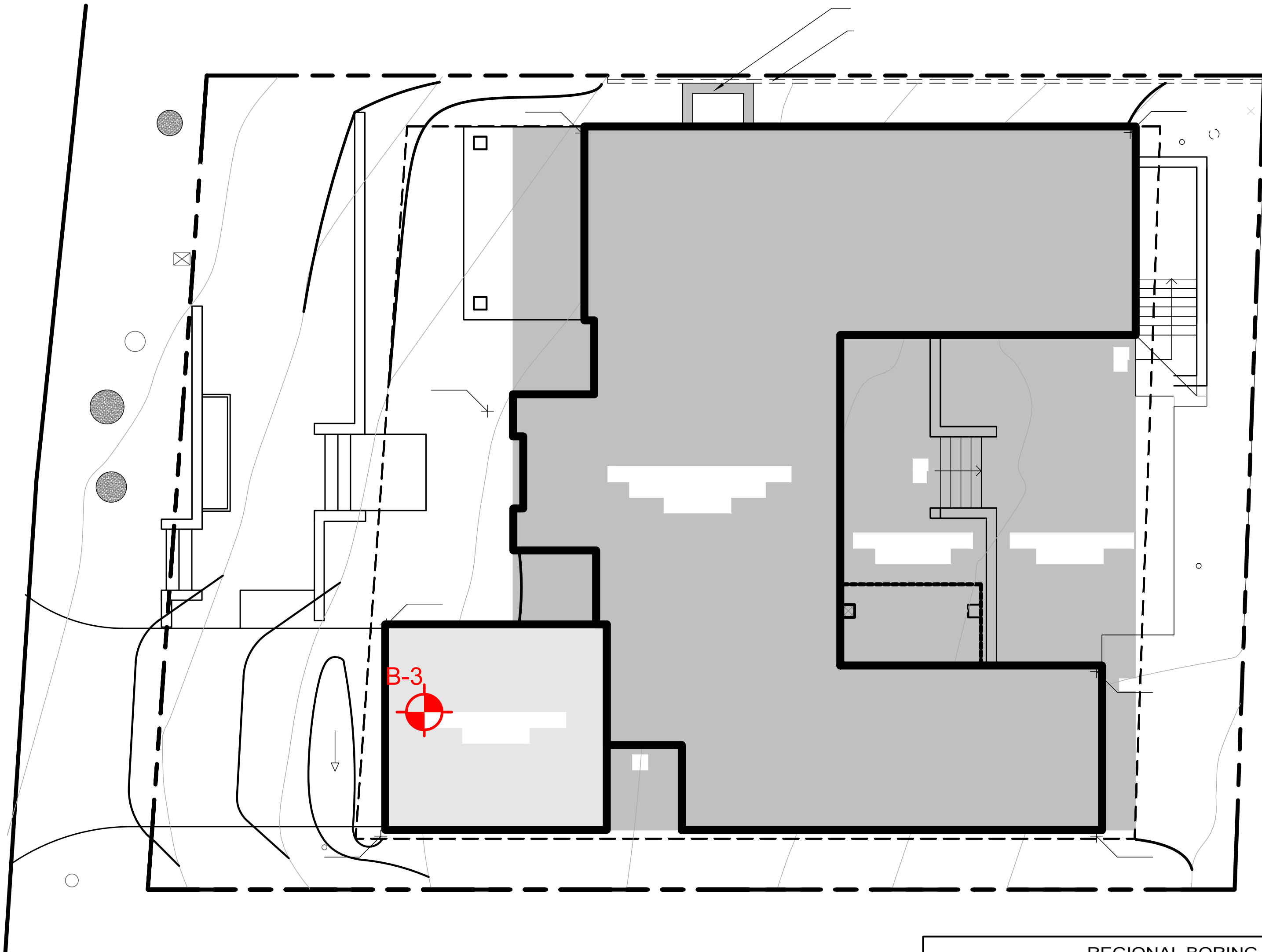
FIGURE NO. 1

SHEET NO.

Denotes Test Bore Hole Location

Base Map Ta□en From Goo□le Earth Ima□ery 2017

REGIONAL BORING SITE MAP 26307 Isabella Ave, APN 009-463-012 Carmel-By-The-Sea, California	
SCALE: NTS	
DRAWN BY: AK	
DATE: DEC 2017	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
REVISED:	
JOB NO. M11382	
FIGURE NO. 2	SHEET NO.



Denotes Test Bore Hole Location
Base Map Prepared By Tom Meaney Architect
Dated 6 April 2017

REGIONAL BORING SITE MAP
26307 Isabella Avenue, APN 009-463-012
Carmel-By-The-Sea, California

SCALE:	NTS
DRAWN BY:	AK
DATE:	DEC 2017
REVISED:	
JOB NO.	M11382

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 3

SHEET NO.

PRIMARY DIVISIONS				GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
				GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
			GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
				GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
		SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines
				SP	Poorly graded sands or gravelly sands, little or no fines
			SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
				SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS	MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
				OL	Organic silts and organic silty clays of low plasticity.
		SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
				CH	Inorganic clays of high plasticity, fat clays.
				OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS				Pt	Peat and other highly organic soils.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS PER FOOT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH (TSF)**	BLOWS PER FOOT*
VERY SOFT	0 - ¼	0 - 2
SOFT	¼ - ½	2 - 4
FIRM	½ - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

SAMPLING METHOD

STANDARD PENETRATION TEST	T	
MODIFIED CALIFORNIA	L or M	
PITCHER BARREL	P	
SHELBY TUBE	S	
BULK	B	

H_2O

Final	
Initial	
Water level designation	

*Number of blows of 140 lb hammer falling 30 inches to drive a 2" O.D. (1⅜" I.D.) split spoon sampler (ASTM D-1586)
**Unconfined compressive strength in tons/ft² as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

26338 Valley View Avenue

PROJECT NO. M11382

LOGGED BY VO/AK | DATE DRILLED 10-9-17 | BORING DIAMETER 6" | BORING NO. B-1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Dark brown Silty SAND, loose, dry	SM					
			Grades to poorly graded SAND, loose, dry	SP					
5			Medium brown poorly graded SAND, damp, medium dense (eolian)	SP					
10									
15									
20			Medium brown poorly graded Silty SAND with mica, damp, medium dense (fluvial)	SP					
25			Crunchy cobbles and gravels	GW					
			Smoother drilling, well graded gravel	GW					
30									
35	1-1 (M)				28				

SuperLog CivilTech Software, USA www.civiltech.com File: C:\superlog4\HKALOGS\M11382 26338 Valley View.log Date: 12/18/2017

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 5

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

26338 Valley View Avenue

PROJECT NO. M11382

LOGGED BY VO/AK **DATE DRILLED** 10-9-17 **BORING DIAMETER** 6" **BORING NO.** B-1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
35			From 36' to 40' grinding of rock, very weathered basaltic andesite bedrock	BR					
40	1-2 (L)		Dark grey brown weathered bedrock		50/6"				
	1-3 (M)		Same as above, volanic basalt is weathered rock		50/2"				
	1-4 (T)		Boring terminated at 42.5 feet						
45									
50									
55									
60									
65									
70									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\superlog4\HKALOGS\M11382 26338 Valley View.log Date: 12/18/2017

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr **FIGURE NO.** 6

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

26346 Valley View Avenue

PROJECT NO. M11382

LOGGED BY AK **DATE DRILLED** 10-9-17 **BORING DIAMETER** 6" **BORING NO.** B-2

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0								
	2-1-1(L)	Dark brown Silty SAND, loose, dry	SM	16		86.4	4.6	
	2-2 (T)			7			3.8	
		Same as above, poorly graded	SP					
5								
		Yellow fine grained poorly graded SAND, medium dense, damp	SP					
10	2-3-1 (L)			26		99.7	5.0	
	2-4 (T)			18			3.3	
		Same as above, (Fluvial)						
15								
20		Same as above, micaccous	SP					
25								
30								
		Lean CLAY in spoils, very dense	CL					
		Grinding with auger, highly weathered basaltic bedrock	BR					
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr **FIGURE NO.** 7

HKA Haro, Kasunich & Associates, Inc. Geotechnical and Coastal Engineers

26346 Valley View Avenue

PROJECT NO. M11382

LOGGED BY AK **DATE DRILLED** 10-9-17 **BORING DIAMETER** 6" **BORING NO.** B-2

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
35				BR					
40			Very dark brown, highly weathered basaltic bedrock	BR	50/6"				
			Boring terminated at 42.5 feet						
45									
50									
55									
60									
65									
70									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\superlog4\HKALOGS\M11382 26338 Valley View.log Date: 12/18/2017

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr **FIGURE NO.** 8

Haro, Kasunich & Associates, Inc.

26307 Isabella Avenue

PROJECT NO. M11382

LOGGED BY AK **DATE DRILLED** 10-9-17 **BORING DIAMETER** 6" **BORING NO.** B-3

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0								
	3-1-1 (L)	Dark brown Silty SAND, dry, loose	SM	15		81.5	4.7	
	3-2 (T)			17			3.3	
5								
		Poorly graded, medium yellow, medium graded SAND (eolian)	SP					
10	3-3-1 (L)	Same as above, medium dense (eolian)		24		92.1	4.0	
	3-4 (T)			22			1.4	
15								
20		Same as above						
25								
		Decomposed basaltic bedrock, very dense Groundwater at 29.25	BR					
30			BR	50/6"				
		Boring Terminated 31.5 feet						
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr **FIGURE NO.** 9

GEOLOGIC EVALUATION
PROPOSED RESIDENCE AND ASSOCIATED IMPROVEMENTS
26307 ISABELLA AVENUE
CARMEL-BY-THE-SEA
MONTEREY COUNTY, CALIFORNIA

November 22, 2017

Prepared for

Emerson Development Group

Prepared by

Craig S. Harwood
Consulting Engineering Geologist
Ben Lomond, California

Craig S. Harwood
Engineering Geologist
239 Park Drive
Ben Lomond, CA 95005
tel 831 325-9327
email kirnig@cruzio.com

File No. G-790.3

November 22, 2017

Emerson Development Group
3375 7th Avenue
Carmel-By-The-Sea, CA 93923

Attention: Chris Adamski

Project: **Proposed Residence**
26307 Isabella Avenue
Carmel, California

Subject: **Geologic Evaluation**

Dear Mr. Adamski:

As you authorized, presented herein is the geologic evaluation of the site of the proposed residence and associated improvements located at 26307 Isabella Avenue, Carmel, California. This report has been prepared for your use in developing the property for the proposed improvements. The report describes the site geologic characteristics, identifies potential geologic hazards, and provides recommendations for the proposed improvements. Three hard copies and one digital copy of this report are submitted to you for distribution to others. One additional digital copy has been provided to the project geotechnical engineer, Haro, Kasunich & Associates, Inc. This concludes our work for the current phase of the project.

I appreciate the opportunity to have provided geologic services for this project and look forward to working with you again in the future. If there are questions concerning this report, please contact me at your earliest convenience.

Sincerely,

Craig S. Harwood
RG #6831/CEG #2275

REGISTERED GEOLOGIST
CRAIG S. HARWOOD
exp 3/31/18
C.E.G. No. 2275
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Distribution: Chris Adamski (3 signed hard copies + 1 digital)
Haro, Kasunich & Associates (Vicki Odello) (1 digital)

TABLE OF CONTENTS

Introduction/Purpose/Scope of Services

Based upon my discussions with Chris Adamski we understand that development of the site (located at 26307 Isabella Avenue) will consist a new two-story, wood frame residence which will include a standard height basement. The design grades will require very minimal grading other than excavating the basement. The site is currently undeveloped. A geotechnical investigation of the site currently being conducted by Haro Kasunich and Associates (in preparation).

This geologic evaluation report has been prepared to evaluate and define the geologic conditions and identify potential geologic hazards associated with the project site, and to offer recommendations that help to minimize the impact of those hazards on the proposed project. The scope of work included but is not necessarily limited to; review of available geologic reports and maps, a review of stereo aerial photo pairs covering the site area, geologic mapping of the site, logging of a coastal bluff fault exposure, co-logging of an exploratory boring alongside Haro, Kasunich and Associates, Inc., reviewing exploratory borings conducted on adjacent lots, and evaluation of the data collected and preparation of this report. The scope of our work for the current evaluation is intended to comply generally with "Guidelines for Geologic/Seismic Reports", a publication of the California Division of Mines and Geology (CDMG Notes No. 37), as referenced by the Monterey County Planning Department (Monterey County Code – Section 20).

It is our intent that this report be used exclusively by the client and the client's architect/engineer to form the geologic/seismic basis of the design of the proposed project as described herein, and in the preparation of plans and specifications.

Site Setting

The project site is located within the Carmel Point area in the community of Carmel-by-the-Sea, in Monterey County. The Site Geologic Map (Appendix A) presents a more detailed depiction of the existing physical features of the site. The site is located on the southeast side of Isabella Avenue. It is undeveloped and there are existing residences located on the southwest and on the northeast. Two undeveloped ajoining lots are located on the southeast. The subject lot and the two above mentioned undeveloped lots are all owned by the client (Emerson Development). The site exists at an average elevation of 44 feet above MSL and there is about 7 feet of topographic relief across the site and about 3 feet of relief across the residence building pad area. Drainage patterns at the site are a function of the physiography. Surface runoff generally sheets downslope toward the southeast property line. There is a sparse growth of Monterey Pines and understory shrubs around the site.

Regional Geology

The site is located within the coast range geomorphic province of central California. Throughout the Cenozoic Era central California has been affected by tectonic forces associated with lateral or transform plate motion between the North American and Pacific crustal plates, producing a complex system of northwest-trending faults that comprise the San Andreas Fault system (Page, 1998). Uplift, erosion and subsequent re-deposition of sedimentary rocks within this province have been driven primarily by the northwest directed, strike-slip movement of the tectonic plates and the associated northeast oriented compressional stress. The northwest-trending coastal mountain ranges are the result of an orogeny (formation of mountains by the process of tectonic uplift) believed to have been occurring since the Pleistocene epoch (approximately 2-3 million years before present). The portion of the coastal region where the site exists is within the Salina Block, which is bound by the San Andreas fault on the east, and by the San Gregorio - Palo Colorado fault system to the west. The Salina block is composed of an elongate prism of granitic and metamorphic rock types. The Salina basement complex is overlain primarily by marine sedimentary rocks of tertiary age and terrestrial rocks of Pliocene to Pleistocene age. Subsequent uplift, and sea level changes have modified the formations along the coastal corridor and produced coastal terraces which are comprised of alluvial deposits.

Local Geology and Geologic Reconnaissance

Published maps covering the regional geology in the general vicinity of the site include those by Clark et al. (1974), Ross (1976), Greene (1977), Clark et. al., (1997), Dibblee (1999), Rosenberg and Clark (1999), and Rosenberg (2001) and the Dibblee Foundation (2007). Additional publications reviewed for this study are discussed in later sections of this report under the appropriate subject headings. These regional maps are based upon aerial photo interpretation, reconnaissance style mapping and field checking at sparsely distributed locations in the area and do not include site-specific data, although they are generally useful in establishing regional context. For our characterization of the site we have adopted the geologic mapping unit nomenclature of Clark and Rosenberg (1997). A portion of that map is reproduced as the Regional Geologic Map (Appendix A).

The map of Clark and Rosenberg (1997) shows that the site exists on an emergent or elevated marine terrace ("Lighthouse Coastal Terrace") which forms a layer of deposits overlying a bedrock platform comprised of granodiorite ("Kgdp") and volcanic rocks (Tvb). These bedrock units are shown on numerous published maps to be in fault contact (see Faulting). The Lighthouse Coastal Terrace (Qtcl) deposits have been dated at 102,000 years old which places the terrace in the middle Pleistocene. These deposits consist of dune sand and an underlying fluvial sequence. A surficial layer of residual soil several feet thick overlies the dune sand. The residual soil within the uppermost 3 feet or so has been dated through the C_{14} radiocarbon method, producing age dates within the uppermost soils extending back to 840 years before present. However local archealogists infer that these upper soils may extend back to as early as 8,000 years before present suggesting there are early Holocene (Archealogical Consulting, 2012).

A geologic reconnaissance of the site and adjacent areas was performed on September 29, 2017 for the purpose of observing features depicted on published maps, and making and recording field observations at natural and man-made exposures. There are no exposures of subsurface materials at or near the site, however the coastal bluffs along Scenic Road 1,200 feet north-northwest of the site provided an extensive natural outcrop exposing the primary geologic units that comprise the terrace; coastal terrace deposits (Qctl), granodiorite (Kgdp) and volcanic rocks (Tvb). Additionally, the Cypress Point Fault zone, which juxtaposes the granitic rock (on the west) against the volcanic rocks (on the east) is partially exposed at that same bluff (see Faulting). The grantitic bedrock (porphoritic granodiorite) is a massive crystalline outcrop that represents the dominate rock type in the northern Santa Lucia Range and, to a lasser degree the Monterey Peninsula. The Tvb unit as exposed along Scenic Road to the northwest of the site consists of flows, flow breccias and agglomerates. These volcanic flows have variable moderate to steep dips toward the northeast. The dominant structure within the Granodiorite as exposed along Scenic Road consists of through-going joint sets with a northwest strike. This structure is disrupted immediately adjacent to the fault zone. The coastal terrace deposits (Qctl) exposed along the coastal bluff exhibit fluvial features and textures including a channelized zone containing a concentration of large clasts adjacent to and within the Cypress Point Fault exposure. At the coastal bluff near the fault zone the terrace deposits vary from 8 to 11 feet thick, however the higher elevation areas across the Carmel Point area is underlain by a thick accumulation of Eolian Dune Sand overlying the fluvial deposits that are exposed at the bluff edge.

Concurrent Geotechnical Investigation

Haro Kasunich & Associates (HKA) is currently conducting an ongoing geotechnical investigation of the site for the proposed project (report in preparation). Their investigation included co-logging an exploratory boring at the site. Additionally, subsurface data collected at two additional borings on the two adjacent lots (26338 Valley View Avenue and 26346 Valley View Avenue) assisted in establishing the larger geologic context surrounding the site relevant to the fault issue. The cluster of three subadjacent sites are all owned by Emerson Development Group. The borings were used to obtain field blow count information and collect subsurface samples for engineering purposes, and to identify the bedrock type at depth. The graphic logs of the borings is included in Appendix C. The bedrock type was relevant to identifying whether or not the Cypress Point Fault trends through any of the three adjacent parcels, as is suggested by the published maps covering the area (see Faulting). The drilling was accomplished with a truck-mounted Mobile B-53 drill rig using standard geotechnical sampling. All three borings (B-1, B-2 and B-3) generally encountered a surficial residual soil overlying terrace deposits (Qctl) which are, in turn underlain by volcanic bedrock (basaltic Andesite; “Tvb”). The borings indicate that the highly weathered zone along the top of the bedrock is encountered at depths of 35.5 feet (B-1) and 33.75 feet (B-2) 27.75 feet (B-3). The surficial residual soil was found to be in a loose condition and the underlying Coastal Terrace deposits were found to be in a medium dense to very dense condition based on field blow counts. The borings on all three subadjacent lots consistently encountered volcanic bedrock within the basal portion of the borings. The uppermost several feet of the volcanic bedrock was much more deeply weathered (disintegrated). Deeper into the bedrock the conditions were less weathered and the bedrock more competent which typically resulted in a grinding effect on the lead bit as it was advanced, as well as practical sampler refusal (+100 blows per foot).

Concurrent Geophysical Investigation (JR Associates, 2017)

JR Associates has recently conducted a geophysical investigation at the site as well as an array along Scenic Road for the purpose of addressing the fact that the Cypress Point Fault is shown on some published maps as projected through or immediately adjacent to the site (depending on source). The geophysical investigation at both locations consisted of parallel shear wave velocity and seismic refraction lines which were oriented to intersect the projected fault as close to perpendicular as possible. The array located on the two adjacent lots on the southeast extended from the northeast corner of 26338 Valley View Avenue and trended southwesterly through 26346 Valley View Avenue. This geophysical array shadowed the subject site with respect to the mapped fault zone, as mapped on the county’s GIS mapping and that of the USGS (2006). Our borings B-1 and B-2 were located on the geophysical survey lines, on opposing ends. The parallel geophyscal lines along Scenic Road were located so that they traversed over the top of the coastal bluff exposure of the fault zone thereby providing a geophysical signature of the fault zone as it transitions from the granodiorite (on the west) into the volcanic rocks (on the east). The justaposition of bedrock types, their differing properties, and the disruption and relative (apparent vertical) offset of the terrace deposits (Qctl unit) through the fault zone is clearly discernible in the geophysical profiles (Appendix B). The Faulting discussion presents our inferences made from the geophysical study, and a copy of the JR Associates report is included in Appendix B.

Groundwater

We encountered groundwater at only one of our three boring locations (at B-3 at the subject) at a depth of 29.25 feet. Based on our experience in the immediate area and our review of exploatory borings conducted on nearby parcels, we consider this to be indicative of a localized condition rather than evidence of a regional groundwater table. In general, groundwater conditions and fluctuations in the level of subsurface water are possible due to variations in rainfall, temperature, irrigation and well withdrawal patterns and other factors.

Landsliding

The site is nearly level (with approximately 6 feet of topographic relief across the lot). There are no slopes located anywhere near the site and the nearest coastal bluffs are located at least 570 feet south. Our review of published literature and maps covering the area does not depict any landslides in the area of the site [Clark et al. (1974), Ross (1976), Greene (1977), Clark et. al., (1997), Dibblee (1999 and 2007), Rosenberg and Clark (1999), Rosenberg (2001)]. There are no conditions at or near the site that would or could generate debris flows hazards for the site.

Faulting

The San Andreas Fault system and related fault systems in the region generally strike northwest and are characterized by a combination of strike-slip and reverse displacement. Refer to the Regional Fault Map (Appendix A) for the relative locations of some of the regional faults. Some active faults in the region include (in order of increasing distance from the site): the Monterey Bay-Tularcitos fault system (6.3mi./9.5km northeast), the San Gregorio-Palo-Colorado fault system (7.9mi./ 11.9km. west), the Rinconada fault zone (16.2mi./24.54km east), the San Andreas fault ("Pajaro" and "Creeping" segments; 29mi./44km northeast), the Calaveras fault southern extension (35.8mi./54km. northeast) and the Hayward fault-southeast extension (49mi./74km. northeast) (Jennings, 1994). Additional local faults which have yet to be classified as active (undivided Quaternary activity status) include the Ord Terrace fault, the Seaside fault, the Berwick Canyon fault, the Navy Fault, the Chupines fault and more locally, the Cypress Point Fault (Rosenberg, 2001). The Cypress Point Fault was first recognized by Bowen (1969) who mapped it from Pescadero Point 3 km northwestward to Cypress Point and showed the northwest side down relative to the southwest. It includes at least 3 en-echelon faults at Fanshell Beach with possible right lateral strike-slip displacement.

According to the California Division of Mines and Geology, the Cypress Point Fault ("CPF") is designated as "undivided Quaternary" in terms of the activity status, due to the fact that the youngest geologic formations that have been cut by this fault are younger than 1,600,000 years old. A one-eight mile wide county-designated regulatory zone (fault rupture hazard) has been established by the county along the CPF fault surface trace. A state-mandated Earthquake Fault Zone has not been established along the Cypress Fault Zone however technically the fault does meet criteria for zoning under the state AP-Zoning act. In preparing the geologic section of the 2001 General Plan Update, Rosenberg classified the CPF fault as "potentially active". Typically, within the professional geologic practice in California, Pleistocene active faults (most recent activity restricted to the Pleistocene epoch) are given a 25-foot building setback for habitable structures unless specific studies justify a narrower setback. Regarding the Cypress Point Fault Rosenberg states;

> At Carmel Point vesicular Carmeloïte flows and Carmeloïte flow breccias are faulted against Cretaceous granodiorite to the southwest in a 4 to 7-m-wide brecciated zone. However, in May 1993, severe beach erosion revealed a 60-m-long exposure of the fault striking N50°W, implying that the faults at Pescadero Point and Carmel Point are en

echelon segments rather than continuous. Clark and others (1974) showed the Cypress Point fault continuing southeastward across Carmel Point, where it was concealed beneath Quaternary sediments, and postulated that it separated Carmeloïte mapped at the mouth of the Carmel River by Lawson (1893), but no longer exposed, from presumably granitic basement to the southwest. Exploratory drilling in the parking lot of Carmel River State Beach encountered Carmeloïte at an elevation of 0.6 m, striking Lawson's "lost outcrop" (Staal, Gardner & Dunne, 1989).

The USGS Quaternary Fault and Fold Database (2006) characterizes the Cypress Fault follows:

> A poorly studied dextral reverse fault that offsets Paleocene Carmelo Formation against Mesozoic crystalline basement rocks. Clark (1989 #6148) mapped a coastal terrace he estimated to be 102 ka as offset along the Cypress Point fault. McCulloch and Greene (1990 #5406) mapped undifferentiated Quaternary deposits offset along an offshore trace of the fault.
>
> Minor northwest-striking fault extends about 12 km from about 3 km northwest of Cypress Point southeast across Carmel Point to near Palo Corona Ranch on the south side of Carmel Valley. Clark (1989 #6148) reported that dip-slip separation may be less than 20 m of down to the northeast offset….Fault is delineated by an eroded east-facing scarp in crystalline basement rocks (Bryant, 1985 #6135). Geomorphic evidence of late Pleistocene to Holocene offset was not observed by Bryant…..Rosenberg and Clark (1994 #6144) reported a late Quaternary vertical slip rate of 0.01 mm/yr, based on a 1 m vertical displaced coastal terrace estimated by Clark (1989 #6148) to be about 102 ka.

Some sources indicate the segment of the CPF that trends through the Carmel Point area as a Pre-Quaternary fault (Dupre, 1990; Staal, Gardner & Dunne, 1994) but we were able to detect evidence of offset within the basal portion of the coastal terrace deposits at the Scenic Road coastal bluff exposure and similar observations have been made at this same exposure by other authors as well (Clark, 1989; Rosenberg and Clark, 1997; Rosenberg 2001; Stahl, Gardner and Dunne, 1994). As for the style of deformation, although some previous workers have characterized it as a high angle reverse fault (down on the east), our interpretation differs from that interpretation (see later discussion, this section). As evidence for this dip-slip deformational style, Stahl, Gardner and Dunne noted the vertical relief across the faulted bedrock platform at Carmel River State Beach (Granodiorite on west / volcanics on the east). This relief may however be due to differential erosion of the differing bedrock types by action of the Carmel River at the river mouth. The granodiorite is generally much less weathered and more competent than the volcanics. This scouring theory was first suggested by The Geophysics Group during their study of the Carmel River Mouth (The Geophysics Group, 1989). However, some geologists have noted several lines of evidence suggesting right-lateral displacement east of the Cypress Point fault location including; the relatively straight trend, its en echelon character, and the parallelism of this fault to the faults of the Monterey Bay fault zone, on which first-motion studies indicate right-lateral slip (Dupré, written communication in 1989 in Rosenberg 1994). Rosenberg opined in 2001 that the larger view of the CPF fault was that it was probably a strike-slip fault. Our recent observations have confirmed this (see below).

As part of our work, we reviewed a series of vertical and oblique aerial photos and more recent Google Earth®

images covering a period between 1956 through 2014. The photos reveal a liner slope break along the ridge that exists south of Carmel River State Beach. This is aligned with the mapped projection of the CPF fault. The photos also reveal the abrupt break in structural patterns between the granodiorite and the volcanic rocks at the Scenic Road bluff fault exposure. Low altitude oblique photography availabe through the California Coastal Records Project were useful in obtaining a good perspective on the Scenic Road exposure both prior to and after the application of shotcrete over the main fault zone.

We logged the Scenic Road bluff exposure of the fault zone (see Log of Fault Exposure, Appendix A). Here the fault consists of a roughly 32 foot wide zone with the central 15 feet being obscured by a wall of shotcrete which was applied apparently to help prevent further bluff retreat due to erosion and sloughing of the bluff face. Additional data points reviewed as part of the background research for this project indicates the fault zone narrows as it trends southeasterly from this bluff exposure. At the Scenic Road bluff the Graniodiorite is exposed on the west side of the shotcrete curtain, and volcanic rocks are exposed on the east side of the shotcrete curtain. Our examination of this exposure revealed a prominent 1.5 inch thick shear zone consisting of fault gouge. Slickensided fault surfaces within this gouge contain striations dipping 15° toward the southeast. This suggests a strike-slip deformational style but with a relatively small-scale vertical component (the vertical component of slip representing 17% of the horizontal component). Our measurements and the adjacent geophysical array on Scenic Road which spanned the fault zone just back from the bluff crest confirms a slight down-drop on the east of the fault zone, however this pattern is not consistent with the results of a geophysical survey across the fault zone adjacent to 26339 Isabella Avenue (for another project unrelated to the current subject site; see Faulting).

Previous subsurface explorations have helped to bracket the location and trend of the fault zone as it trends through the Carmel Point area. These include an investigation by Haro Kasunich & Associates (1997) for a site located on Inspiration Avenue (located 470 feet north-northwest of the site), as well as a compilation by Pacific Geotecnical Engineerimg (2013) of previous data and additonal explorations conducted at the Carmel River State Beach area (825 feet southeast) which bracketed the location of the fault zone as trending southeasterly through the midde of the state beach parking lot. As already mentioned, an additonal data point occurs at Scenic Road where the fault zone is exposed (1,200 feet north-northwest of the site). The geophysical survey across the adjacent lots on Valley View Avenue (26338 and 26346) revealed that the depth to the top of the bedrock was consistent between the geophysical array and the borings B-1 and B-2. That geophysical survey also confirmed that the geophysical properties of the volcanic bedrock across along the geophysical array (across the two adjacent sites) is consistent with those same properties of that same formation located east of the fault zone at the coastal bluff exposure. Hence the volcanic formation is consistently present beneath the terrace deposits across the site subsurface and is not impacted by fault surface movement. During the period of our site investigation we also performed a geologic investigation of an nearly adjacent site (26337 Isabella Avenue). As part of that sudy JR Associates conducted a geophysical survey along Isabella Avenue fronting 26339 Isabella which confirmed the location of the surface trace of the fault zone. The fault zone appears to trend through the neighboorhood and clips the far southwest corner of the subject site as shown on the Site Geologic Map (Appendix A; see also Fault Surface Rupture). Interestingly, the relative offset along the geophysical array at Scenic Road suggests a relative down–drop on the east side of the fault zone whereas a recent geophysical array conducted at 26339 Isabella Avenue suggests relative uplift on the east side of the fault zone. These inconsistent relative changes in the surface of the bedrock platform suggests that a strike-slip deformational style is more probable than a dip-slip (down on the east) deformational movement (see later discussions).

Historical Earthquakes

Within historic time, significant earthquakes have severely damaged man-made structures over a large part of the central coastal area surrounding the Monterey Bay area. These earthquakes included the 1906 M 8.3 San Francisco (Lawson, 1908), the 1926 Monterey Bay doublet, the 1984 M 6.2 Morgan Hill (Stover, 1984), and the 1989 M 7.1 Loma Prieta earthquakes (Shakal, 1989; Rosenberg, 2001). The 1989 Mw 6.9 October 17, 1989 Loma Prieta Earthquake is notable because it was a major earthquake event with an epicentral area located within the general region of the site (38 mi/58 km north) and resulted in widespread damage throughout the central coastal region. Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The Monterey Bay – San Francisco regions is one of the most seismically active areas in the country. While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earth- quake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults. During such an earthquake the danger of fault surface rupture at the site is slight, but very strong to severe ground shaking would occur.

During such an earthquake the danger of fault surface rupture at the site is slight, but very strong ground shaking would occur.

Primary Seismic Hazards

Ground Shaking

The severity of ground shaking during an earthquake depends upon a number of factors examples of which include earthquake magnitude, epicenter distance to site, local geologic conditions, thickness and wave-propagation properties of earth materials, groundwater conditions, and topographic setting. Ground shaking from a seismic event is considered the primary hazard that will impact the proposed residence additions during its design life span. According to the 1997 Uniform Building Code (ICBO, 1997, Figure 16.2), all of Monterey County lies within Seismic Zone 4, the most active seismic zone rated.

Rosenberg (2001) indicates Monterey County is subject to very strong (0.3 - 0.6g) to severe (greater than 0.6g) shaking from the Holocene age active faults in the county, including the San Andreas, the San Gregorio, the Reliz/Rinconada or the Monterey Bay-Tularcitos fault zones. The above-listed faults are considered the key seismic sources in the vicinity due to their location relative to the site, their slip rate, the maximum moment earthquake that these faults are capable of, and the fault rupture surface area. However, as mentioned earlier, there are a number of potential sources of large magnitude earthquakes in the region. The Monterey County General Plan Update (2001) suggests there is a 10% probability that a 0.45g level of ground shaking could occur in the vicinity of the site in the next 50 years – the typical design life of a wood frame structure (Rosenberg, 2001).

Surface-Fault Rupture

Earthquakes are generally caused by a sudden slip or displacement along a zone of weakness in the earth's crust, termed a fault. Surface-fault rupture is a manifestation of the fault displacement at the ground surface and is usually associated with moderate to large-magnitude earthquakes (M > 6.5: Sutch and Dirth, 2003). The amount of surface-fault displacement depends on the earthquake magnitude and other factors. The displacements associated with surface fault rupture can have devastating effects to structures and lifelines situated astride the zone of rupture.

As already noted, this evaluation has confirmed that the Cypress Point Fault "CPF" apparently trends through the neighboorhood, the nearpoint of which is approximately 35 feet southwest of the southwest property corner. The North County Land Use Plan defines high hazard areas to include zones 1/8 mile on each side of active or potentially active faults (i.e. faults that cut Quaternary age formations). The county general plan update (2001) states; " All structures shall be sited a minimum of 50 feet from an identified active fault or potentially active faults.[1] It should be noted that the Quaternary geologic period is divided into two time frames; 1) the Pleistocene (extending from 1,600,000 years before the present to 11,000 years before present), and the 2) the Holocene (beginning at 11,000 years before the present day). Fifty-foot fault-building setbacks have traditionally been used for Holocene faults (as opposed to Pleistocene active faults) however this arbitrary fault setback width has never been adequately justified on any scientific or technical basis. It was only a suggestion by state geologists that ''Unless proven otherwise, the area within 50 feet of an active fault is presumed to be underlain by active branches of the fault'' (Bryant and Hart, 2007, p. 2). Some federal and state jurisdictions even have mandated setbacks of 200 ft (61 m) in an equally unsubstantiated belief that an increase in setback would provide an increase in safety (e.g., California Integrated Waste Management Board, 2002, pp. 13, 59). As pointed out by Glenn Borchardt in his paper "Establishing Appropriate Setback Widths for Active Faults", "The age of a freeboard soil and the mature width of an associated shear zone may be used to determine the probable hazard due to SFR. The "freeboard soil" in the current situation would be represented by the Coastal Terrace Deposits (i.e., the Pleistocene "Lighthouse Coastal Terrace") which has been determined to be approximately 102,000 years old.

A late Quaternary vertical slip rate of 0.01 mm/yr, has been assigned to the CPF based on a 1-meter vertical displaced coastal terrace estimated by Clark (1989) to be about 102 ka (102,000 years old). However our examination of slip vectors (striations on a slickensided fault surface) at the Scenic Road exposure suggests that the vertical component of slip may be on the order of 17% of the total slip rate (or 15° rake of striations on the fault plain) which is largely horizontal (i.e., the horizontal component being 83% of the geologic slip rate). This suggests a horizontal component of geologically driven slip equal to approximately 0.008mm/yr., a very low value and the vertical component is even lower. The Scenic Road bluff exposure indicates the fault offset is restricted to the basal portion of the 102 ka terrace where the terrace deposits are relatively thin (on the order of 8 to 11 feet). The terrace deposits are substantially thicker in the area of Valley View Avenue where there is approximately 28 feet of unfaulted terrace deposits overlying the fault zone and adjacent areas. Propagation of a vertical component of fault surface rupture would most likely be somewhat dissipated through such a thick section of semi-consolidated to unconsolidated alluvium. Nevertheless the CPF fault is a Quaternary fault and

[1] Policy 2.8.3 A2 Geologic Hazards), item 2. Monterey County General Land Use Plan, 2001.

the county follows the state law which dictates that habitable structures should not be built astride Quaternary faults (Alquist-Priolo Earthquake Fault Zoning Act of 1972).

We agree with the judgement of Rosenberg in his geologic summary presented of the 2001 General Plan Update that the likelihood of fault surface rupture along this particular fault is very low and the magnitude of displacement is anticipated to be very small. In order to establish a building-fault setback we conducted an exercise by first determining the potential fault surface displacement. To determine the magnitude of fault surface rupture (displacement) we considered the method of Wesnousky (2008) who has developed regression curves relating the parameters of fault surface length, and estimated average fault displacement. We measured the length of the mapped surface trace (per Rosenberg, 1994) as an indicator of a potential surface rupture length (12 km).

Table 1: Fault Displacement Components – Cypress Point Fault

Deformational Style[1]	Length[2]	Horizontal Compnent of Displacement[3]	Vertical Compnent of Displacement[4]
Oblique (83% Strike Slip / 17% dip slip)	12 km	30.8 inches (2.56 feet)	6.3 inches (0.53 feet)

1 Confirmed in this study to be approximately 83% horizontal/ 17% vertical movement
2 U.S. Geological Survey Quaternary Fault and Fold Database (2011)
3 Per the regression of Wesnousky (2008)
3 Dip slip component of movement is equal to 17% of calculated horizontal component)

Mitigating Fault Surface Rupture (Fault-Building Foundation Setback)

We used the measured vertical compnent of fault surface rupture for the Cypress Point Fault (17% of the total anticipated offset) and calculated the fault-building setback based on the relation of Batatian & Nelson (1999) that has been adopted by Salt Lake County, Utah and is a recommended method in the San Luis Obispo County (California) Geologic Report Guidelines[2].

Hanging Wall block: $S = U (2D + F/\tan Ø)$
Foot wall block: $S = U \times D$

Where: S = fault setback, U is a constant (1.5), D = predicted fault surface vertical component of displacement (calculated at 6.3 inches or 0.53 feet), F = maximum depth of building footing (12 feet at the basement), and Ø = dip of fault (fault angle Ø = 74°) as measured at the Scenic Road exposure. If we use the geometry of the easterly bounding fault exposed at the Scenic Road bluff as a guide (down on the east/ fault inclined 43° to the southwest) then we must assume that same bounding fault adjacent to the subject site would dip away from the proposed habitable structure. Therefore, the adjacent proposed building is located on the "foot wall" side of the easterly bounding fault trace and the equation takes the simplier form of: S = U x D.

[2] This method has also been used in the City of San Jose Jurisdiction.

Table 2: Building Foundation - Fault setbacks for Cypress Point Fault

Fault angle[1]	Foundation depth	Total Displacement (vertical component)	Calculated[2] Setback on hanging wall side	Calculated[2] Setback on footwall side
43°	N/A	0.53 feet	N/A	0.80 feet

1 as measured at the Scenic Drive bluff exposure

2 Per Batatian & Nelson, 1999.

Given the very low level of hazard posed by the Cypress Point Fault, the relatively small estimated fault displacements and the calculated setback values, we judge that the proposed residence which is located approxiumately 35 feet northeast of the fault zone is not potentially directly impacted in terms of fault surface rupture.

Secondary Seismic Hazards

Soil Liquefaction

The Monterey County General Plan Update indicates the site is located within a zone that is designated as having a low potential for liquefaction (Rosenberg, 2001). Due to the presence of very dense sedimentery bedrock at very shallow depths and the lack of a laterally continuous groundwater table in the area, we concur with this interpretation and judge that the potential for liquefaction impacting the site is low.

Seismically-Induced Landsliding

The subject site is located within a zone designated as having a low potential for seismically-induced landsliding (Rosenberg, 2001). This is primaruily due to the fact that the site area is located on a generally competent and stable bedrock platform that lacks evidence of past landsliding, and although localized instability can occur along the coastal bluffs, the nearest bluffs are located 570 feet to the south of the site. The subsurface conditions at the site and the minimal topographic relief in the area indicates there is low potential for seismically induced landslides to impact the site.

Other Secondary Effects of Seismic Shaking

Seismically induced settlement of sufficient magnitude to cause structural damage is normally associated with poorly consolidated, predominantly sandy soils, or variable consolidation characteristics within the building areas. The presence of medium to very dense Pleistocene terrace deposits at the site indicates there is a low potential for this particular phenomenon to occur within the residence building envelope. The site is located on an elevated marine terrace at at an elevation of approximately 44 feet above mean sea level and at least 570 feet north of the nearest coastal bluffs. Due to the topographic position and geographic location, the potential for the site to be affected by Tsunamis is very low. This is judgement is consistent with Rosenberg's characterization of the nearby bluffs for the 2001 County General Plan Update. No bodies of impounded water are known to be located proximal to the subject property. Therefore, the subject site is not susceptible to the effects of seiches.

DISCUSSION

Developing property in the rugged, seismically active coastal region of central California carries with it a somewhat elevated level of risk from geologic hazards when compared to areas of the state where the geologic hazards are generally lessened by the lack of topographic relief, seismicity and proximity to active faults. Persons developing land in this region must be cognizant of this fact, and willing to accept this somewhat elevated level of risk. Furthermore, whereas the level of risk can be reduced to an acceptably low level by implementing mitigative measures (for example, building setbacks from potential hazards, or adherence to building codes), the risk cannot be totally eliminated. Modern building codes are intended to prevent collapse of structures but not to preclude the need for significant repairs or even rebuilding after a major earthquake.

Changes to the natural conditions at or adjacent to the site can directly affect the risk levels from geologic hazards to the proposed development. For example, grading activities (cutting or filling), altering natural drainage characteristics, removing vegetative ground cover or excessive landscape irrigation activity can upset the natural equilibrium of forces and conditions present in a slope therefore, increasing the risk from geologic hazards at a site. Conclusions are drawn considering the current site conditions and recommendations offered considering the current proposed development concept.

7.0 CONCLUSIONS AND RECOMMENDATIONS

General

Based on the information obtained during this geologic evaluation, we judge that there are no geologic conditions or geologic hazards that would preclude construction of the proposed residence and at the site as it is currently proposed. We should be notified in writing of any changes to the development concept so that we might review and, if necessary, to modify the conclusions and recommendations.

Landsliding

The site is in an area of very minimal topographic relief and no landslides have been mapped in the area. The area of the site is characterized on compilation and interpretve maps as having a low potential for landsliding. It is our opinion that the potential for landsliding or debris flows in any area that could affect the site is nil.

Primary Seismic Hazards

Although the Cypress Point Fault trends through the neighboorhood and just beyond the southwest property corner of the site, the recommended fault-building setback provides an adequate mitigation from ground displacement in a fault surface rupture event during the design life of the proposed reseidence. Fault surface rupture poses an equal level of hazard for the ground or main floor of the proposed residence as it does for the proposed basement (low). As with all sites throughout central coastal California, the geologic hazard that poses the greatest impact to the site is the potential for very strong to severe seismic shaking. The San Gregorio fault, the San Andreas fault zone or the Monterey Bay-Tularcitos fault system are likely to produce the highest level of seismic shaking at the site, although there are a number of active faults in the region that are capable of producing very strong levels of seismic shaking during the design life of the improvements. Selection of seismic design parameters should be made after careful consideration of the site profile, analytical procedures, and past performance of similar structures during magnitudes of shaking similar to those expected for the site. The planned residence should be designed to resist damage associated with very strong to severe ground shaking in accordance with current building codes and design standards. Refer to the geotechnical report for the project by Haro, Kasunich & Associates, Inc. (in progress) for the recommended seismc design criteria.

Secondary Seismic Hazards

The site is located in an area characterized on interpretaive maps as having a low potential for liquefacton. The relative consolidation of the subsurface sandy soils and the absence of a laterally extensive or continuous groundwater table indicate that there is a very low potential for liquefaction to occur at the site. For similar reasoning as that stated for liquefaction, there is a low potential for the occurrence of lateral spreading, or seismically induced settlement to occur.

Water Related Seismic Hazards

Due to the lack or stored or otherwise confined bodies of water in the area, the potential for the subject site to be affected by seiches is nil. Due to the geographic and topographic characteristics of the site, the potential for the site being inundated by a Tsunami is nil.

Drainage and Erosion Control

In general, control of surface runoff and appropriate design of drainage facilities are critical to the long term stability of the site slopes as well as provide protection from severe erosion. We recommend all surface runoff and any new runoff generated from the proposed construction (roofs, flatwork, etc.) should also be collected and directed to appropriate discharge facilities. Additionally the drainage coming off the street should be collected and or diverted awasy from the front yard area. Estimates of runoff quantities for the project should be provided by an engineer familiar with the site conditions.

8.0 LIMITATIONS

1. The conclusions of this report are based on data acquired and evaluated from this study and are intended to apply only to the development concept that is currently being proposed. The conclusions of this report are based upon the assumption that the site geologic and soil conditions do not deviate substantially from those disclosed in the research and our observations of a limited number of natural exposures at and immediately adjacent to the site. Although exploratory boring logs from previous consultants studies were reviewed as part of this work, we make no warrantee as to the accuracy of those those characterizations and they are merely referred to for background information. If any variations or unforeseen conditions are encountered during construction, or if the proposed construction will differ substantially from that planned at the present time, the geologic and consultant should be notified so that reevaluation of the conditions and supplemental recommendations can be given. In the event that I am not notified of such changes, the conclusions and recommendations presented in this report would be invalidated.

2. This report is issued with the understanding that it is the responsibility of the owner or the owner's representative to ensure that the information presented herein is called to the attention of the project architect and engineer.

3. The findings of this report are valid as of the present date. Changes in the conditions of a property can occur with the passage of time. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of the control of the consulting geologist and geotechnical engineer. Therefore, this report should not be relied upon after a period of one year without being reviewed by a qualified engineering geologist.

4. This report was prepared in general accordance with currently accepted standards of professional geologic practice in this area at this time. No warranty is intended, and none shall be inferred from the statements or opinions expressed.

5. All earthwork and associated construction should be observed by our field representative, and tested where necessary, to compare the generalized site conditions assumed in this report with those found at the site at the time of construction, and to verify that construction complies with the intent of our recommendations.

End of Text

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STEREO PAIR AERIAL PHOTOGRAPHS REVIEWED

Date	Scale	Type	Source	Flight I.D./Frames
8/24/1956	1:20,000	B&W	Aero Service Corp	ABG-4R-149, 146
5/15/1970	1:12,000	B&W	Calif Dept Fish and Game	76-471-170, 171
10/5/1976	1:12,000	Color	Calif Dept Fish and Game	DNOD-AFU-C-36, 37
1982	1:24,000	Infrared	unknown	AR574001673
9/28/1986	1:12,000	Nat. Color	unknown	CDBW-ADU-C-97, 98

CALIFORNIA COASTAL RECORDS PROJECT –OBLIQUE AERIAL PHOTOGRAPHY ALONG SCENIC DRIVE AT CYPRESS POINT FAULT EXPOSURE

Date	Scale	Type	Source	Flight I.D./Frames
1972	~800 ft	Natural Color	unknown	722098
8/12/2003	~200 ft	Natural Color	unknown	13920
8/12/2003	~200 ft	Natural Color	unknown	13920
10/11/2004	~400 ft	Natural Color	unknown	200402294
9/24/2010	~132 ft	Natural Color	unknown	201005275

APPENDIX A

Vicinity Map

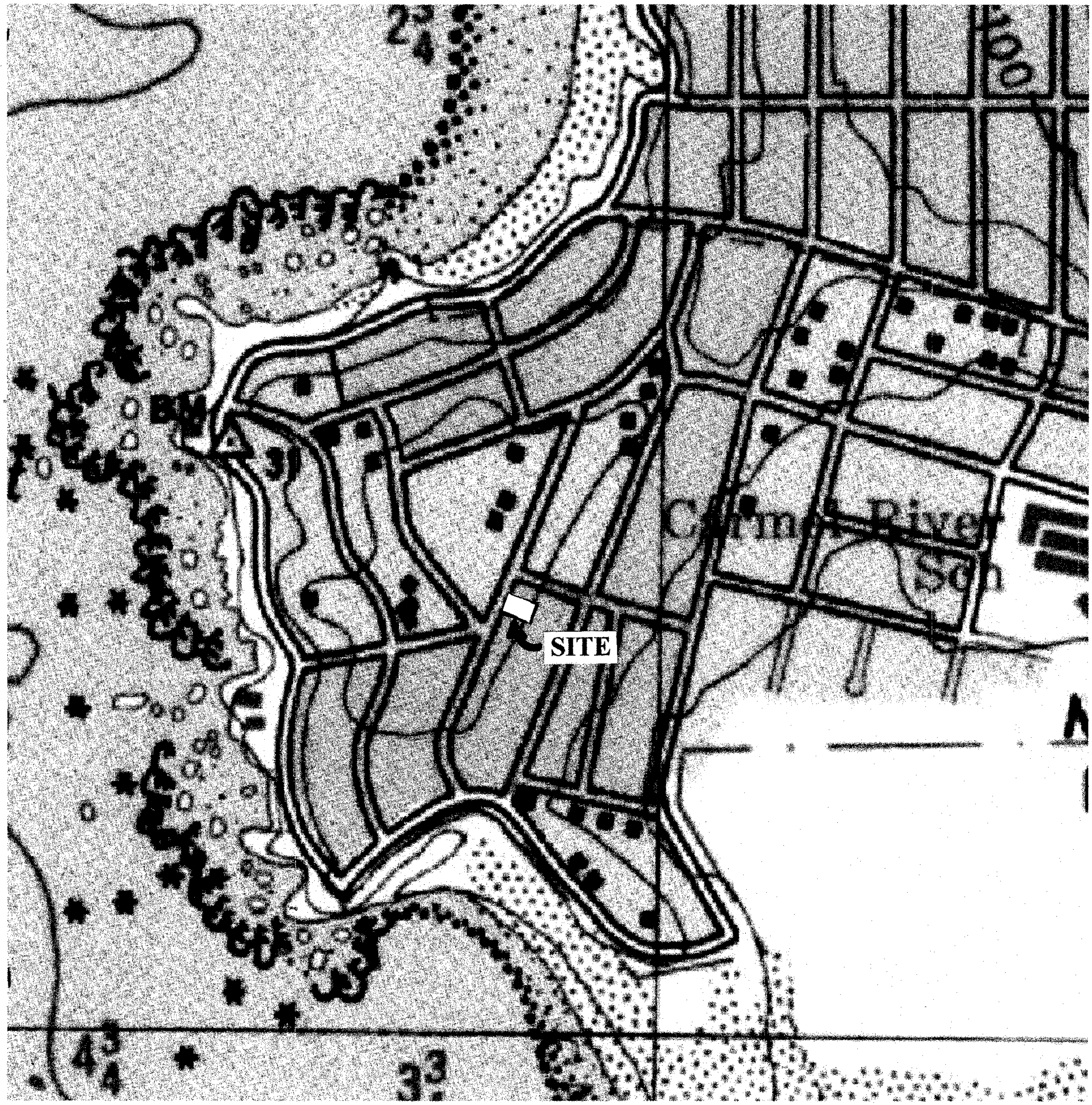
Regional Geologic Map

Regional Fault Map

Site Geologic Map

Log of CPF Fault Exposure at Scenic Road

VICINITY MAP



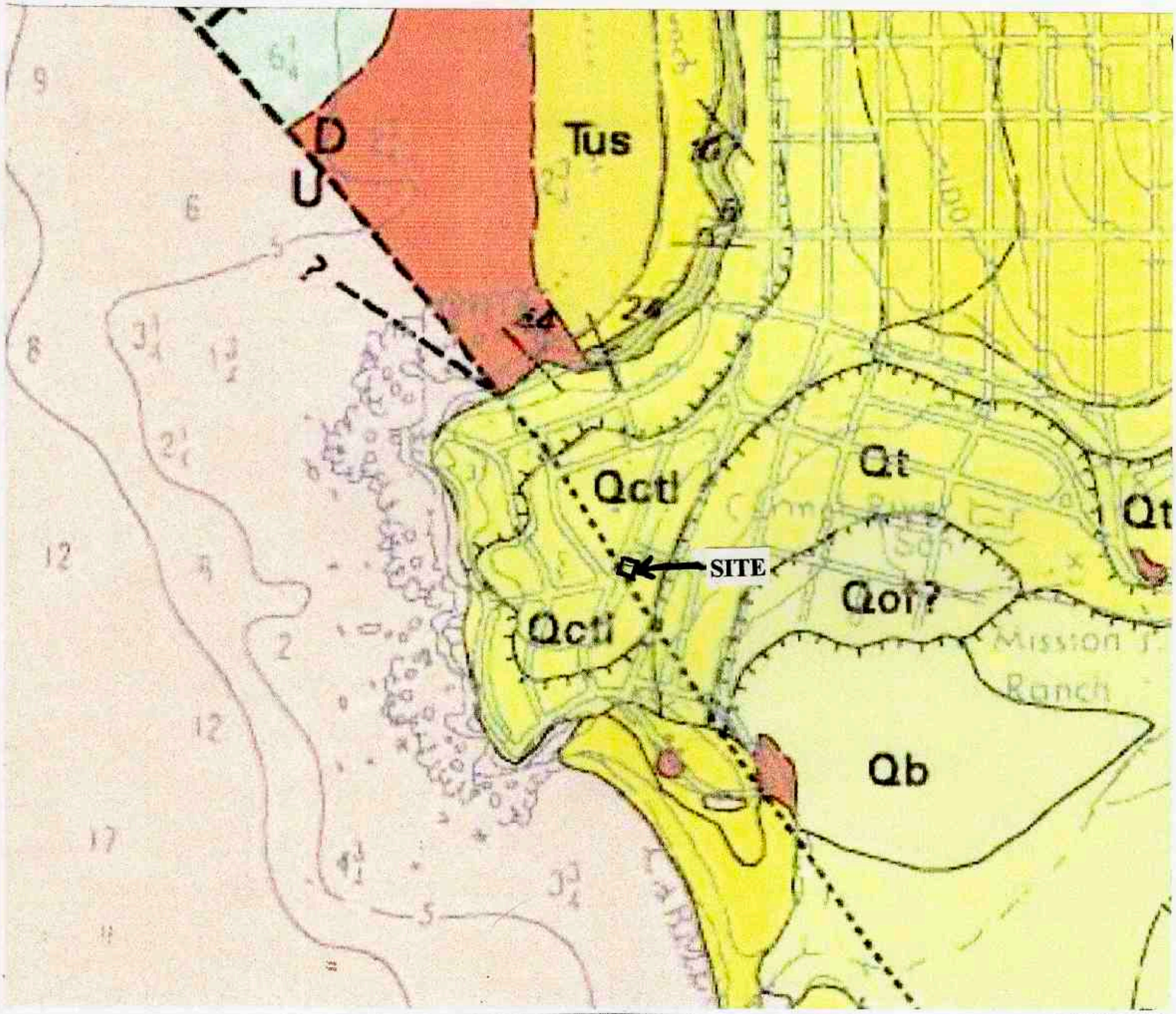
File No. G-790.3

Date: November, 2017

Craig S. Harwood, PG, CEG
Engineering Geologist

Proposed Residence at
26307 Isabella Avenue
Carmel-By-The-Sea, California

REGIONAL GEOLOGIC MAP



Explanation (Adapted from Clark et al. 1974)

Not to Scale

Geologic units

Qctl	Lighthouse coastal terrace (Pleistocene)
Qt	Coastal terrace (Pleistocene)
Tus	Unnamed sandstone (Miocene)
Tvb	Volcanic rocks (Oligocene)
dgp	Granodiorite bedrock (Cretaceous)

Symbols

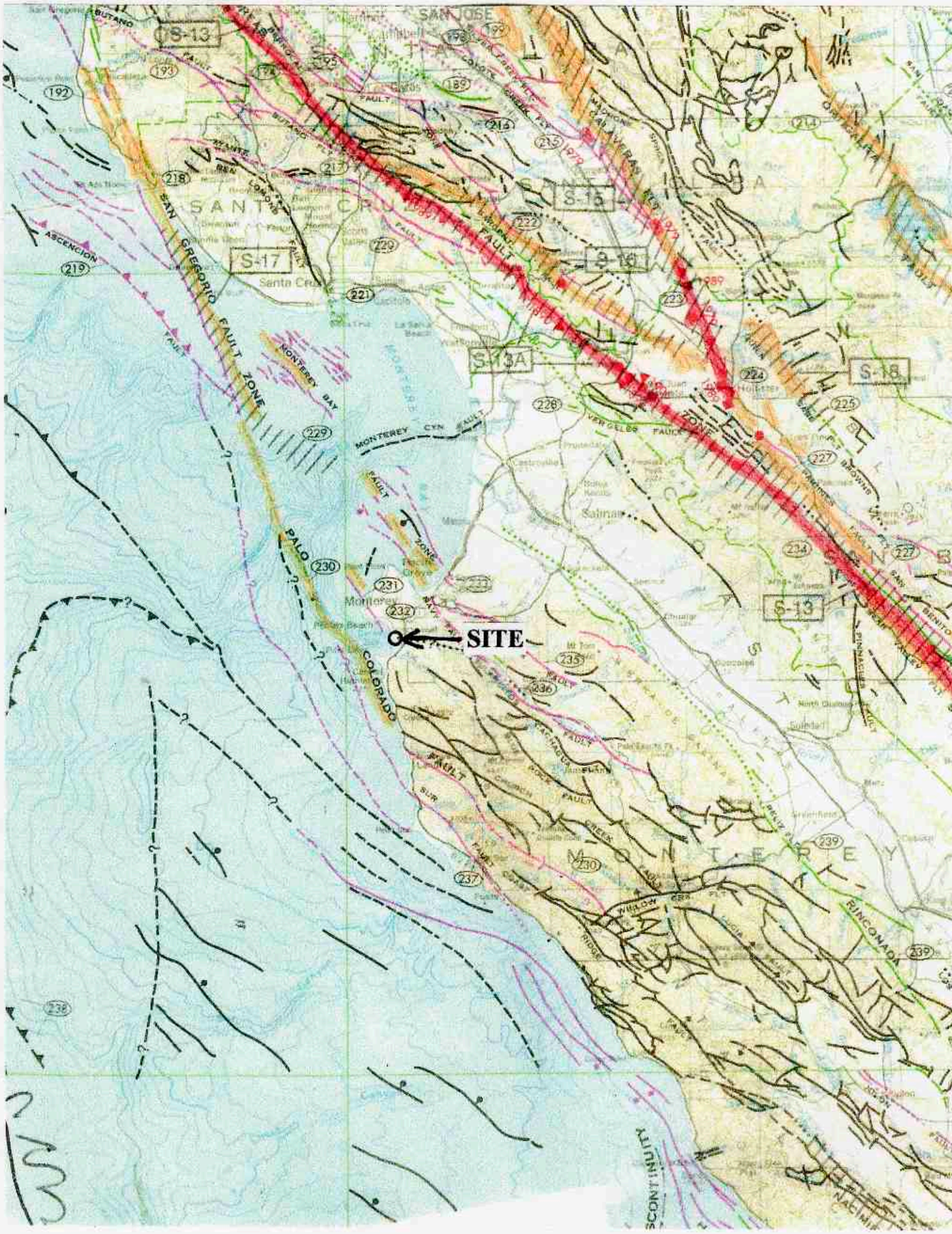
(symbol)	Bedding attitude
(symbol)	Geologic contact
● ● ● ●	Fault surface trace

File No. G-790.3

Date: November, 2017

Craig S. Harwood, PG, CEG
Engineering Geologist

Proposed Residence at
26307 Isabella Avenue
Carmel-By-The-Sea, California



Source: Jennings, 1994

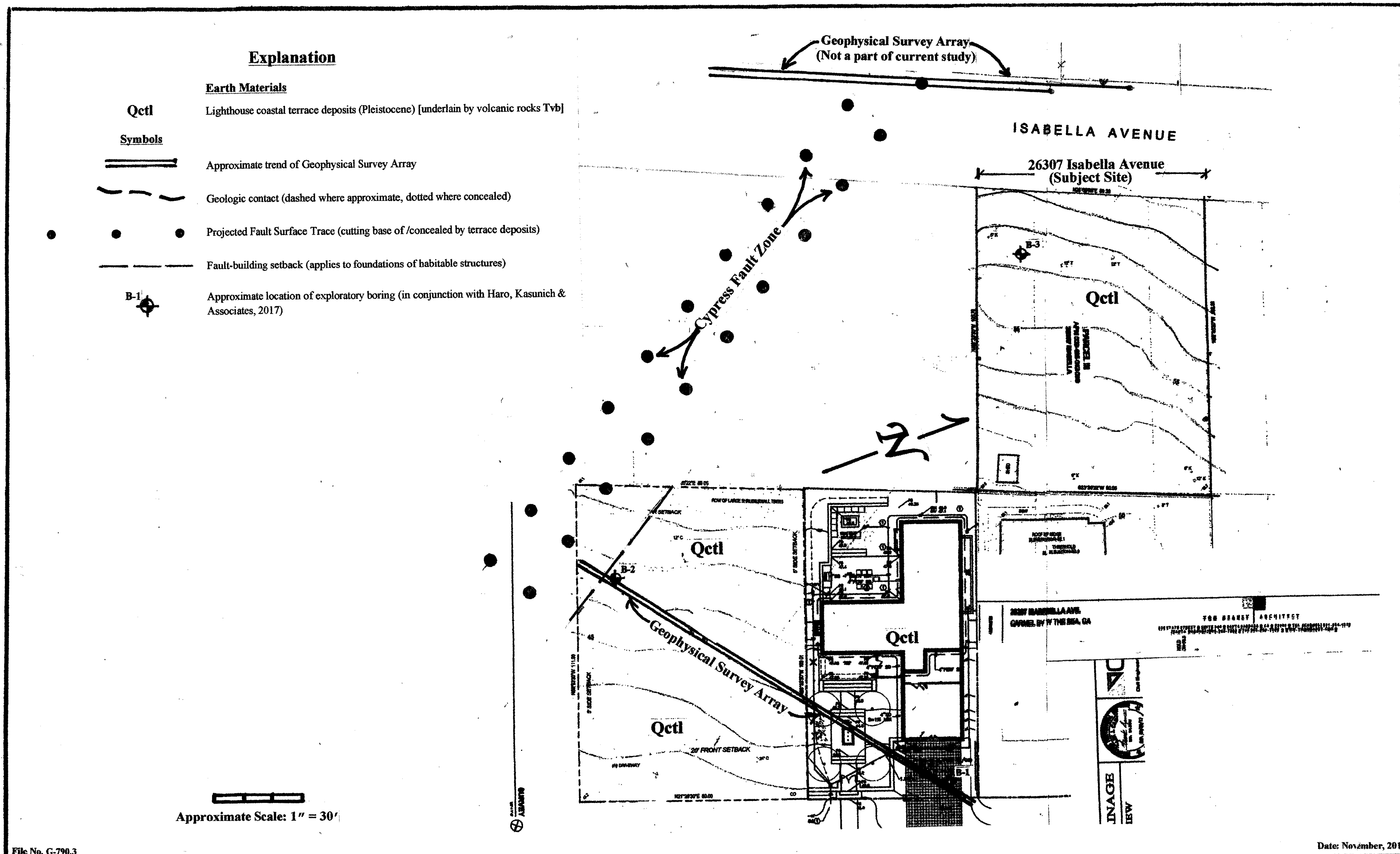
20 kilometers

File No. G-790.3

November, 2017

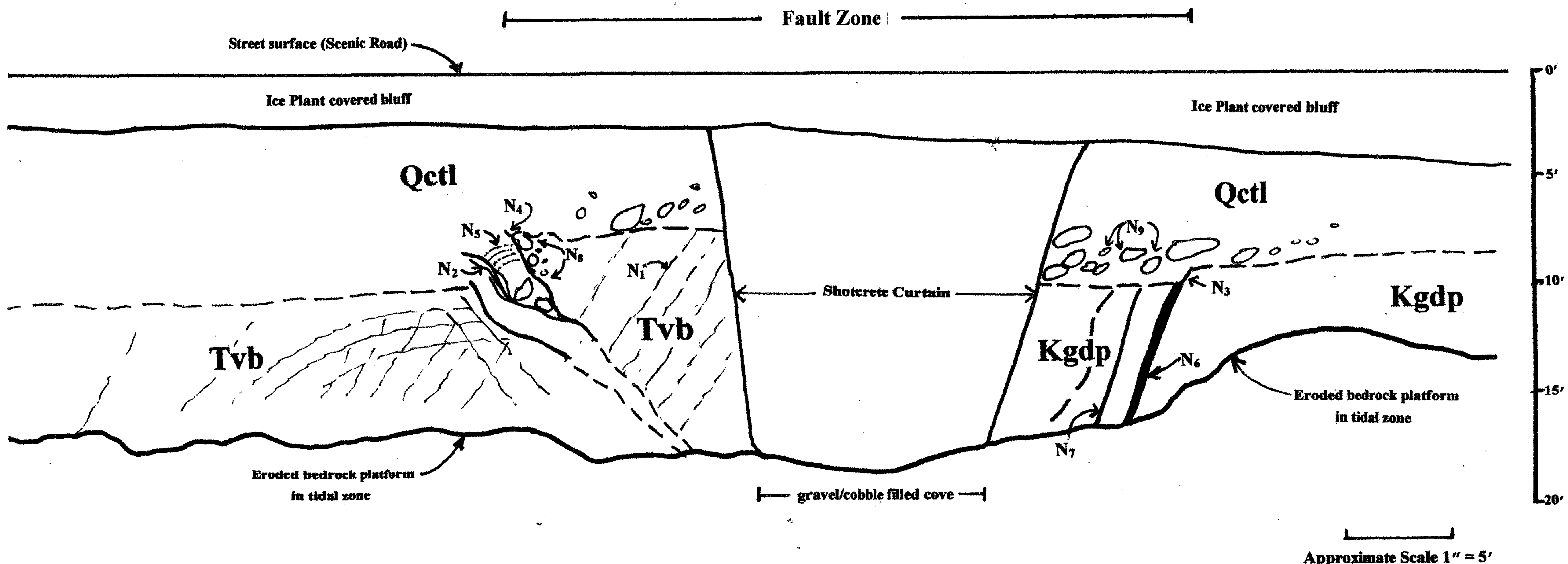
Craig S. Harwood, PG, CEG
Engineering Geologist

Proposed Residence at
26307 Isabella Avenue
Carmel-By-The-Sea, California



Proposed Residence at
26307 Isabella Avenue
Carmel-By-The-Sea, California

Log of Fault Exposure



Explanation

Qctl	Lighthouse Coastal Terrace (Pleistocene): silty sand with gravel and clay, cobbles concentrated within fault zone.
Kgdp	Granodiorite (Cretaceous): porphyritic granodiorite

Notes:

N_1: flow attitude N9°W/87°NE

N_2 shear surface N90°W/43°SW

N_3 apparent 3" down step (down on the east) on west side of fault zone

N_4 secondary shear

N_5 bedding within Qctl oriented N33°W/15°NE

N_6 planar surface within 1.5" thick sheared fault gouge: surface = N38°W/74°NE/plunge of striations

N_7 rare volcanic clast on west of fault zone

N_8 24 inches of apparent vertical offset within Qctl

File No. G-790.3

Date: November, 2017

Proposed Residence at
26307 Isabella Avenue
Carmel-by-the-Sea, California

APPENDIX B

Geophysical Report (JR Associates)

J R ASSOCIATES

Engineering Geophysics
17040 Oak Leaf Drive
Morgan Hill, CA 95037
(408) 293-7390

SEISMIC SURVEY AT 26332 AND 26338 VALLEY VIEW AVENUE CARMEL, CALIFORNIA

October 25, 2017

for

Emerson Development Group
3345 7th Avenue
Carmel, CA 93923

by

James Rezowalli, GP-921

PROFESSIONAL GEOPHYSICIST
JAMES REZOWALLI
GP-921
STATE OF CALIFORNIA

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

I INTRODUCTION

This report presents the results of a seismic investigation performed at 26332 and 26338 Valley View Avenue in Carmel, California (Drawing 1). The investigation was performed for Emerson Development Group by J R Associates. The purpose of the investigation was to look for geophysical evidence indicative of faulting beneath two adjoining residential lots. James Rezowalli, Principal Geophysicist, and Brian Rezowalli, Technician, of J R Associates performed the field work in October of 2017.

A. Site Conditions

The area of interest was two adjoining residential lots off Valley View Avenue (Drawing 2). At the time of this investigation the site was an empty dirt lot. A fault was mapped in a beach bluff approximately 1200 feet northwest of the site (Drawing 2). Craig Harwood, PG, CEG, provided us geologic information indicating the fault juxtaposes granodiorite (Kgpd) on the fault's southwest side against basaltic andesite (Tvb) on the fault's northeast side. The strike of the fault suggested it may cross into the area of interest. The purpose of this investigation was to look for geophysical evidence of the fault at the properties.

In addition to the seismic study Mr. Harwood drilled and logged two borings, B1 and B2, at the property. The borings were near each end of the seismic line (Drawing 2).

II METHODOLOGY

We used two different seismic techniques to look for geophysical evidence of faulting, seismic refraction and shear wave profiling. Seismic refraction uses compressional (P) waves that can refract off the top of the bedrock. We used the refraction technique to look for offsets in the top of the bedrock that could be caused by faulting. Because the top of the bedrock could be at about the same elevation on both sides of the fault we also performed shear (S) wave profiling. Shear wave profiles show changes in S-wave velocity with depth. We used it to look for changes in shear velocity that may either be caused by different S-wave velocities in formations on either side of the fault or a low S-wave velocity zone caused by shearing and gouge within the fault zone.

A. Field Procedures

We collected refraction data along two seismic lines (Drawing 2). A test line was placed on the bluff above the fault and straddled the fault. The test line was collected to look at geophysical properties of the formations on either side of the fault. The second line crossed the property on Valley View Avenue at a right angle to the possible fault trace. The test line was 200 feet long and the line on the site was 160 feet long. The refraction survey contained geophones on ten-foot centers and multiple shot points. The shot points were at the beginning, the end, and along the lines. A twelve-pound sledge hammer striking an aluminum plate was used to create P-waves at the shot point locations.

An array consisting of a shot point placed 30 feet away from a string of 24 geophones was used to collect data for the shear wave profiling. The geophones were spaced 2.5 feet apart. A measurement of shear wave velocity with depth was calculated using the multichannel analysis of surface waves (MASW) technique developed by the University of Kansas at Lawrence. For the test line on the bluff we collected two S-wave Vs depth measurements over the granodiorite to the southwest of the fault, one measurement directly over the fault zone, and two measurements over the basaltic andesite to the northeast of the fault. On the Valley View Avenue properties S-wave Vs depth measurements were collected at ten-foot intervals along the seismic line. The S-wave velocity Vs depth measurements were concatenated together to create a two dimensional S-wave profile across the properties.

.

B. Data Reduction

Seismic refraction data reduction began by picking the arrival times from the seismograph recordings. An arrival time is the time a P-wave spent traveling from shot point to geophone. The wave could either travel along the ground surface or be refracted from an interface between materials. For a refraction to occur, the materials below the interface must have a greater P-wave velocity than the materials above the interface. The arrival times were entered into a computer program with elevation, location, and layer control information.

The interpretation program, FSIP, performs a first approximation delineation of the refracting horizons using a delay-time method. The approximation is then tested and improved by the program's ray-tracing procedure in which ray travel times computed for the model are compared against measured travel times. The model is subsequently adjusted iteratively to minimize the discrepancy between the computed and measured travel times. A Bureau of Mines Report of Investigation describes the program[1].

---

[1] Scott, James H., Computer Analysis of Seismic Refraction Data, BuMines RI 7595, 1972.

The program Surfseis developed by the Kansas Geological Survey was used to process the seismic records into S-wave profiles. From each seismic recording a fundamental-mode dispersion curve was extracted. The dispersion curve is related to the shear wave velocities of the different wave lengths contained in the surface wave. Longer wave lengths are related to the S-wave velocity of deeper soils and shorter wave lengths are related to the S-wave velocities of near surface soils. The dispersion curves are inverted into a series of one-dimensional S-wave velocity profiles. More information of the MASW can be found at the Kansas Geological Survey's web site at www.kgs.edu/software/surfseis/.

III RESULTS

A. Refraction Results

The results of the computer analysis of the refraction data are presented in Drawing 3 and Table 1. The drawing contains two-dimensional diagrams profiling the seismic layering and layer velocities measured along the refraction lines. Table 1 summarizes the results presented in the drawing.

Table 1. Summary of Refraction Results

Line	Depth to Layer 2 (feet)	Layer 1 Velocity (fps)	Layer 2 Velocity (fps)
Test	8 to 14	1200 to 1400	6300 to 6900
Property	34 to 37	1600	8200

We found two different seismic layers beneath the refraction lines. The layers were distinguished by their compressional (P) wave velocities. Layer 1 included the ground surface and had a P-wave velocity ranging between 1200 and 1600 feet per second (fps). The geologic logs from the two borings on the property indicate the first seismic layer consists of sands with some gravel.

The second seismic layer was distinguished by a P-wave velocity that ranged from 6300 to 6900 fps at the bluff and was 8200 fps at the properties. There was a small change in P-wave velocity across the fault and the data suggest that at a given location the granodiorite may have a slightly higher P-wave velocity than the basaltic andesite. The higher bedrock P-wave velocity measured at the property suggest the bedrock is more competent and less weathered there than at the bluff. The depth to the top of the second seismic layer ranged from 8 to 14 feet at the bluff and 34 to 37 feet at the properties. There was a slight change in elevation across the fault with the top of the granodiorite being on average a foot or two higher than the top of the basaltic andesite. This is probably more due to weathering than movement caused by the fault. The refraction horizon was relatively flat at the properties and we saw no significant elevation change that would suggest a fault there. The geologic logs indicated the second seismic layer beneath the property consisted of basaltic andesite. The basalt was found at both ends of the seismic line.

B. Shear Wave Results

The results of the MASW shear wave profiling are presented in Drawing 4. At the bluff the shear wave velocities ranged from 1200 fps to 4200 fps. (We believe the S-wave velocities greater than 4200 fps were cause by low frequency noise coming from the nearby breaking waves.) There was a distinct change in the S-wave velocity across the fault with the granodiorite having velocities around 4200 fps and the basaltic andesite around 2200 fps. The S-wave profile collected across the property only changed with depth. There was no lateral change in S-wave velocity that would suggest a fault there.

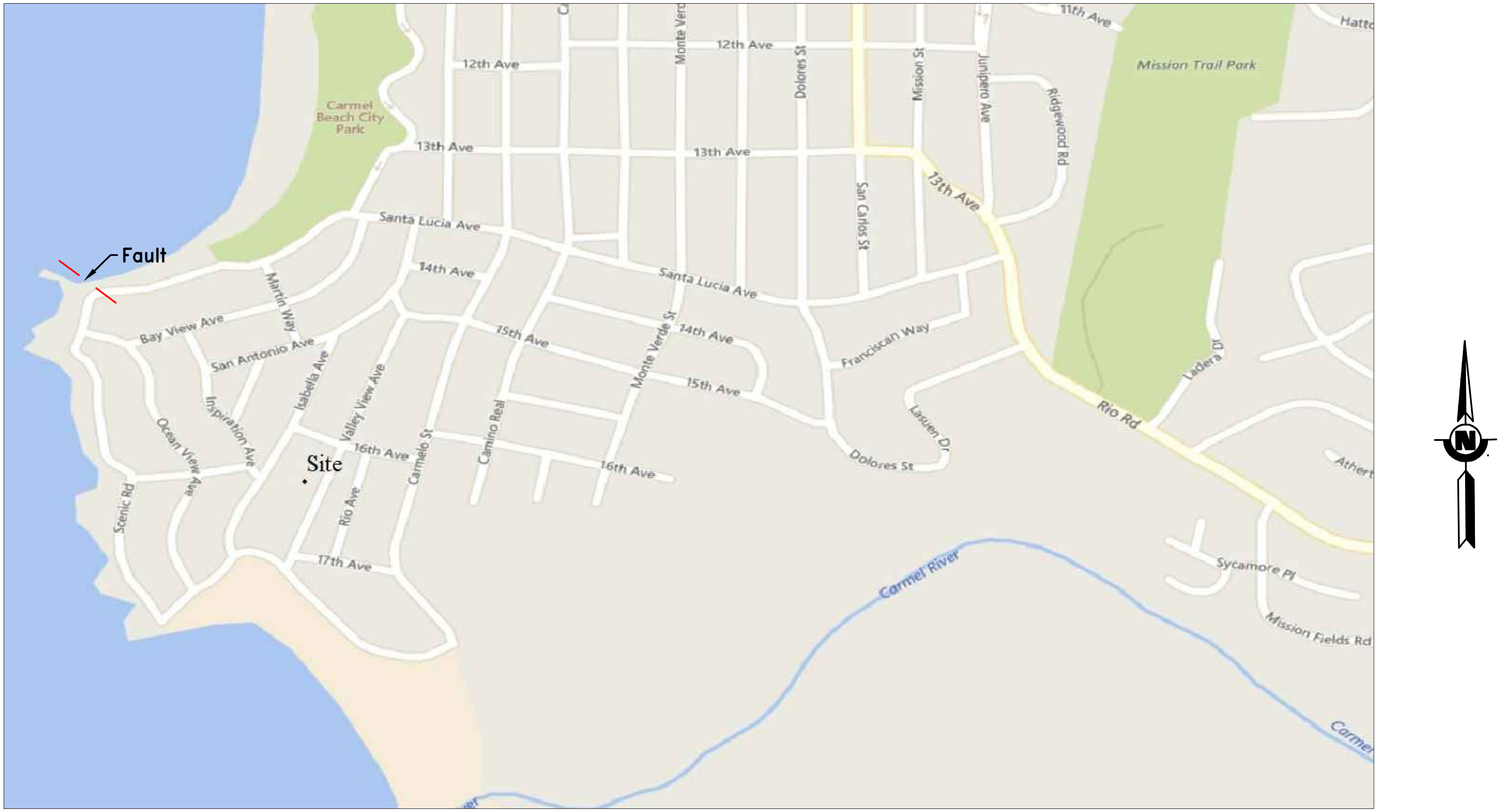
C. Summary

We found no geophysical evidence for a fault beneath the properties. The top of the bedrock at the properties was relatively flat in both the refraction and shear wave profiles showing no vertical offsets that might be caused by a fault. The S-wave velocity across the properties only changed with depth which would be expected as the materials beneath the site went from unconsolidated soil to competent rock. There were no lateral changes in velocity at the properties that would suggest a fault has juxtaposed materials with different seismic velocities.

C. Limitations

Seismic layers do not always correspond directly to lithologic changes that might be found in borehole or trenching data. A seismic layer is an interface between materials with different P- or S-wave velocities. Factors such as weathering, cementation, induration, and saturation as well as lithologic changes can create changes in seismic velocities. Also, there can be lithologic changes without velocity changes. However, our field experience indicates that seismic layers often correspond to major changes in lithology or saturation to within ±20% of the depth to the interface. In order to detect a fault there must be a change in the physical rock properties across the fault. If there is no change in physical properties across a fault then it will not be detected. Our data should be compared with available geologic and other data before conclusions are drawn.

IV DRAWINGS



Vicinity Map
Valley Vie□ Avenue
Carmel, California

SCALE: No Scale		DRAWN BY: J.J.R.	
DATE: 10-16-2017	JOB NUMBER: 127-010-17	REVISED:	

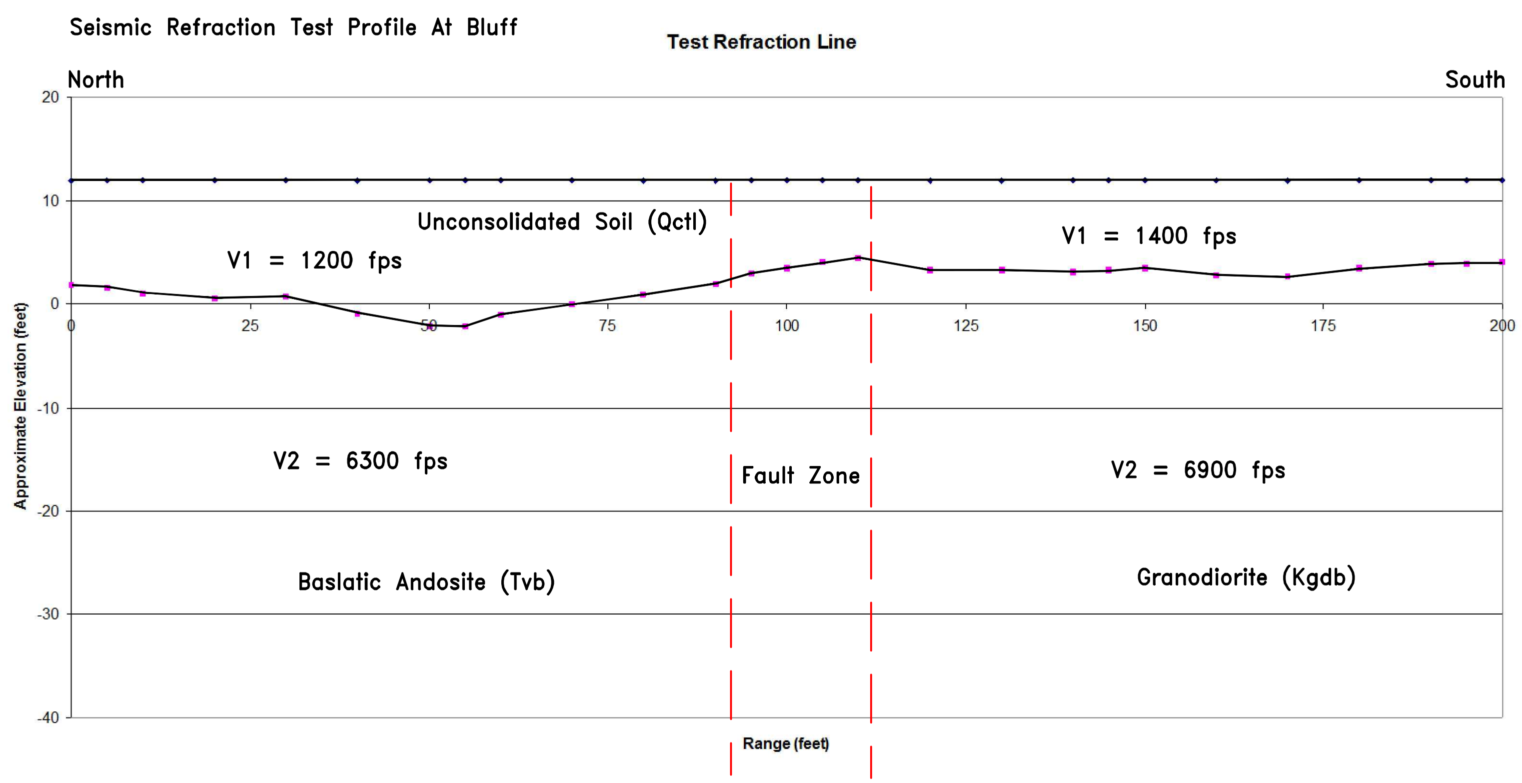
J R Associates Civil and Environmental Geophysics
17040 Oa□Leaf Drive Mor□an Hill, CA (408) 293-7390

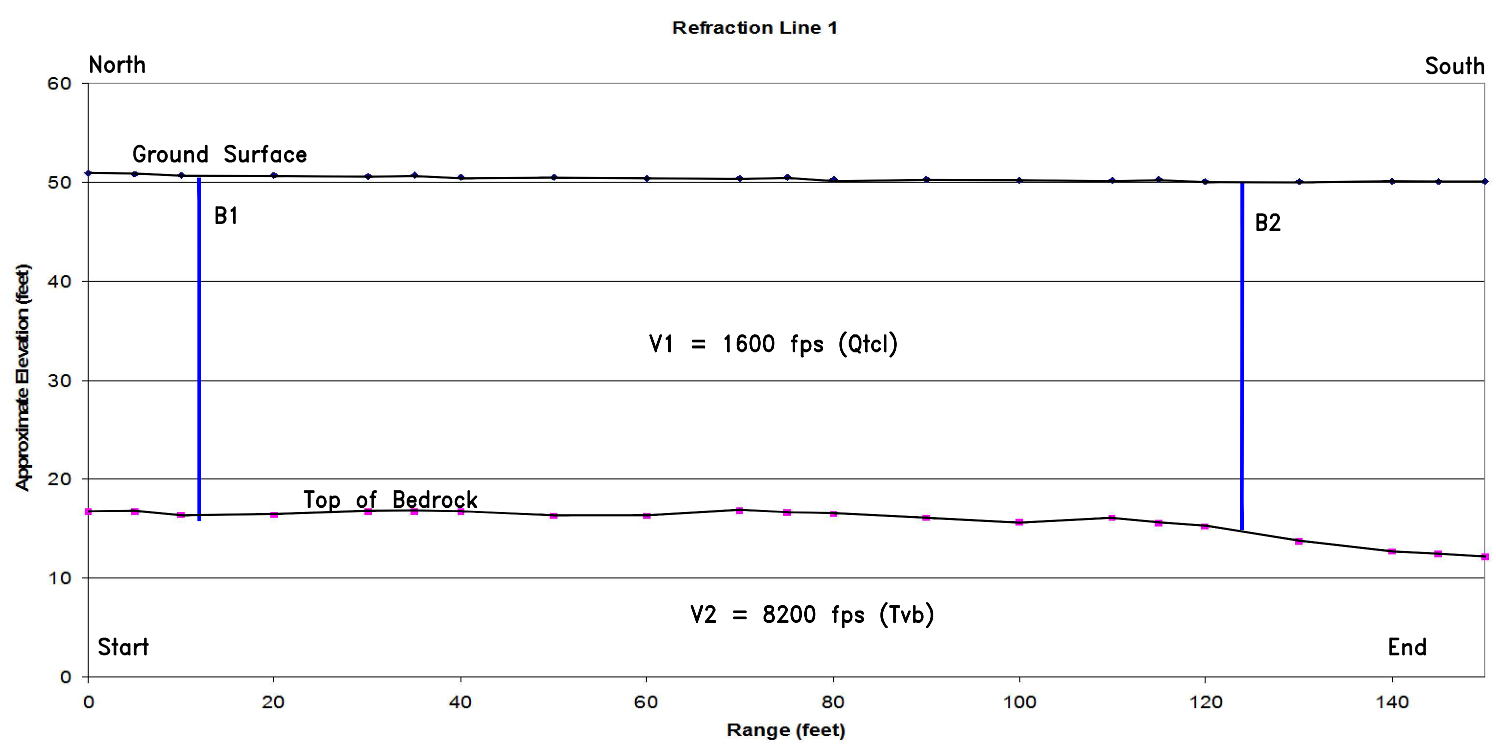
DRAWING NUMBER: 1 of 4

Site Map Valley Vie□ Drive Carmel, California			
SCALE: No Scale			DRAWN BY: J.J.R.
DATE: 10-13-2017	JOB NUMBER: 170-291-17		REVISED:

J R Associates Civil and Environmental Geophysics
17040 Oa□Leaf Drive Mor□an Hill, CA (408) 293-7390

DRAWING NUMBER: 3 of 4



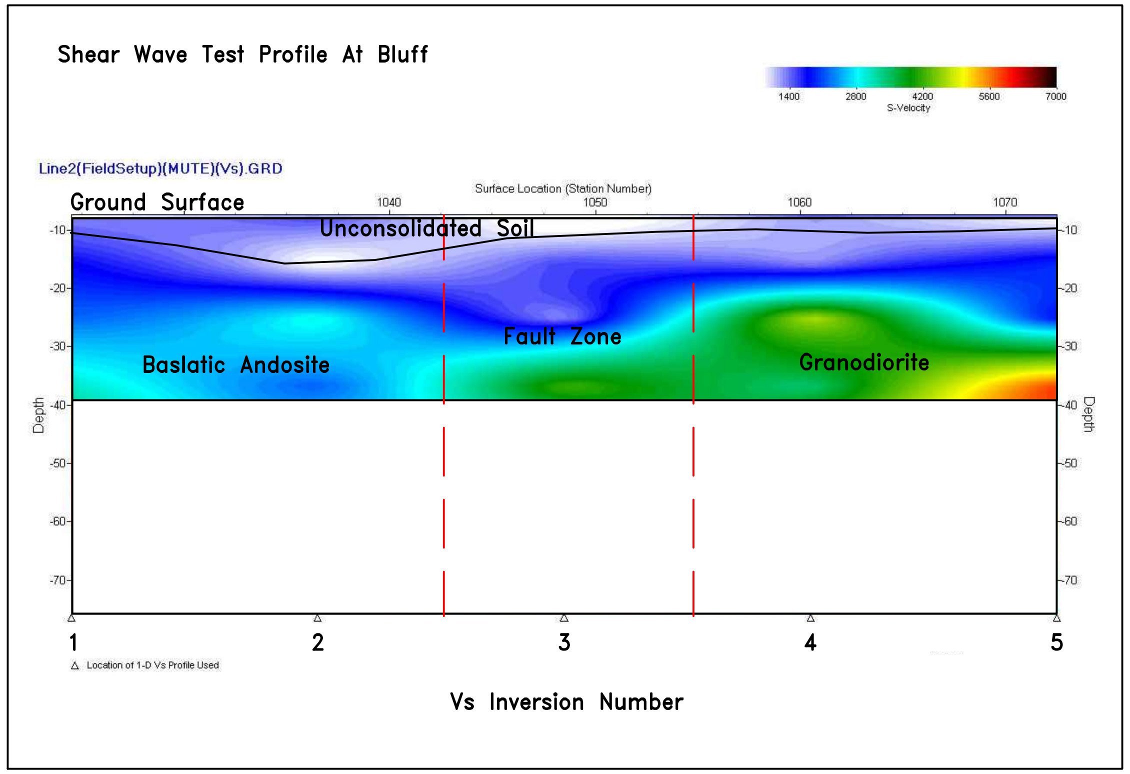


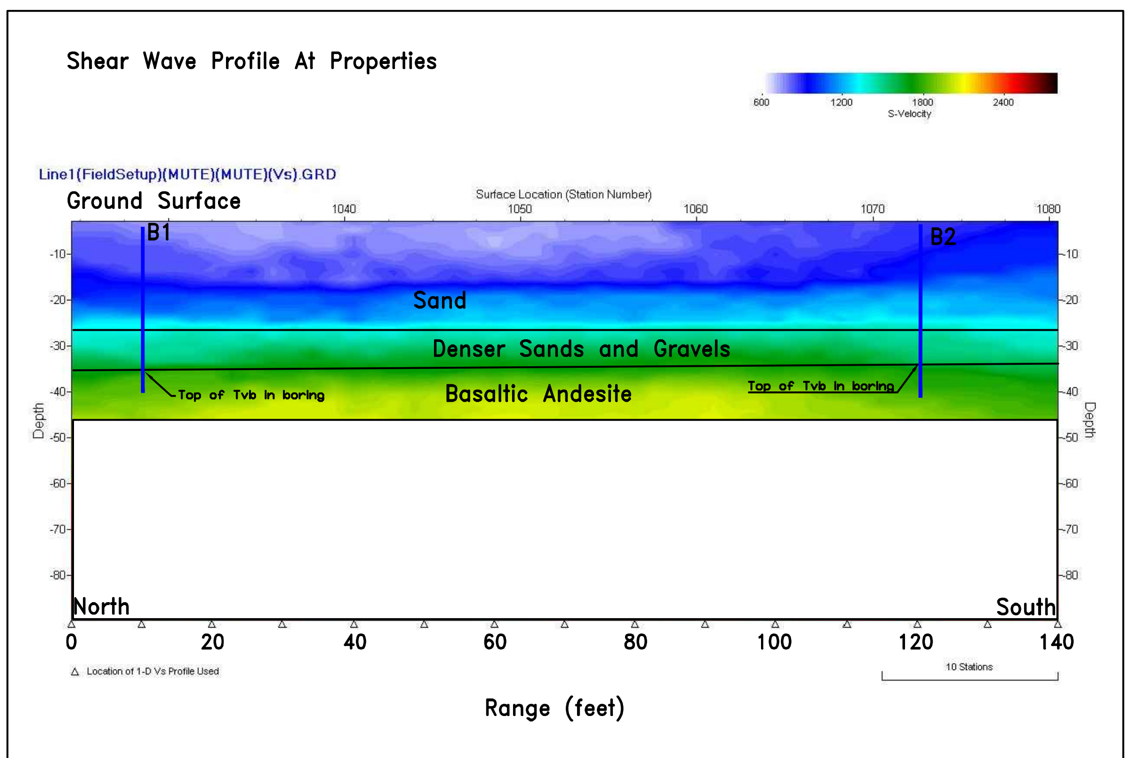
Seismic Refraction Profiles
Valley Vie□ Drive
Carmel, California

SCALE: See Dia□rams		DRAWN BY: J.J.R.	
DATE: 10-13-2017	JOB NUMBER: 170-291-17	REVISED:	

J R Associates Civil and Environmental Geophysics
17040 Oa□Leaf Drive Mor□an Hill, CA (408) 293-7390

DRAWING NUMBER: 3 of 4





Shear Wave Profiles
Valley Vie□ Drive
Carmel, California

SCALE: See Dia□rams			DRAWN BY: J.J.R.
DATE: 10-13-2017	JOB NUMBER:	170-291-17	REVISED:

J R Associates Civil and Environmental Geophysics
17040 Oa□Leaf Drive Mor□an Hill, CA (408) 293-7390

DRAWING NUMBER: 4 of 4

APPENDIX C

Logs of Exploratory Borings

No. B-1

PROJECT	**26338 Valley View Avenue**	DATE	10/9/17	LOGGED BY	CSH
DRILL RIG	Mobile B-53	HOLE DIA.	6"	SAMPLE=	MC - California Modified, S - SPT, C - California 2.5"
GROUND WATER DEPTH INITIAL	N/A	FINAL	NE	HOLE ELEVATION	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	% Recovery
silty fine SAND: very dark brown, dry, loose Residual soil	SM	1										
		2										
Poorly graded SAND: medium brown, damp, medium den [Eolian dune sand facies of Qctl]		3										
	SP	4										
		5										
		6										
Poorly graded SAND: lt yellow brown, damp, medium dense fine grained [Fluvial facies of Qctl]		7										
		8										
		9										
	SP	10										
		11										
		12										
		13										
		14										
		15										
Poorly graded SAND: medium yellow brown, damp, dense, medium grained, micaceous		16										
	SP	17										
		18										
		19										
log continued on page 2 of 2		20										

				No.	**B-1**
PROJECT	**26338 Valley View Ave**	DATE	10/9/17	LOGGED BY	CSH
DRILL RIG	Mobile B-53	HOLE DIA.	6"	SAMPLE=	MC - California Modified, S - SPT, C - California 2.5"
GROUND WATER DEPTH INITIAL	N/A	FINAL	NE	HOLE ELEVATION	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	% Recovery
Poorly graded SAND: medium yellow brown, damp,												
dense, medium grained, micaceous		21										
[Fluvial facies of Qctl]	SP											
		22										
		23										
		24										
		2										
		26										
Well Graded GRAVEL w/sand and cobbles: Med yel-		27										
brown, very dense	GW											
		28										
Poorly graded SAND: Lt yellow brown, very dense, damp												
		29										
	SP	30										
		31										
Well Graded GRAVEL w/sand: med yellow-												
brown, very dense	GW	32										
grindng/moderatly difficult advance rete												
		33										
		34	S	3								
			S	8								
andesite clasts in sampler shoe		35	S	19								
medium dense												
Basaltic Andesite: very dark gray brown to dark		36										
reddish brown, damp, moderately strong (ASTM =												
ery dense to hard), very severely weathered		37										
drilled out slough and straight drilled between 36		38										
and 40 ft. Steady moderately difficult drilling												
		39										
see page 3 of 3 for continuaton of log		40										

						No.	B-1
PROJECT	**26338 Valley View Avenue**			DATE	10/9/17	LOGGED BY	CSH
DRILL RIG	Mobile B-53	HOLE DIA.	6"	SAMPLE=	MC - California Modified, S - SPT, C - California 2.5"		
GROUND WATER DEPTH INITIAL	N/A	FINAL	NE		HOLE ELEVATION		

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	% Recovery
Basaltic Andesite: very dark brown, damp,			MC	25								
moderately strong, very severely weathered,		41	MC	50-6"								90
Basalt: black, damp, strog, mderately severely weathered			C	26								
		42	C	50-2"								90
			S	50-5"								90
		43										
Bottom of boring at 42.25 feet												
		44										
No groundwater encountered												
		45										
		46										
		47										
		48										
		49										
		50										
		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

	No.	B-2
PROJECT **26338 Valley View Avenue**	DATE 10/9/17	LOGGED BY CSH
DRILL RIG Mobile B-53	HOLE DIA. 6"	SAMPLE= MC - California Modified, S - SPT, C - California 2.5"
GROUND WATER DEPTH INITIAL N/A	FINAL NE	HOLE ELEVATION

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	% Recovery
silty fine SAND: very dark brown, dry, loose												
Residual soil	SM	1										
			MC	3								
		2	MC	6								
			MC	10								70
Poorly graded SAND: medium brown, damp, medium den		3	S	1								
[Eolian dune sand facies of Qctl]			S	2								
	SP	4	S	5								70
		5										
		6										
		7										
		8										
		9										
Poorly graded SAND: lt yellow brown, damp, medium dense												
fine grained		10										
[Fluvial facies of Qctl]			MC	10								
		11	MC	12								
			MC	14								
		12	S	4								
			S	8								
		13	S	10								
		14										
		15										
Poorly graded SAND: medium yellow brown, damp,												
dense, medium grained, micaceous		16										
	SP	17										
		18										
		19										
log continued on page 2 of 3		20										

		No.	**B-2**
PROJECT **26346 Valley View Ave**	DATE 10/9/17	LOGGED BY	CSH
DRILL RIG Mobile B-53	HOLE DIA. 6"	SAMPLE=	MC - California Modified, S - SPT, C - California 2.5"
GROUND WATER DEPTH INITIAL N/A	FINAL NE	HOLE ELEVATION	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	% Recovery
Poorly graded SAND: medium yellow brown, damp,												
dense, medium grained, micaceous		21										
[Fluvial facies of Qctl]	SP											
		22										
		23										
		24										
		2										
		26										
		27										
		28										
Poorly graded SAND: Lt yellow brown, very dense, damp												
		29										
	SP	30										
		31										
		32										
Lean CLAY in spoils	CL	33	MC	2								
brown, very dense, grinding			MC	8								
Basaltic Andesite: med gray brown, damp,		34	MC	50-6"								
disintegrated, grinding throughout the interval			C	50-1"								
drilled out between 34.5 feet to 40 feet		35	S	50-4"								
		36										
		37										
		38										
Basaltic Andesite: very dark gray brown to dark												
reddish brown, damp, weak (severely weathered)		39										
slough in inititial 12" of sampling interval												
see page 3 of 3 for continuaton of log		40	MC	1								

				No.	**B-2**
PROJECT	**26338 Valley View Avenue**	DATE	10/9/17	LOGGED BY	CSH
DRILL RIG	Mobile B-53	HOLE DIA.	6"	SAMPLE=	MC - California Modified, S - SPT, C - California 2.5"
GROUND WATER DEPTH INITIAL	N/A	FINAL	NE	HOLE ELEVATION	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	% Recovery
Basaltic Andesite: very dark brown, damp,			MC	2								
moderately strong, very severely weathered,		41	MC	2								90
(slough in uinitial 12" of sampling interval)			C	12								
		42	C	16								
			C	30								90
		43	S	50-6"								90
Bottom of Boring at 42.5 feet		44										
No groundwater encountered		45										
		46										
		47										
		48										
		49										
		50										
		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

							No.	**B-3**
PROJECT	**26307 Isabella Avenue**				DATE	10/9/17	LOGGED BY	CSH
DRILL RIG	Mobile B-53	HOLE DIA.	6"	SAMPLE=		MC - California Modified, S - SPT, C - California 2.5"		
GROUND WATER DEPTH INITIAL		29.25	FINAL	29		HOLE ELEVATION		

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	% Recovery
[straight drilled to 30 feet]												
Poorly graded SAND: medium yellow brown, damp,		21										
dense, medium grained, micaceous	SP											
[Fluvial facies of Qctl]		22										
		23										
		24										
		25										
		26										
		27										
Basaltic Andesite: very dark brown, damp,		28										
moderately strong, decomposed to very severely												
weathered		29										
		30										
			MC	50-6"								
		31	MC	50-6"								
			MC	50-.5"								
		32										
Bopttom of Boring at 31.2 feet												
		33										
Groundwater encountered at 29.25 feet												
		34										
		35										
		36										
		37										
		38										
		39										
		40										

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