1850

Staff Report to the Zoning Administrator

Application Number: 181024

Applicant: Matson Britton Architects
Owner: Jim and Sue Vaudagna
APN: 043-095-14
Site Address: 379 Beach Drive, Aptos

Agenda Date: April 17, 2020
Agenda Item #: 6
Time: After 9:00 a.m.

Project Description: The proposed project includes demolition of an existing three-story single-family dwelling and construction of a new three-story replacement single-family dwelling. The project includes excavation of the toe of the coastal bluff, construction of several coastal bluff stabilization features including two retaining walls (9 and 11-feet high respectively) in the rear yard, as well as a 10-foot high geobrugg debris flow fence located 30-40 feet upslope from the proposed home. Requires a Coastal Development Permit and Variance approval to reduce the minimum 20 foot front yard setback to 10 feet, to increase the maximum height of 28 feet to 32.5 feet, to increase the maximum number of stories from two to three and to increase the allowed 50% floor area ratio to approximately 60%.

Location: Property located on the northeast side of Beach Drive approximately ½ mile south of the intersection with Rio Del Mar Boulevard (379 Beach Drive).

Permits Required: Coastal Development Permit, Variances

Supervisorial District: Second District (District Supervisor: Zach Friend)

Staff Recommendation:

- Determine that the proposed action is exempt from further Environmental Review under the California Environmental Quality Act. CEQA Section 15270 states that "CEQA does not apply to projects which a public agency rejects or disapproves.
- Denial of Application 181024, based on the attached findings.

Project Description & Setting

The subject parcel is approximately 6,000 square feet in size and zoned Single Family Residential (minimum parcel size 6,000 square feet), (R-1-6) which is consistent with the Land Use Designation of Urban Low Residential Density (R-UL). The property is developed with an existing three-story single-family dwelling with a two-car garage at the lower level.

The project site is situated within a long stretch of homes sandwiched between the base of a coastal bluff and Beach Drive. The east side of Beach Drive contains a sidewalk and public beach resulting in unimpeded views of the Monterey Bay. The majority of the homes along this stretch of Beach Drive are two and three stories in height and consist of a variety of architectural styles. New homes constructed along this stretch within the past 10 years have been designed as "bunker style" homes, with retaining walls at the base of the bluff comprising the rear wall of the structures which are engineered to withstand a landslide event from the bluff above.

This is a proposal to demolish the existing home and construct a new home. The project includes a proposal to grade approximately 119 cubic yards of material at the base of the coastal bluff for the construction of two concrete retaining walls, 9 and 11-feet tall, along with a 10-foot tall geobrugg debris fence approximately 30-40 feet upslope from the rear wall of the proposed residence. Geotechnical (Soils) and Geologic Reports have been prepared and submitted to the County for review by the County Geologist and County Civil (Geotechnical) Engineer under application REV181023. County staff have not accepted the Geotechnical (Soils) and Geologic Reports for the reasons outlined in the attached review letters (Exhibits H, L and P).

Exhibit F – *Geotechnical (Soils) Report* prepared by Pacific Crest Engineering dated November 30, 2017
Exhibit G - *Geologic Report* prepared by Zinn Geology dated 11 February 2018
Exhibit H - *County of Santa Cruz Report Review letter* dated 26 March 2018
Exhibit I – *Response to Review of Geotechnical Investigation and Supplemental Analysis* prepared by Pacific Crest Engineering dated August 16, 2018
Exhibit J – *Response to County of Santa Cruz comments* prepared by Zinn Geology dated August 16, 2018
Exhibit K - *County of Santa Cruz Incomplete Letter* dated November 7, 2018
Exhibit L - *County of Santa Cruz Clarification of Technical Issues for 379 Beach Drive* letter dated 18 July 2019
Exhibit M – *Response to "Clarification of Technical Issues for 379 Beach Drive"* prepared by Pacific Crest Engineering dated August 31, 2019
Exhibit N - *Response to 18 July 2019 County of Santa Cruz Comments* prepared by Zinn Geology dated 23 August 2018
Exhibit O – *Email string between Anna DiBenedetto, Rick Parks and Carolyn Burke* dated September 11, 2019 thru October 8, 2019
Exhibit P - *County of Santa Cruz Response Review letter* dated 7 October 2019

Geologic Hazards

The parcel is located within a coastal hazard area, subject to physical hazards as a result of coastal processes including landsliding, coastal bluff erosion, and inundation and erosion by wave action. Landslides in the coastal bluff are typically fluid debris flows and occur within the marine terrace deposits at the top of the bluff, and the weathered and fractured "rind" forming within the bedrock in the face of the bluff (Zinn 2018). Although, as stated in the 19 July 2019 County Review letter (Exhibit L), there is evidence that moderate to large scale landsliding in excess of 15-feet deep, originating at or near the crest of the coastal bluff, of either translational or rotational mechanism are possible at the project site.

The parcel is located within a mapped Federal Emergency Management Agency (FEMA) coastal flood hazard zone. FEMA has established an elevation of the "100-year coastal flood" as 21 feet NAVD. The proposed home will be subject to high velocity wave run-up and impacts from coastal flooding. In addition to flooding, the beach sand underlying the proposed home may be completely scoured down to the bedrock platform (roughly 1.5 feet NAVD), and as a result it is estimated that the upper 16 feet of the foundation system will lose all vertical and lateral support (Pacific Crest, 2017) in the event of severe wave inundation.

The parcel also lies within an area mapped as a high potential for liquefaction. Both the project geologist and geotechnical engineer conclude that liquefaction and lateral spreading may occur during the lifetime of the proposed residence.

The project as proposed incorporates mitigations to address potential impacts from shallow slope failures, coastal flooding and wave run-up via elevation of the habitable portions of the building and incorporation of break-away walls into the design of the ground-level garage. These design features allow for the material generated by shallow "debris flow" type bluff failures to flow through the rear yard and come to rest beneath the residence; similarly, the break-away walls would allow for entry and exit of coastal flood waters beneath the residence while limiting the redirection of waves toward adjacent properties. The proposed deep foundation design was recommended by the applicant's geologic and geotechnical consultants to address the potential impacts related to liquefiable soils beneath the structure and scour due to coastal flooding.

Presently, the potential for large scale landsliding has not been adequately considered in the geotechnical and geologic investigations submitted for review. As noted above, the potential for moderate to large scale landsliding in excess of 15-feet deep, originating at or near the crest of the coastal bluff may occur at the project site. The crest of the bluff above the subject site was modified by the installation of a Tecco Slope Protection System between December 2012 and November 2014 as part of an emergency bluff repair project, which involved the installation of 32 helical anchors ranging in depth from 14 to 16 feet, and 129 anchors installed to depths ranging from 10 to 18 feet, under the observation of Pacific Crest Engineering, Inc. and confirmed in final construction summary and site observation letters prepared by the firm (referenced in Exhibit P).

The slope stability analysis included with the technical reports submitted to date for the proposed residence at 379 Beach Drive do not accurately reflect as-built conditions at the crest of the bluff, and instead relies on a slope stability model that includes vertical rows of six grouted tiebacks approximately 18 feet long at the top of the bluff face. The substitution of grouted tiebacks (or grouted soil nails) installed between two to eight feet deeper than the verified helical anchor installation depths could impact the results of the slope stability model by increasing the calculated global stability of the bluff face.

County staff requested a revised global bluff face slope stability analysis that does not incorporate blufftop soil reinforcement from grouted tiebacks or grouted soil nails via a letter dated 7 October 2019 (Exhibit P). The applicant has not submitted a response to date, and the soils and geologic reports remain in "not accepted" status.

It should also be noted that the lack of resolution of the technical issues outlined above renders staff unable to confirm that the proposed scope of work is compliant with respect to other portions of the County Geologic Hazards Ordinance, as referenced in County correspondence to the applicant dated November 7, 2018 (Exhibit K).

Coastal Development Permit

The proposed project includes demolition of an existing three-story single-family and construction of a new three-story replacement single-family dwelling. The project includes excavation of the toe of the coastal bluff, construction of a combination of coastal bluff stabilization features including two retaining walls (9 and 11-feet high respectively) in the rear yard, as well as a ten-foot high geobrugg debris flow fence located 30-40 feet upslope from the proposed home. The project is located within the Coastal Appeals Jurisdiction and requires approval of a Coastal Development Permit.

Variance

As noted in the Project Description, the subject property is located in an area consisting primarily of two- and three-story homes. Many of the homes along this stretch of Beach Drive are non-conforming to the required 20-foot front yard setback. The project as proposed includes a rear yard area, which is atypical of new construction along this stretch of Beach Drive. Recently redeveloped homes in this area are designed to mitigate the hazard posed by failure of the bluff located above the residences by siting the residence against the bluff toe and structurally reinforcing the walls and roof of the home to allow debris to flow over and around the structure in the event of a landslide.

The property is zoned for residential uses and located inside the Urban Services Line (USL), an area which allows a maximum of two stories per SCCC 13.10.323 and General Plan Policy 8.6.3. Due to the project location within the mapped FEMA VE Flood zone, the applicant requests a variance to increase the allowed number of stories to three. The first story of the home is proposed to be a non-habitable garage, entry and storage area in order to comply with required FEMA regulations for construction of habitable structures within the coastal high hazard zone. Although the first story is non-habitable, the area is included in the calculation for Floor Area Ratio (FAR). If the lower floor was not counted toward FAR, the project would comply with the maximum allowed FAR of 50% of the parcel area. The proposed garage and entry have a ceiling height of 7 feet 6 inches and per SCCC 13.10.323 all areas which contain a ceiling height of 5 feet or greater count toward FAR. As proposed, the project requires a variance to increase the allowed 50% FAR to approximately 60%.

Under State law and SCCC 13.10.230, a variance may be approved where, because of special circumstances applicable to a property, strict application of the zoning ordinance deprives the property of privileges enjoyed by other property in the vicinity and under identical zoning classification. In addition, a finding must be made that the granting of a variance will be in harmony with the general intent and purpose of zoning objectives, and will not constitute a grant of special privileges inconsistent with the limitations on other properties in the vicinity and zone. Upon preliminary evaluation of the proposed variances, it appears the variances could be supported, as the strict application of the zoning ordinance would deprive the property of privileges enjoyed by neighboring properties in that the project site is located adjacent to a coastal bluff in the FEMA flood zone. The completion of the technical report reviews may result in modifications to the proposed development (i.e. bunker style home, larger setback, higher retaining walls). Formal evaluation of the any proposed variances associated with the final design of the project will occur following acceptance of the reports and finalization of the project design.

Environmental Review

The proposed development was submitted on February 14, 2018 and the project was deemed complete on January 22, 2019. The technical reports and associated updates provided by the applicant have not been accepted by the County. Additional information is required in order to determine compliance with Santa Cruz County Code (SCCC) Chapter 16.10 – Geologic Hazards Ordinance, which requires the applicant to identify appropriate mitigations for all geologic hazards affecting the site. At this time, the County Geologist and Civil Engineer have determined that adequate mitigations have not been identified with respect to the threat of potential deep-seated bluff failure.

Pursuant to Article 19 (Categorical Exceptions) of the California Environmental Quality Act CEQA Guidelines, replacement structures including new single-family residences in areas designated for residential uses are typically exempt from further environmental review under CEQA. However, at the time staff deemed project was deemed "complete" the submitted technical information was insufficient to determine the project will not have a "significant effect" on the environment due to the unusual circumstances posed by the threat of both shallow and deep-seated landsliding associated with the bluff immediately above the subject site, and that the project would directly or indirectly cause potential adverse effects, including the risk of loss, injury, or death involving seismic-related ground failure or landsliding.

Article 20 (Definitions) of the CEQA Guidelines defines "significant effect" as "a substantial, or potentially substantial, adverse change in any of the physical conditions within the area affected by the project", and "environment" is defined as "the physical conditions which exist within the area which will be affected by a proposed project including land, air water etc." A preliminary CEQA determination was made on March 8, 2019 that there is a reasonable possibility that the project will have a significant effect on the environment, therefore the project is not categorically exempt from CEQA (Article 19, Section 15300.2) and requires completion of an initial study if the project was recommended for approval.

Pursuant to Article 5 of the CEQA Guidelines, Section 15060(a), and as necessary to facilitate acceptance of the Geotechnical (Soils) Report prepared by the geotechnical engineer/consultant, County staff requested additional information to evaluate the potential environmental effects in the event of a deep-seated landslide originating above the proposed residence, as noted in the Geologic Hazards section above.

To date, County staff have not received the requested additional information, and as such cannot comprehensively evaluate the potential environmental effects of deep-seated landsliding above the proposed residence, and cannot conclude that the fundamental siting and design aspects of the proposed home would comply with the LCP and County Code.

Zoning & General Plan Consistency

Santa Cruz County General Plan (SCCGP) Objective 6.2 aims "to reduce safety hazards and property damage caused by landslides and other ground movements affecting land use activities in areas of unstable geologic formations, potentially unstable slopes and coastal bluff retreat". SCCGP Policy 6.2.6 requires location of structures away from potentially unstable slopes if feasible, and SCCGP Policy 6.2.4 sets forth that location of a proposed development shall be denied if it is found that geologic hazards cannot be mitigated to within acceptable risk levels.

In addition, SCCC section 16.10.070(H)(5)(a) (Coastal Bluff and Beaches) requires for all "development[1]", demonstration that the potential hazards on the site can be mitigated, over the 100-year lifetime of the structure, as determined by the geologic hazards assessment or a full geologic report[2] and any other appropriate technical reports. Mitigations can include, but are not limited to building setbacks, elevation of the proposed structure and foundation design.

For these reasons, outlined in the Geologic Hazards section above, the Geologic and Geotechnical Reports have <u>not</u> been accepted by the County Geologist and County Geotechnical Engineer, as they do not accurately characterize the potential impacts of deep-seated landsliding, rendering staff unable to determine that the proposed design adequately mitigates geologic hazards in compliance with SCCGP Policy 6.2.4 and SCCC Section 16.10.070(H)(5)(a).

Local Coastal Program Consistency

The proposed project is not in conformance with Objective 6.2 (Slope Stability) of the County of Santa Cruz certified Local Coastal Program (LCP). As outlined above, the proposed development is in an area subject to geologic hazards. Geotechnical (Soils) and Geologic Reports have <u>not</u> been accepted, which are required to ensure the project adequately mitigates the potential hazards that exist on the project site. Additionally, the visual simulations of the proposed development do not sufficiently demonstrate whether the proposed geobrugg debris flow fencing would be visible from the public beach/viewshed.

While the proposed home appears to be in scale with, and integrated with the character of the surrounding neighborhood, the project scope may not be completely defined, as additional information has been requested in order to accept the Geotechnical (Soils) and Geologic Report review. The completion of the technical report reviews may result in modifications to the proposed development (i.e. bunker style home, larger setback, higher retaining walls). Therefore, it cannot be determined that the project complies with the Local Coastal Program until the technical reports are accepted by the County Geologist and Geotechnical Engineer.

Conclusion

As proposed, the project is inconsistent with several applicable codes and policies of the Zoning Ordinance and General Plan/Local Coastal Program. Specifically, Coastal Development Permit Findings cannot be made, therefore it is not necessary to further evaluate the appropriateness of the other approvals being requested. Please see Exhibit "B" ("Findings") for a complete listing of findings and evidence related to the above discussion.

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[1] Per SCCC Section 16.10.040(19)(a), *"Development"* for the purposes of the Geologic Hazards Chapter includes "the construction or placement of any habitable structure, including a manufactured home and including a non-residential structure occupied by property owners, employees and/or the public". As such, the proposed residence is considered "development" for the purposes of determining compliance with the SCCC Geologic Hazards ordinance.

[2] SCCC Section 16.10.040(33) defines *"Geologic report, full"* as a complete geologic investigation conducted by a certified engineering geologist hired by the applicant and completed in accordance with the County geologic report guidelines.

Staff Recommendation

- Determine that the proposed action is exempt from further Environmental Review under the California Environmental Quality Act. CEQA Section 15270 states that "CEQA does not apply to projects which a public agency rejects or disapproves."

- **DENIAL** of Application Number **181024,** based on the attached findings.

Supplementary reports and information referred to in this report are on file and available for viewing at the Santa Cruz County Planning Department, and are hereby made a part of the administrative record for the proposed project.

The County Code and General Plan, as well as hearing agendas and additional information are available online at: www.co.santa-cruz.ca.us

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Exhibits

A. Statutory Exemption (CEQA determination)
B. Findings
C. Parcel Information
D. Project Plans & Visual Simulations
E. Assessor's, Location, Zoning and General Plan Maps
F. *Geotechnical (Soils) Report* prepared by Pacific Crest Engineering dated November 30, 2017
G. *Geologic Report* prepared by Zinn Geology dated 11 February 2018
H. *County of Santa Cruz Report Review letter* dated 26 March 2018
I. *Response to Review of Geotechnical Investigation and Supplemental Analysis* prepared by Pacific Crest Engineering dated August 16, 2018
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N. *Response to 18 July 2019 County of Santa Cruz Comments* prepared by Zinn Geology dated 23 August 2018
O. *Email string between Anna DiBenedetto, Rick Parks and Carolyn Burke* dated September 11, 2019 thru October 8, 2019
P. Exhibit P - *County of Santa Cruz Response Review letter* dated 7 October 2019
Q. Comments & Correspondence

CALIFORNIA ENVIRONMENTAL QUALITY ACT
NOTICE OF EXEMPTION

The Santa Cruz County Planning Department has reviewed the project described below and has determined that it is exempt from the provisions of CEQA as specified in Sections 15061 - 15332 of CEQA for the reason(s) which have been specified in this document.

Application Number: 181024
Assessor Parcel Number: 043-095-14
Project Location: 379 Beach Drive

Project Description: **Demolition of an existing single family dwelling and construction of a replacement single family dwelling.**

Person or Agency Proposing Project: **Matson Britton Architects**

Contact Phone Number: (831) 425-0544

A. _____ The proposed activity is not a project under CEQA Guidelines Section 15378.
B. _____ The proposed activity is not subject to CEQA as specified under CEQA Guidelines Section 15060 (c).
C. _____ **Ministerial Project** involving only the use of fixed standards or objective measurements without personal judgment.
D. __X__ **Statutory Exemption** other than a Ministerial Project (CEQA Guidelines Section 15260 to 15285).

E. _____ **Categorical Exemption**

F. Reasons why the project is exempt: 15270. PROJECTS WHICH ARE DISAPPROVED
(a) CEQA does not apply to projects which a public agency rejects or disapproves.
(b) This section is intended to allow an initial screening of projects on the merits for quick disapprovals prior to the initiation of the CEQA process where the agency can determine that the project cannot be approved.
(c) This section shall not relieve an applicant from paying the costs for an EIR or Negative Declaration prepared for his project prior to the Lead Agency's disapproval of the project after normal evaluation and processing.

In addition, none of the conditions described in Section 15300.2 apply to this project.

Nathan MacBeth, Project Planner

Date: 3-11-20

EXHIBIT A

Development Permit Findings

1. That the proposed location of the project and the conditions under which it would be operated or maintained will not be detrimental to the health, safety, or welfare of persons residing or working in the neighborhood or the general public, and will not result in inefficient or wasteful use of energy, and will not be materially injurious to properties or improvements in the vicinity.

This finding cannot be made, in that the project is located in an area subject to geologic hazards. Santa Cruz County Code Section 16.10.070(H)(5)(a) (Coastal Bluff and Beaches) states that for all development, demonstration that the potential hazards on the site can be mitigated, over the 100-year lifetime of the structure, as determined by the geologic hazards assessment or full geologic report and any other appropriate technical reports. Mitigations can include, but are not limited to building setbacks, elevation of the proposed structure and foundation design. Geologic and geotechnical (soils) reports have not been accepted for the proposed development, therefore the hazards affecting the site have not been adequately addressed and mitigated as required by County Code.

The proposed single family dwelling is located at the base of a coastal bluff and constitutes "Development/Development Activities" as defined in Santa Cruz County Code Section 16.10.040(19). SCCC Section 16.10.070(E)(1) (Slope Stability) states all development activities shall be located away from potentially unstable areas as identified through the geologic hazards assessment, full geologic report, soils report or other environmental or technical assessment. The geologic and geotechnical (soils) reports have not been accepted, and the potentially unstable area has not been properly identified, nor have the hazards associated with development at the base of a coastal bluff been adequately addressed.

Santa Cruz County Code Section 16.10.060(C) (Report Acceptance) states all geologic, geotechnical, engineering, and hydrologic reports or investigations submitted to the County as a part of any development application shall be found to conform to County report guidelines. The geologic and geotechnical (soils) reports have not been accepted, as they do not conform to County report guidelines.

2. That the proposed location of the project and the conditions under which it would be operated or maintained will be consistent with all pertinent County ordinances and the purpose of the zone district in which the site is located.

This finding cannot be made, in that geologic and geotechnical (soils) reports have not been accepted, as they do not conform to County report guidelines. Consequently, the proposed location and design of the single family dwelling cannot be comprehensively evaluated to ensure compliance with County Code and the LCP with respect to the potential environmental effects of deep-seated landsliding above the proposed residence.

3. That the proposed use is consistent with all elements of the County General Plan and with any specific plan which has been adopted for the area.

This finding cannot be made, in that the proposed use will be inconsistent with General Plan Policy 6.2.15 (New Development on Existing lots of record) which allows development activities

EXHIBIT B

in areas subject to storm wave inundation or beach or bluff erosion on existing lots of record, within existing developed neighborhoods, under the following circumstance:

(a) A Technical report (including a geologic hazards assessment, engineering geology report and /or soils engineering report) demonstrates that the potential hazards can be mitigated over the 100-year lifetime of the structure. Mitigations can include but not limited to, building setbacks, elevation of the structure, and foundation design;
(b) Mitigation of the potential hazard is not dependent on shoreline or coastal bluff protection structures except on lots where both adjacent parcels are already similarly protected; and
(c) The Owner records a Declaration of Geologic Hazards on the property that describes the potential hazard and the level of geologic and /or geotechnical investigation conducted.

The project is located within an are subject to tidal and wave inundation and located at the base of a steep, eroding coastal bluff, an area identified by the County Geologist and Senior Civil Engineer as being subject to Geologic Hazards. Technical reports in the form of an Engineering Geology and geotechnical report have been required, however these reports have not been accepted by the County due to technical deficiencies. In absence of accepted technical reports, this finding cannot be made in that the proposed design does not adequately mitigate potential geologic hazards.

Coastal Development Permit Findings

5. That the proposed development is in conformity with the certified Local Coastal Program.

This finding <u>cannot</u> be made in that the proposed project is not in conformance with Objective 6.2 (Slope Stability) of the County of Santa Cruz certified Local Coastal Program (LCP). The proposed development is in an area subject to geologic hazards. Geotechnical (Soils) and Geologic Reports have <u>not</u> been accepted, which are required to ensure the project adequately mitigates the potential hazards that exist on the project site. Additionally, the visual simulations of the proposed development do not sufficiently demonstrate whether the proposed geobrugg debris flow fencing located 30-40 feet up slope from the proposed home would be visible from the public beach/viewshed.

While the proposed dwelling appears to be in scale with, and integrated with the character of the surrounding neighborhood, the project scope may not be completely defined, as additional information has been requested in order to accept the Geotechnical (Soils) and Geologic Report review. The completion of the technical report reviews may result in modifications to the proposed development (i.e. bunker style home, larger setback, higher retaining walls). Therefore, it cannot be determined that the project complies with the Local Coastal Program until the technical reports are accepted by the County Geologist and Geotechnical Engineer.

Variance Findings

2. That the granting of the variance will be in harmony with the general intent and purpose of zoning objectives and will not be materially detrimental to public health, safety, or welfare or injurious to property or improvements in the vicinity.

This finding <u>cannot</u> be made in that technical reports have not been accepted for the proposed development. As proposed, the project has not demonstrated that the design adequately mitigates potential geologic hazards affecting the project site. The design of the proposed development and the variances being requested are predicated on technical information that has not been accepted by County Staff. In the absence of accepted geologic and geotechnical reports that identify adequate mitigations for the geologic hazards present at the site, it cannot be determined that the project, as designed, will not result in adverse impacts to public health, safety, or welfare or injurious to property of improvements in the vicinity.

EXHIBIT B

Parcel Information

Services Information

Urban/Rural Services Line:	_X_ Inside __ Outside
Water Supply:	Soquel Creek Water District
Sewage Disposal:	County of Santa Cruz Sanitation District
Fire District:	Aptos La Selva Fire Protection District
Drainage District:	Flood Control District

Parcel Information

Parcel Size:	5,763 Square Feet
Existing Land Use - Parcel:	Residential
Existing Land Use - Surrounding:	Residential
Project Access:	Beach Drive
Planning Area:	Aptos
Land Use Designation:	R-UL (Urban Low Density Residential)
Zone District:	R-1-6 (Single Family Residential - 6,000 square foot minimum)
Coastal Zone:	_X_ Inside __ Outside
Appealable to Calif. Coastal Comm.	_X_ Yes __ No

Technical Reviews: Combined Geologic and Geotechnical Report Review (REV181023)

Environmental Information

Geologic Hazards:	Located at the toe of a coastal bluff and within VE Flood Zone
Fire Hazard:	Not a mapped constraint
Slopes:	Coastal Bluff at the rear of property
Env. Sen. Habitat:	Not mapped
Grading:	119 cubic yards of cut
Tree Removal:	No trees proposed to be removed
Scenic:	Mapped scenic resource
Archeology:	Not mapped

EXHIBIT C

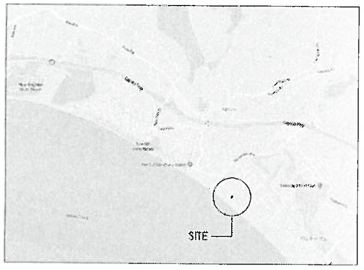
ABBREVIATIONS

&	AND
L, A	ANGLE
@	AT
°	DEGREE
A.B.	ANCHOR BOLT
(A)	ABOVE
A.C.I.	AMERICAN CONCRETE INSTITUE
ADJ.	ADJACENT
A.F.F.	ABOVE FINISH FLOOR
A.I.S.C.	AMERICAN INSTITUE OF STEEL CONSTRUCTION
ALT.	ALTERNATE
ALUM.	ALUMINUM
APPROX.	APPROXIMATELY
ARCH.	ARCHITECTURAL
A.S.T.M.	AMERICAN SOCIETY OF TESTING MATERIALS
	BELOW
(B)	BOARD
BD.	BUILDING
BLDG.	BLOCKING
BLKG.	BEAM
BM.	BOUNDARY NAILING
B.N.	BOTTOM OF
B.O.	BOTTOM
BOT., BOTT.	BETWEEN
BTWN.	CABINET
CAB.	CEILING BEAM
C.B.	CEILING JOIST
C.J.	CEILING
CLG.	CLEAR
CLR.	COLUMN
COL.	CONCRETE
CONC.	CONTINUOS
CONT.	CENTER
CTR.	CENTERLINE
CL	BAR DIAMETER
Db	DOUBLE
DBL.	DEGREE
DEG.	DEMOLISH
DEMO.	DETAIL
DET., DTL.	
D.W.	DISHWASHER
DWG.	DRAWING
DWN., DN.	DOWN
(E)	EXISTING
EA.	EACH
E.N.	EDGE NAILING
EL., ELEV.	ELEVATION
ELEV.	ELEVATOR
ENG.	ENGINEER
EQ.	EQUAL
EXT.	EXTERIOR
E.W.	EACH WAY
F.B.	FLOOR BEAM
F.F.	FINISHED FLOOR
	FINISH(ED)
FIN.	FLOOR JOIST
F.J.	FLUSH
FL.	FLOOR
FLR.	FIELD NAILING
F.N.	FOUNDATION
FND.	FACE OF
F.O.	FIREPLACE
FP.	FIRE RATED
F.R.	FOOT OR FEET
FT.	FOOTING
FTG.	FREEZER
FZR.	GAUGE
GA.	GALVANIZED
GALV.	GRADE BEAM
G.B.	GLU-LAM BEAM
GLB.	GYPSUM WALL
GYP. BD., G.W.B.	BOARD

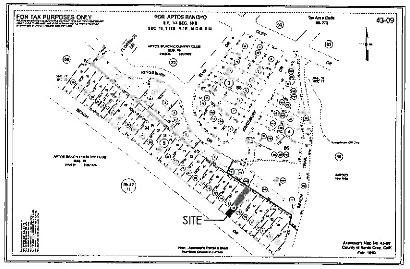
H.B.	HOSE BIB
HDR.	HEADER
HDWR.	HARDWARE
HORIZ.	HORIZONTAL
HT., H.	HEIGHT
I.D.	INSIDE DIAMETER
IN.	INCH(ES)
INSUL.	INSULATION
INT.	INTERIOR
JT.	JOINT
K.P.	KING POST
L.	LENGTH
LIN.	LINEAR
MAX.	MAXIMUM
M.B.	MACHINE BOLT
MEMB.	MEMBRANE
MFR.	MANUFACTURER
MIN.	MINIMUM
MISC.	MISCELLANEOUS
MTL.	METAL
MW.	MICROWAVE
N.	NORTH
(N)	NEW
N.T.S.	NOT TO SCALE
O/	OVER
O.C.	ON CENTER
O.D.	OUTSIDE DIAMETER
O.H.	OPPOSITE HAND
OV.	OVEN
N.I.C.	NOT IN CONTRACT
PL.	PLATE
PLYWD.	PLYWOOD
PKG.	PARKING
P.S.F.	POUNDS PER SQUARE FOOT
P.S.I.	POUNDS PER SQUARE INCH
QTY.	QUANTITY
RAD.	RADIUS
R.B.	ROOF BEAM
RCP.	REFLECTED CEILING PLAN
RE:	REFERENCE
REF.	REFRIGERATOR
REINF.	REINFORCED
REQ'D.	REQUIRED
RM.	ROOM
R.O.	ROUGH OPENING
R.R.	ROOF RAFTER
SCHED.	SCHEDULE
SF., SQ. FT.	SQUARE FOOT
SHTG.	SHEATHING
SHT.	SHEET
SIM.	SIMILAR
SL.	SLOPED
SPKL.	SPRINKLER
SQ.	SQUARE
STAGG.	STAGGER
STD.	STANDARD
STL.	STEEL
STR., STRUCT.	STRUCTURAL
T&B	TOP & BOTTOM
T&G	TONGUE & GROOVE
THK.	THICK
T.O.	TOP OF
T.P.	TOILET PAPER
TYP.	TYPICAL
U.B.C.	UNIFORMED BUILDING CODE
VERT.	VERTICAL
W.	WIDTH
WD.	WOOD
WH.	WATER HEATER

VAUDAGNA RESIDENCE

NEW RESIDENCE
379 BEACH DRIVE
APTOS, CALIFORNIA



VICINITY MAP



PARCEL MAP

CODE COMPLIANCE

THIS RESIDENTIAL CONSTRUCTION COMPLIES WITH TITLE 24 AND THE FOLLOWING CODES:
2016 CALIFORNIA RESIDENTIAL CODE (CRC),
2016 CALIFORNIA BUILDING CODE (CBC),
2016 CALIFORNIA MECHANICAL CODE (CMC),
2016 CALIFORNIA PLUMBING CODE (CPC),
2016 CALIFORNIA ELECTRICAL CODE (CEC) AND THE
2016 CALIFORNIA ENERGY CODE (CEnC).

FIRE PROTECTION NOTES

1. THESE PLANS SHALL COMPLY WITH 2016 CALIFORNIA BUILDING CODE AND 2016 CALIFORNIA FIRE CODE AND DISTRICT AMENDMENTS.
2. OCCUPANCY R-3 & U, TYPE V-B, SPRINKLERED.
3. ADDRESS NUMBERS SHALL BE POSTED AND MAINTAINED AS SHOWN ON THE SITE PLAN. NUMBERS SHALL BE A MINIMUM OF 4 INCHES IN HEIGHT AND OF A COLOR CONTRASTING TO THEIR BACKGROUND.
4. ROOF COVERING SHALL BE NO LESS THAN CLASS "B" RATED.
5. ALL CHIMNEYS SHALL BE APPROVED WITH AN APPROVED SPARK ARRESTOR ON THE TOP OF THE CHIMNEY. WIRE MESH NOT TO EXCEED ½" IS ACCEPTABLE.
6. THE JOB COPIES OF THE BUILDING PLANS AND PERMITS MUST REMAIN ON-SITE DURING INSPECTIONS.
7. PUBLIC FIRE HYDRANT REQUIRED WITHIN 400 FT. OF ANY PORTION OF THE BUILDING WITH A MINIMUM 1500 GALLON FIRE FLOW. AVAILABLE FIRE HYDRANT APPROXIMATELY 360' FROM BUILDING.

CONSULTANTS

ARCHITECTS:	MATSON BRITTON ARCHITECTS 728 N. BRANCIFORTE SANTA CRUZ, CA 95062 PHONE: 831-425-0544 FAX: 831-425-4795
ENGINEERING:	R.I. ENGINEERING, INC. 303 POTRERO STREET, STE. 42-202 SANTA CRUZ, CA 95060 PHONE: 831-425-3901 FAX: 831-425-1522
SURVEYING:	HANAGAN LAND SURVEYING, INC 305-C SOQUEL AVE SANTA CRUZ, CA 95062 PHONE: 831-469-3428 FAX: 831-469-3400
GEOLOGIST:	ZINN GEOLOGY 2231 40th Avenue Santa Cruz, CA 95062 Voice: (831) 334-4833
GEOTECHNICAL ENGINEERS	PACIFIC CREST ENGINEERING INC 444 Airport Blvd, Suite 106 Watsonville, CA 95076 PHONE: (831) 722-9446 FAX: (831) 722-9158

PROJECT INFORMATION

OWNER:	JIM & SUE VAUDAGNA 379 BEACH DRIVE APTOS, CA 95003
A. P. N.:	043-095-14
ZONING:	R-1-6
OCCUPANCY GROUP:	R-3 & U (PER 2016 CBC)
CONSTRUCTION TYPE:	V-B SPRINKLERED

PROJECT DESCRIPTION:
THE DEMOLITION OF AN EXISTING 3 STORY 2070 SF RESIDENCE, AND THE CONSTRUCTION OF A NEW 3 STORY 2631SF RESIDENCE WITH EXTERIOR STAIRS, GARAGE, DECK AND BALCONY.

THIS PROJECT CONSISTS OF:
THE DEMOLITION OF AN EXISTING 2070 SF RESIDENCE AND THE CONSTRUCTION OF A NEW 2631 SF RESIDENCE, INCLUDING A 400 SF GARAGE AND 691 SF ENTRY AND STORAGE AT THE GROUND FLOOR; A 1456 SF SECOND STORY LIVING AREA , A 1175 SF THIRD STORY LIVING AREA WITH A 200 SF DECK AND 38 SF BALCONY.
THE PROPERTY IS LOCATED AT 379 BEACH DRIVE.

UTILITY DISTRICTS: SOQUEL CREEK WATER DISTRICT
SANTA CRUZ COUNTY SANITATION DISTRICT

SHEET INDEX

P1	TITLE SHEET
P2	SITE PLAN
P3	PROPOSED SITE SECTION
P4	PROPOSED GROUND FLOOR PLAN & ROOF PLAN
P5	PROPOSED SECOND AND THIRD FLOOR PLANS
P6	PROPOSED EXTERIOR ELEVATIONS
C-1	GRADING AND DRAINAGE PLAN
C-2	DETAILS
C-3	STORMWATER POLLUTION CONTROL PLAN
SU-1	SURVEY PLAN
SU-2	SURVEY SITE SECTION

PROJECT CALCULATIONS

LOT AREA:	
GROSS AREA:	5,763 S.F.
MAX LOT COVERAGE (40% OF NET LOT SIZE) 5,763 S.F. x .40 =	2,305 S.F. (40%)
PROPOSED LOT COVERAGE AREA:	
BUILDING FOOTPRINT:	1,110 S.F. (19%)
MAXIMUM FLOOR AREA RATIO (50% OF LOT SIZE): 5,763 S.F. x .50 =	2,882 S.F. (50%)
CONDITIONED AREA:	
1ST FLOOR (NON-HABITABLE, UNDER 7' HIGH:)	N.A. S.F.
2ND FLOOR	1,456 S.F.
3RD FLOOR (EXCLUDES STAIRWELL & ELEVATOR):	1,175 S.F.
TOTAL CONDITIONED AREA:	2,631 S.F.
GARAGE AREA:	399 S.F.
GARAGE (LESS 225 S.F. CREDIT):	174 S.F.
TOTAL F.A.R.	2,805 S.F. (48.7%)
PROPOSED ADDITION AREAS (CONDITIONED):	
1ST FLOOR (NON-CONDITIONED):	NA
2ND FLOOR	1,456 S.F.
3RD FLOOR	1,175 S.F.
TOTAL PROPOSED CONDITIONED AREA:	2,631 S.F.
3RD FLOOR OPEN DECKS AREA:	200 S.F.
3RD FLOOR OPEN BALCONY AREA:	38 S.F.
IMPERVIOUS AREAS: APPROX	CIVIL DWG

2 COVERED PARKING SPACES PROVIDED
2 UNCOVERED PARKING SPACES PROVIDED

MATSON BRITTON ARCHITECTS

728 N BRANCIFORTE
SANTA CRUZ
CA 95062
831-425-0544

NOTICE

THIS DOCUMENT IS CONFIDENTIAL INCORPORATING PROPRIETARY RIGHTS. ANY PARTY ACCEPTING THIS DOCUMENT AGREES THAT IT SHALL NOT BE DUPLICATED WHOLE OR IN PART NOR DISCLOSE TO OTHERS WITHOUT THE EXPRESS WRITTEN CONSENT OF MATSON BRITTON ARCHITECTS MATSON BRITTON ARCHITECTS, A CALIFORNIA CORPORATION

REVISIONS
1 3/29/18 PLANNING
2 9/26/18 PLANNING

VAUDAGNA
RESIDENCE
379 BEACH DRIVE
APTOS, CA 95003
APN: 043-095-14

COVE BRITTON
NO. C-23616
8/31/19
RENEWAL DATE

DATE 10 / 11 / 17
DRAWN FK
JOB VAUDAGNA
SHEET P1

EXHIBIT D

NOTE:
ALL EXISTING TREES, LANDSCAPE AND HARDSCAPE FEATURES TO REMAIN. ALL EXISTING SITE WALLS, FENCES AND GATES TO REMAIN.

NO NEW, OR DEMOLITION OF LANDSCAPE AND HARDSCAPE, OTHER THAN AT IMMEDIATE AREAS OF NEW CONSTRUCTION.
NO TREES 6" AND OVER TO BE REMOVED.

SITE LEGEND

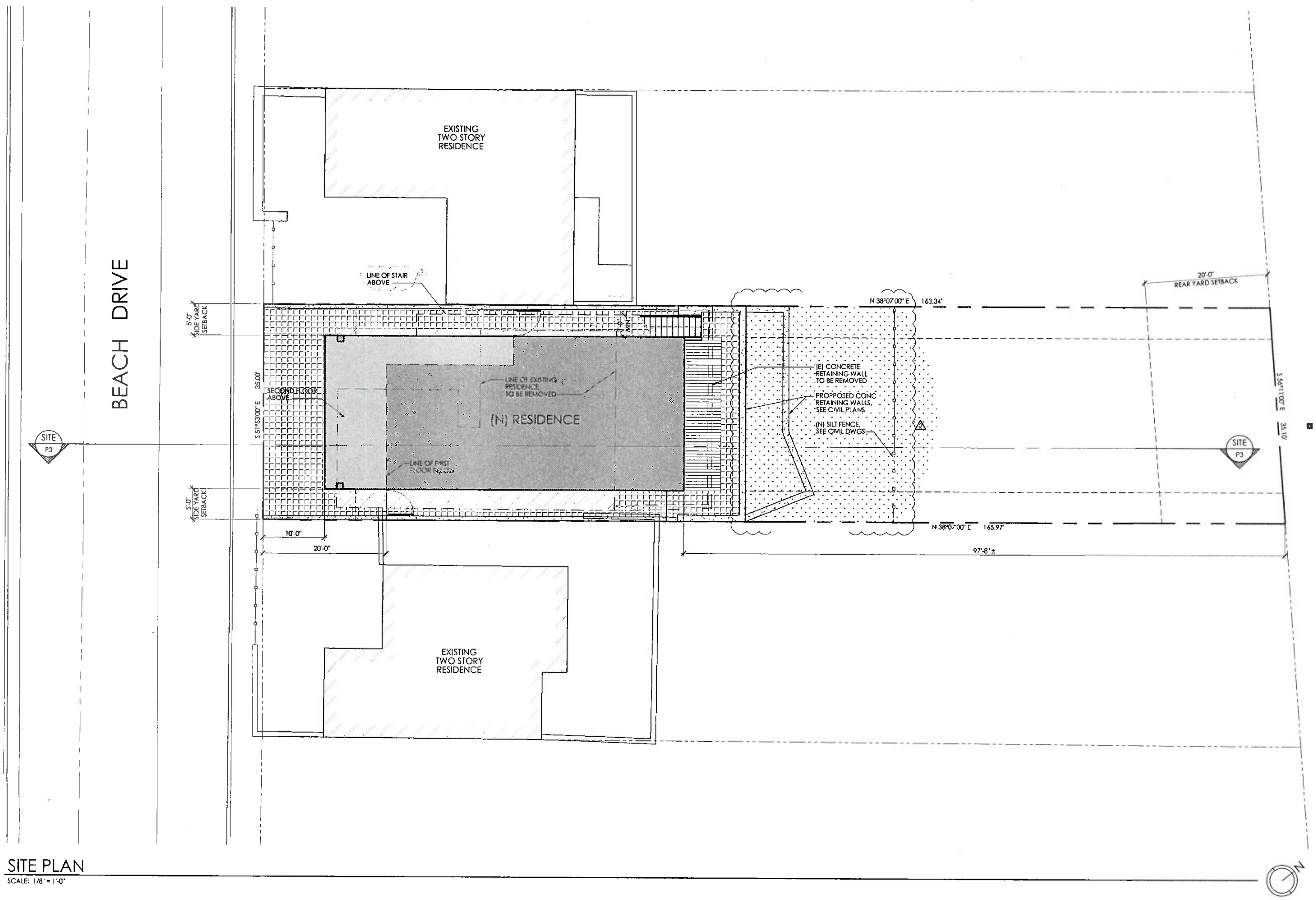
— — — —	PROPERTY LINE
— — — —	SETBACK LINE
—o—o—o—	LINE OF EXISTING WOOD FENCE
- - - -	LINE OF NEW ROOF OUTLINE
(E) AREA TO BE REMOVED	EXISTING RESIDENCE TO BE REMOVED
(N) 1ST FLOOR	PROPOSED 1ST FLOOR ADDITION
(N) 2ND FLOOR	PROPOSED 2ND FLOOR ADDITION
(N) 2ND FLOOR DECK	PROPOSED 2ND FLOOR OPEN DECK
☐	EXISTING IMPERVIOUS CONCRETE DRIVE & PATIOS
▦	NEW IMPERVIOUS DRIVE, PATIO & WALKWAY
⁘	HILLSIDE GRADE SEE CIVIL DWGS

SITE PLAN NOTES

1. SEE CIVIL DRAWINGS FOR PERTINENT INFORMATION REGARDING ALL EARTHWORK & GRADING, SITE HOUSEKEEPING, EROSION CONTROL AND STORM DRAINAGE.

DRAINAGE NOTES

1. DOWNSPOUTS TO GO TO EXISTING DRAINS.
2. ARCHITECT TO FIELD VERIFY LOCATIONS OF DOWNSPOUTS.
3. PROJECT TO MAINTAIN EXISTING DRAINAGE PATTERNS.



SITE PLAN
SCALE: 1/8" = 1'-0"

MATSON BRITTON ARCHITECTS
728 N BRANCIFORTE
SANTA CRUZ
CA 95062
831-425-0544

NOTICE

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VAUDAGNA
RESIDENCE
379 BEACH DRIVE
APTOS, CA 95003
APN: 043-095-14

SITE PLAN

COVE BRITTON
NO. C-23616
8/31/19

DATE 10/11/17
DRAWN FK
JOB VAUDAGNA
SHEET P2

EXHIBIT D

SITE SECTION

SCALE: 1/8" = 1'-0"

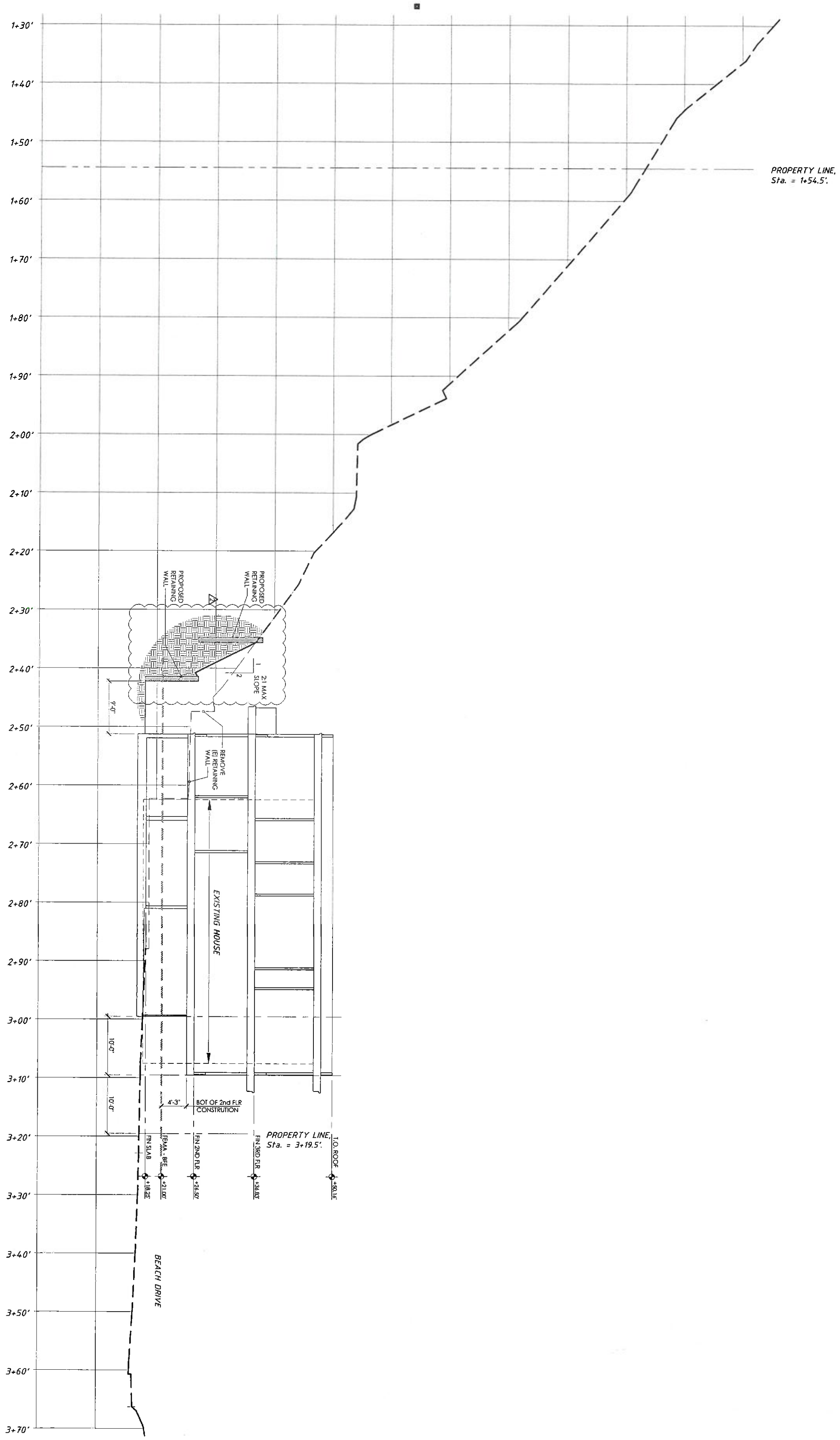


EXHIBIT D

DATE	10/11/17	
DRAWN	FK	
JOB	VAUDAGNA	
SHEET	P3	

LICENSED ARCHITECT
COVE BRITTON
NO. C-23616
8/31/19
RENEWAL DATE
STATE OF CALIFORNIA

SITE SECTION

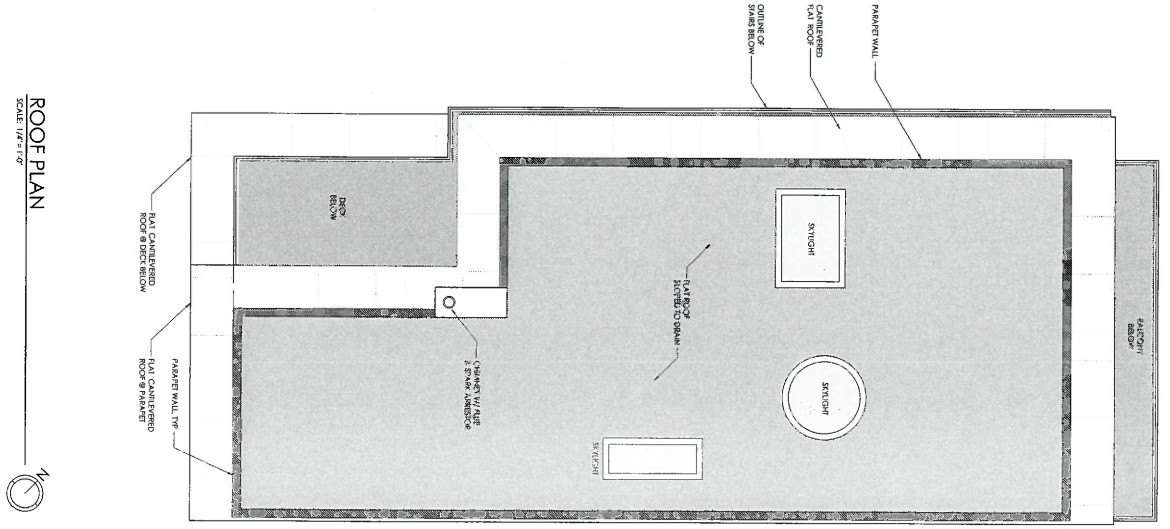
VAUDAGNA
RESIDENCE
379 BEACH DRIVE
APTOS, CA 95003
APN: 043-095-14

REVISIONS
3/29/18 PLANNING
9/26/18 PLANNING

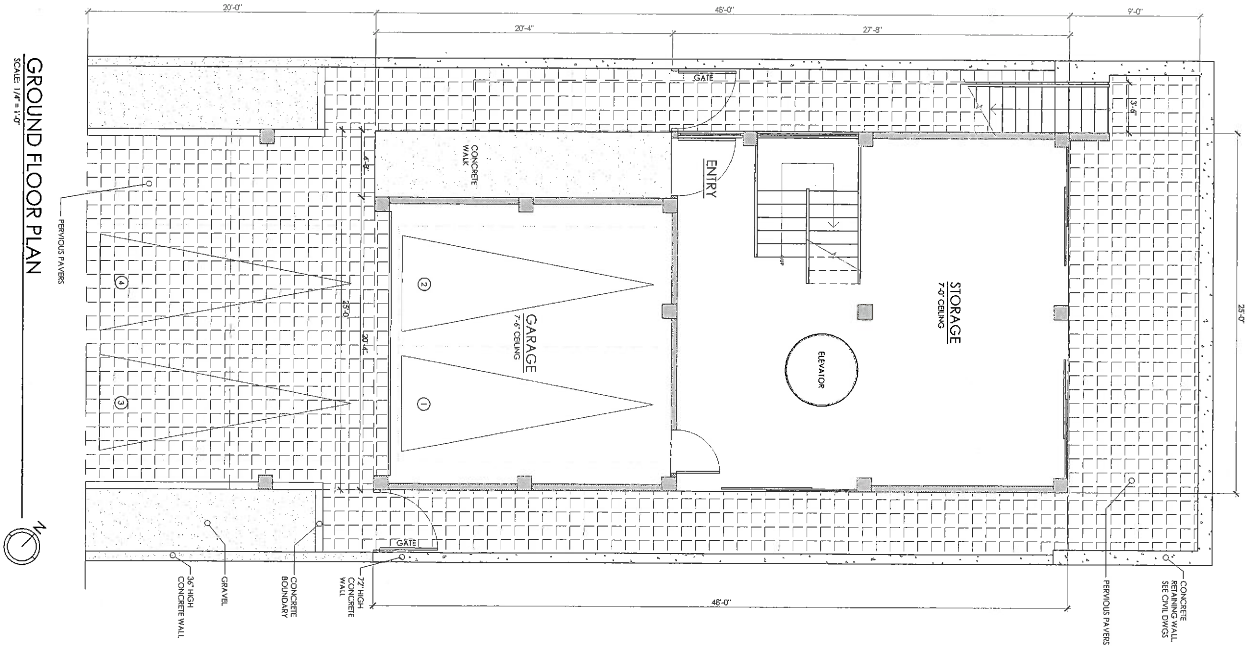
NOTICE

725 N BRANCIFORTE
SANTA CRUZ
CA 95062
831-425-0544

MATSON BRITTON



ROOF PLAN
SCALE: 1/4" = 1'-0"



GROUND FLOOR PLAN
SCALE: 1/4" = 1'-0"

EXHIBIT D

MATSON BRITTON
728 N BRANCIFORTE
SANTA CRUZ
CA 95062
831-425-0544

NOTICE

REVISIONS
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2 9/26/18 PLANNING

VAUDAGNA
RESIDENCE
379 BEACH DRIVE
APTOS, CA 95003
APN: 043-095-14

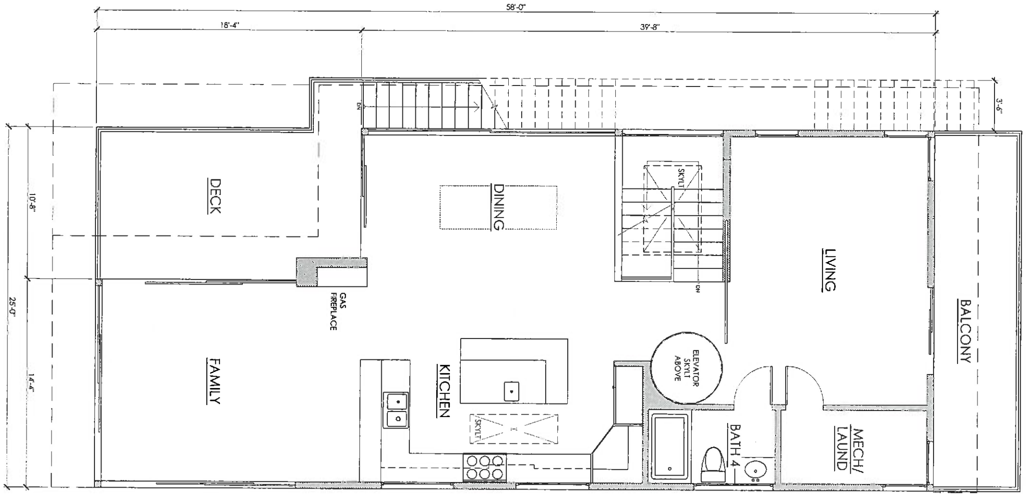
GROUND FLOOR PLAN
ROOF PLAN

LICENSED ARCHITECT
COVE BRITTON
NO. C-23616
8/31/19
RENEWAL DATE
STATE OF CALIFORNIA

DATE 10/11/17
DRAWN FK
JOB VAUDAGNA
SHEET P4

THIRD FLOOR PLAN

SCALE: 1/4" = 1'-0"



SECOND FLOOR PLAN

SCALE: 1/4" = 1'-0"

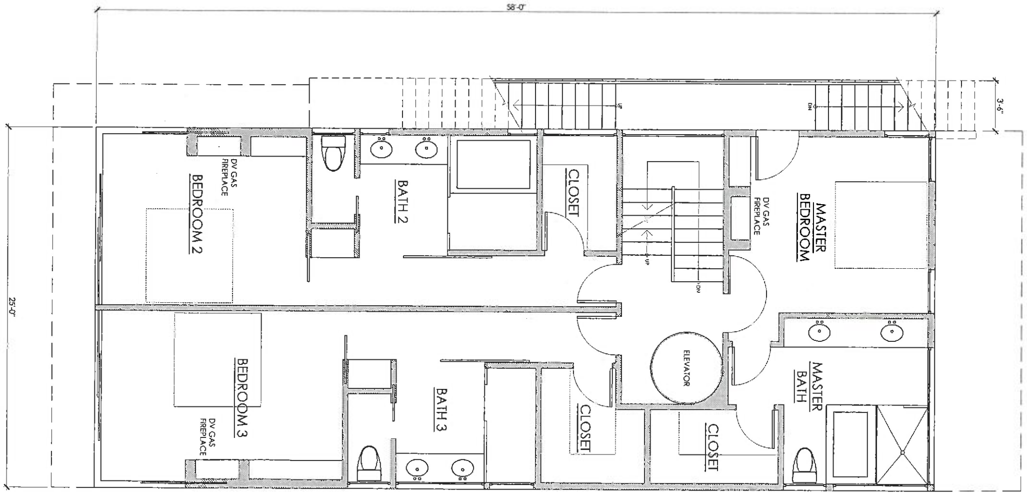


EXHIBIT D

SHEET	P5
JOB	VAUDAGNA
DRAWN	FK
DATE	10/11/17

LICENSED ARCHITECT
COVE BRITTON
NO. C-23616
8/31/19
STATE OF CALIFORNIA

SECOND & THIRD
FLOOR PLANS

VAUDAGNA
RESIDENCE
379 BEACH DRIVE
APTOS, CA 95003
APN: 043-095-14

REVISIONS
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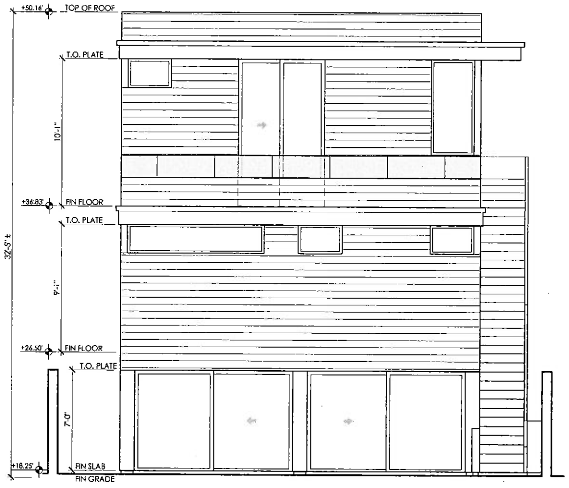
NOTICE

MATSON BRITTON ARCHITECTS
728 N BRANCIFORTE
SANTA CRUZ
CA 95062
831-425-0544

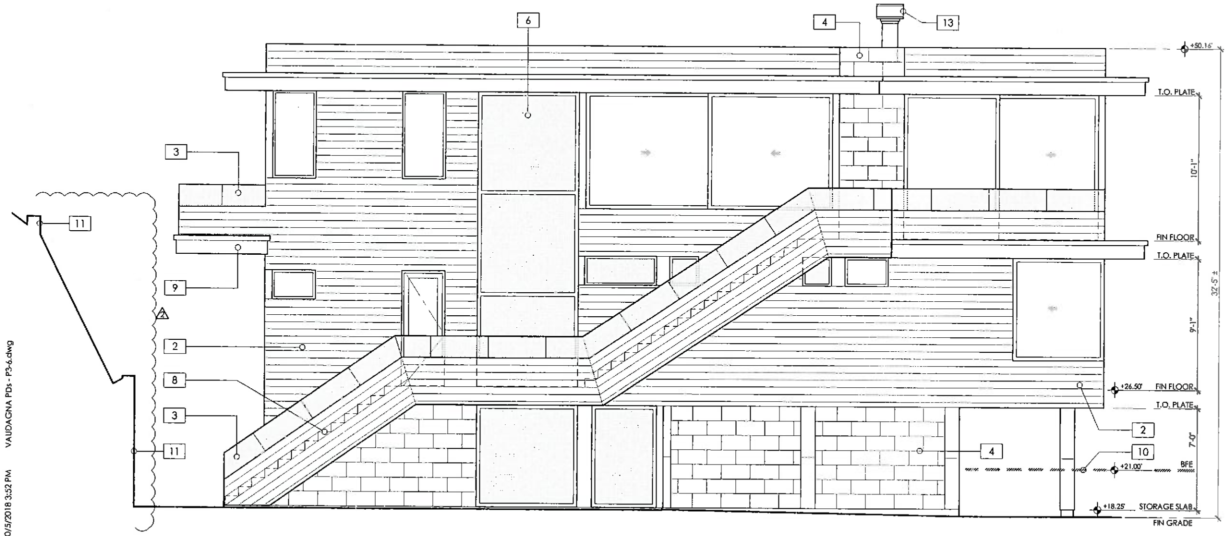
EAST ELEVATION

SCALE: 1/4" = 1'-0"



NORTH ELEVATION

SCALE: 1/4" = 1'-0"



WEST ELEVATION

SCALE: 1/4" = 1'-0"

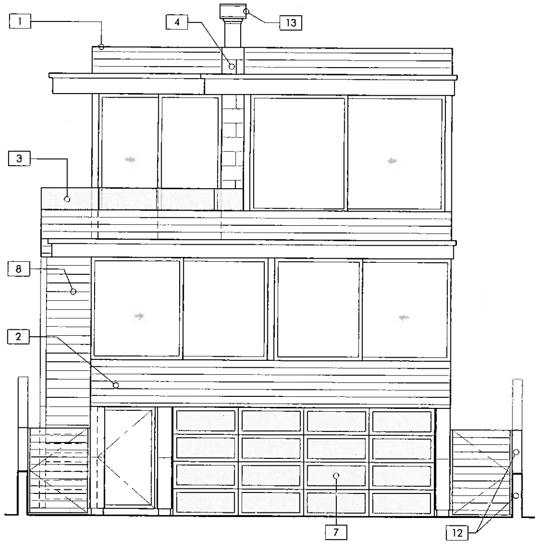
ELEVATION NOTES

1	FLAT PARAPET ROOF
2	IPE SIDING
3	WOOD/GLASS STAIR & BALCONY GUARD
4	LIMESTONE, BEIGE AT BREAKAWAY WALLS
5	BREAKAWAY WALLS
6	OBSCURE GLAZING @ STAIRWELL
7	GARAGE DOOR - METAL/GLASS PANELS
8	CANTILEVERED STAIRS
9	CANTILEVERED BALCONY
10	BFE, BASE FLOOD ELEVATION
11	CONCRETE RETAINING WALLS, SEE CIVIL DWGS
12	CONCRETE WALLS, 3' HIGH & 6' HIGH
13	CHIMNEY W/ SPARK ARRESTOR

NOTES

PROJECT TO COMPLY WITH ALL FEMA REQUIREMENTS

ALL MECHANICAL SHALL BE INSTALLED ABOVE THE BFE



SOUTH ELEVATION

SCALE: 1/4" = 1'-0"

MATSON BRITTON
ARCHITECTS

728 N BRANCIFORTE
SANTA CRUZ
CA 95062
831-425-0544

NOTICE

REVISIONS
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2 9/26/18 PLANNING

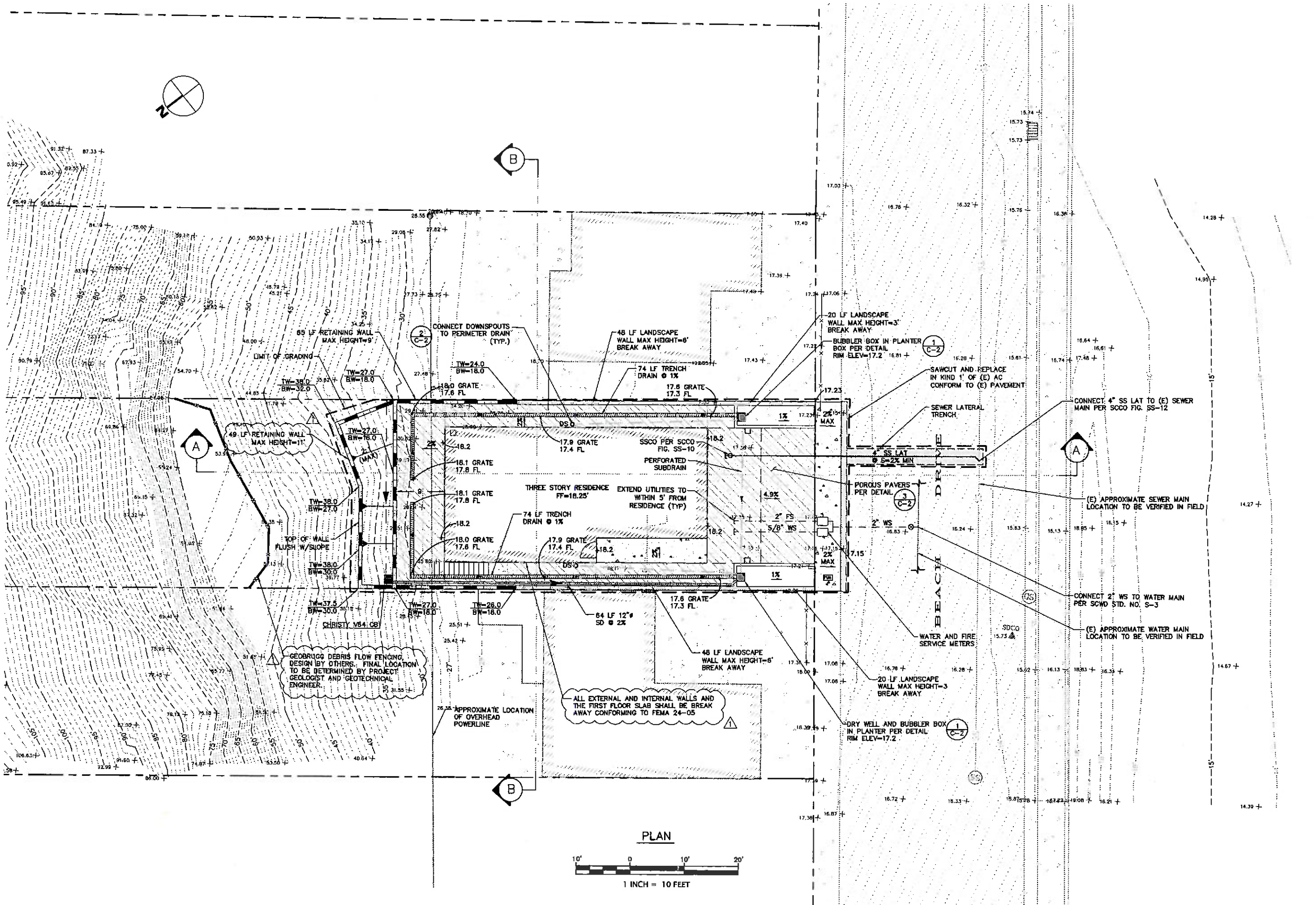
VAUDAGNA
RESIDENCE
379 BEACH DRIVE
APTOS, CA 95003
APN: 043-095-14

EXTERIOR ELEVATIONS

LICENSED ARCHITECT
COVE BRITTON
NO. C-23616
8/31/19
RENEWAL DATE
STATE OF CALIFORNIA

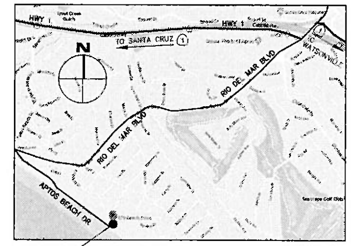
DATE
10/11/17
DRAWN
FK
JOB
VAUDAGNA
SHEET
P6

EXHIBIT D



PLAN

1 INCH = 10 FEET



VICINITY MAP

NTS

LEGEND

▭	(E) AC
▭	(E) CONCRETE
▨	PROPOSE POROUS PAVERS
▨	PROPOSED AC
─ ─ ➤ ─	(E) FLOWLINE
▬▬▬	(E) RETAINING WALL
── ─ ──	PROPERTY LINE
─ ─ ─ ─	PROPOSED SETBACK
── ── ──	PROPOSED LIMIT OF GRADING
━━━━	PROPOSED RETAINING WALL
▬▬▬	PROPOSED TRENCH DRAIN
─ · ➤ · ─	PROPOSED SWALE
━━➤━━	PROPOSED SD
─ ─ ➤ ─ ─	PROPOSED PERIMETER SD
○	PROPOSED SDCO
■	PROPOSED CB

ABBREVIATIONS

BW	BOTTOM OF WALL
CB	CATCH BASIN
CONST	CONSTRUCT
DIA, ø	DIAMETER
DS	DOWNSPOUT
DTL	DETAIL
DWY	DRIVEWAY
(E)	EXISTING
EL	ELEVATION
EOP	EDGE OF PAVEMENT
FF	FINISH FLOOR
FG	FINISH GRADE
FS	FIRE SERVICE
HP	HIGH POINT
INV	INVERT
LF	LINEAR FEET
LP	LOW POINT
MAX	MAXIMUM
N.T.S.	NOT TO SCALE
RW	RETAINING WALL
RIM	RIM ELEVATION
S	SLOPE
SCCO	COUNTY OF SANTA CRUZ
SCWD	SOQUEL CREEK WATER DEPARTMENT
SSCO	SANITARY SEWER CLEANOUT
SDCO	STORM DRAIN CLEANOUT
TYP	TYPICAL
TW	TOP OF WALL
WS	WATER SERVICE

STORM DRAINAGE NOTES

1. CULVERTS SHALL BE REINFORCED CONCRETE PIPE (RCP), POLYVINYL CHLORIDE (PVC SDR35), OR HIGH DENSITY POLYETHYLENE (HDPE ADS N12 OR EQUAL) AND SHALL HAVE A SMOOTH INTERIOR CONFORMING TO SECTION E – STORM DRAINAGE FACILITIES OF COUNTY OF SANTA CRUZ DESIGN CRITERIA.
2. INLETS SHALL BE CHRISTY CONCRETE PRODUCTS OR APPROVED EQUAL WITH SMOOTH CONCRETE BOTTOM.
3. CONNECT ALL DOWNSPOUTS TO PERIMETER STORM DRAIN.

IMPERVIOUS AREA TABLE

IMPERVIOUS AREA	EXISTING CONDITIONS	PROPOSED CONDITIONS
RESIDENCE	1,067 SF	1,129 SF
DRIVEWAY	513 SF	** 617 SF
WALKWAYS/PATIOS	170 SF	92 SF
TOTAL	1,750 SF	1,838 SF

** USE 0.5 FACTOR FOR 'SEMI-POROUS' SURFACE
1234 SF * 0.5 = 617 SF

APPROXIMATE EARTHWORK QUANTITIES

	CUBIC YARDS		
	CUT	FILL	NET
SITE GRADING	119		119 CUT
SUBGRADE PREPARATION	90		90 CUT
		TOTAL	209 CUT

NOTES:
1. EARTHWORK QUANTITIES ARE APPROXIMATE AND SHALL BE INDEPENDENTLY VERIFIED BY THE CONTRACTOR FOR BIDDING PURPOSES.
2. EARTHWORK VOLUMES FOR RESIDENCE GRADING INCLUDE EXCAVATION TO ROUGH GRADE FOR CONSTRUCTION OF THE PROPOSED RESIDENCE. EARTHWORK VOLUMES REQUIRED TO CONSTRUCT THE FOUNDATIONS HAVE NOT BEEN INCLUDED.
3. EXCESS SOIL SHALL BE HAULED OR PLACED IN A COUNTY APPROVED LOCATION.

POST CONSTRUCTION STORM DRAIN SYSTEM MAINTENANCE SCHEDULE

ITEM	INTERVAL	INSPECTION	REPAIR
CATCH BASINS	ANNUAL	1. SEDIMENT BUILD UP 2. TRASH & DEBRIS	1. REMOVE SEDIMENT 2. REMOVE TRASH & DEBRIS
POROUS PAVERS	ANNUAL	1. RAVELING OF PAVERS 2. SCOUR AROUND PERIMETER 3. ACCUMULATION OF SEDIMENT	1. REPLACE/RESET PAVERS 2. VACUUM SWEEP PAVERS

TOPOGRAPHIC SURVEY

THE TOPOGRAPHIC SURVEY AND BOUNDARY INFORMATION PROVIDED HEREON WAS COMPLETED BY HANAGAN LAND SURVEYORS. RI ENGINEERING INC. MAKES NO GUARANTEE AS TO THE ACCURACY OF BOTH. THE CONTRACTOR SHALL VERIFY THE BOUNDARY LOCATION AND TOPOGRAPHIC INFORMATION PRIOR TO COMMENCING WORK.

BASIS OF BEARINGS

THE BASIS OF BEARING FOR THE MAP IS S 51' 53' E BETWEEN FOUND MONUMENTS ON THE NORTH EASTERLY LINE BEACH DRIVE PER RECORD MAP 024-M-26

BASIS OF ELEVATION

COUNTY BENCHMARK 477 BEING A 2"BRASS DISK STAMPED "SANTA CRUZ COUNTY SURVEYOR BM 477".
ELEVATION = 17.19 FEET NAVD88

THE CONTOUR INTERVAL IS 1 FOOT.

PLANNING SUBMITTAL

⚠ SANTA CRUZ COUNTY COMMENTS, APRIL 23, 2018

RICHARD J. IRISH
No. 45820
Exp. 12-31-18
CIVIL

RI Engineering, Inc.
303 Potrero St., Suite 42-202, Santa Cruz, CA 95060
831-425-3901 www.riengineering.com

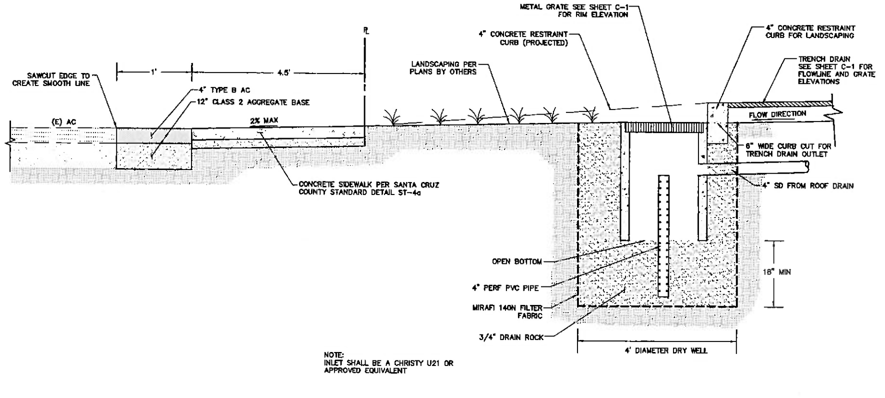
NEW RESIDENCE
FOR
JIM & SUE VAUDAGNA
379 BEACH DRIVE
APTOS, CALIFORNIA
APN 043-095-14

GRADING & DRAINAGE PLAN

project no.
17-133-1
date
SEPTEMBER 2018
scale
AS SHOWN
dwg name
CIVIL1.DWG

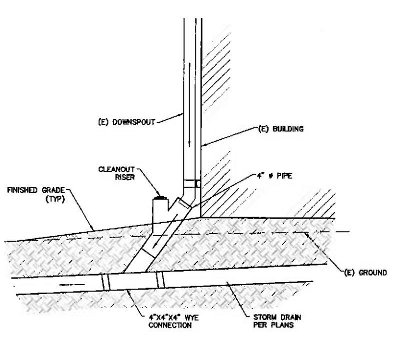
C-1

EXHIBI



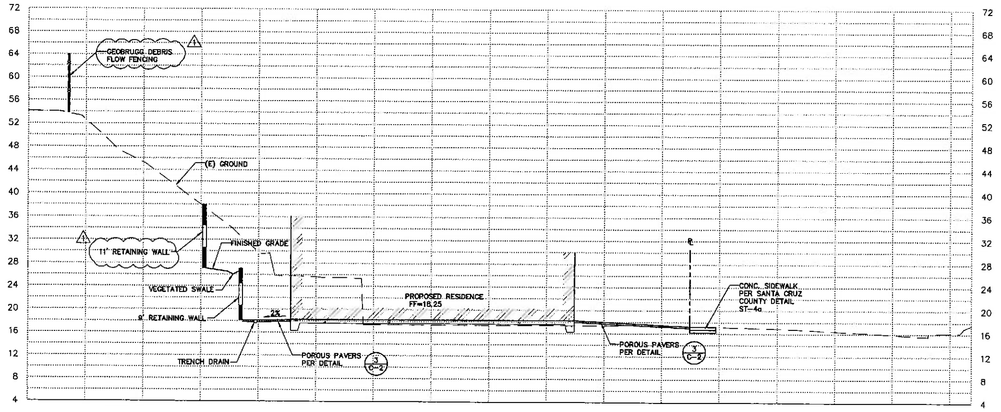
BUBBLER BOX AND DRY WELL/OUTLET DETAIL ① C-2

NTS



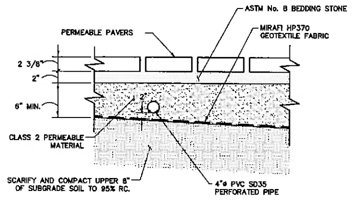
DOWNSPOUT CONNECTION DETAIL ② C-2

NTS



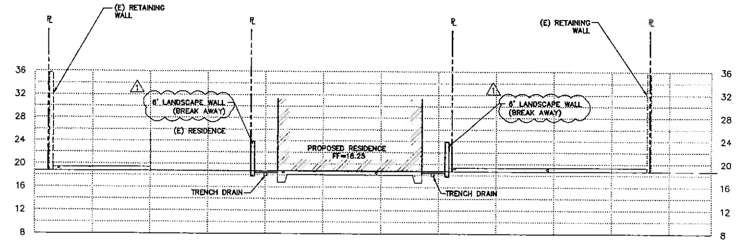
SECTION A–A

SCALE: 1"=10' HORIZONTAL, VERTICAL



PERVIOUS PAVER DRIVEWAY/WALKWAY SECTION ③ C-2

NTS



SECTION B–B

SCALE: 1"=10' HORIZONTAL, VERTICAL

GENERAL NOTES

1. THE CONTRACTOR SHALL MAKE A DETAILED AND THOROUGH STUDY OF THESE PLANS IN THEIR ENTIRETY PRIOR TO ANY WORK ON THE JOBSITE. THE CONTRACTOR IS TO VERIFY ALL EXISTING CONSTRUCTION CONDITIONS AND IS TO COORDINATE THESE DRAWINGS WITH ALL OTHER TRADE DISCIPLINES FOR THE COMPLETED WORK. THE CONTRACTOR IS ALSO TO UNDERSTAND THAT ANY FEATURE OF CONSTRUCTION NOT FULLY SHOWN OR DETAILED SHALL BE OF THE SAME TYPE AS SHOWN FOR SIMILAR CONDITIONS.

2. THE CONTRACTOR SHALL IMMEDIATELY REPORT TO THE ENGINEER ANY DISCREPANCY OCCURRING ON THE DRAWINGS OR FOUND IN HIS COORDINATION WORK. NO CHANGES IN APPROVED PLANS SHALL BE MADE WITHOUT PRIOR WRITTEN APPROVAL OF THE PROJECT ENGINEER AND THE COUNTY OF SANTA CRUZ DEPARTMENT OF PUBLIC WORKS.

3. ANY REQUEST FOR ALTERATIONS OR SUBSTITUTIONS MUST BE PRESENTED DIRECTLY TO THE PROJECT ENGINEER, ACCOMPANIED BY A DETAILED SKETCH, FOR REVIEW, BEFORE ANY APPROVAL WILL BE GIVEN AND BEFORE PROCEEDING WITH THE WORK. ABSOLUTELY NO ALTERATIONS OF THESE DOCUMENTS OF ANY KIND WILL BE APPROVED ON ANY SHOP DRAWINGS.

4. ALL CONSTRUCTION SHALL CONFORM TO THE APPLICABLE PROVISIONS OF THE STATE OF CALIFORNIA STANDARD SPECIFICATIONS (CALTRANS), LATEST EDITION, THE COUNTY OF SANTA CRUZ DESIGN CRITERIA AND ALL APPLICABLE CODES AND ORDINANCES.

5. THE LOCATIONS OF EXISTING UTILITIES AS SHOWN ON THE PLANS WERE OBTAINED FROM RECORD DRAWINGS AND MAY NOT REPRESENT TRUE LOCATION. IT IS THE CONTRACTOR'S RESPONSIBILITY TO VERIFY THE LOCATION AND DEPTH OF THESE UTILITIES PRIOR TO CONSTRUCTION, AND TO NOTIFY THE PROJECT ENGINEER IN CASE OF CONFLICT.

6. THE CONTRACTOR SHALL SECURE ALL REQUIRED CONSTRUCTION PERMITS FROM THE COUNTY OF SANTA CRUZ PRIOR TO THE START OF WORK.

7. CONTRACTOR AGREES THAT IN ACCORDANCE WITH GENERALLY ACCEPTED CONSTRUCTION PRACTICES, CONTRACTOR WILL BE REQUIRED TO ASSUME SOLE AND COMPLETE RESPONSIBILITY FOR JOB SITE CONDITIONS DURING THE COURSE OF CONSTRUCTION OF THE PROJECT, INCLUDING SAFETY OF ALL PERSONS AND PROPERTY; THAT THIS REQUIREMENT SHALL BE MADE TO APPLY CONTINUOUSLY AND NOT BE LIMITED TO NORMAL WORKING HOURS, AND CONTRACTOR FURTHER AGREES TO DEFEND, INDEMNIFY AND HOLD PROJECT ENGINEER HARMLESS FROM ANY AND ALL LIABILITY, REAL OR ALLEGED, IN CONNECTION WITH THE PERFORMANCE OF WORK ON THIS PROJECT, EXCEPTING LIABILITY RISING FROM THE SOLE NEGLIGENCE OF THE PROJECT ENGINEER.

8. GRADING SHALL BE DONE IN ACCORDANCE WITH THE RECOMMENDATIONS OF THE GEOTECHNICAL INVESTIGATIONS AND THE REQUIREMENTS OF THE COUNTY OF SANTA CRUZ.

9. THE CONTRACTOR SHALL NOTIFY THE COUNTY GRADING INSPECTOR, GEOTECHNICAL ENGINEER, THE CIVIL ENGINEER, & THE COUNTY CONSTRUCTION ENGINEER AT LEAST 48 HOURS PRIOR TO TO THE START OF CONSTRUCTION.

10. WORK SHALL BE LIMITED TO 8:00 A.M. TO 5:00 PM WEEKDAYS. NON-NOISE PRODUCING ACTIVITIES, SUCH AS INTERIOR PAINTING, SHALL NOT BE SUBJECT TO THIS RESTRICTION.

11. ALL FIGURE (FIG.) REFERENCES, UNLESS OTHERWISE SPECIFIED, REFER TO STANDARD DRAWINGS IN THE CURRENT EDITION OF THE "COUNTY OF SANTA CRUZ DESIGN CRITERIA."

12. NO LAND CLEARING, GRADING OR EXCAVATING SHALL TAKE PLACE BETWEEN OCTOBER 15 AND APRIL 15 UNLESS THE PLANNING DIRECTOR APPROVES A SEPARATE WINTER EROSION CONTROL PLAN.

13. BETWEEN OCTOBER 15 AND APRIL 15, EXPOSED SOIL SHALL BE PROTECTED FROM EROSION AT ALL TIMES. DURING CONSTRUCTION SUCH PROTECTION MAY CONSIST OF MULCHING AND/OR PLANTING OF NATIVE VEGETATION OF ADEQUATE DENSITY. BEFORE COMPLETION OF THE PROJECT, ANY EXPOSED SOIL ON DISTURBED SLOPES SHALL BE PERMANENTLY PROTECTED FROM EROSION.

14. THE CONTRACTOR SHALL APPLY WATER TO ALL EXPOSED EARTH SURFACES AT INTERVALS SUFFICIENT TO PREVENT AIRBORNE DUST FROM LEAVING THE PROJECT SITE. ALL EXPOSED EARTH SHALL BE WATERED DOWN AT THE END OF THE WORK DAY.

15. WHILE IN TRANSIT TO AND FROM THE PROJECT SITE, ALL TRUCKS TRANSPORTING FILL SHALL BE EQUIPPED WITH TARPS.

16. PURSUANT TO SECTIONS 16.40.040 AND 16.42.100 OF THE COUNTY CODE, IF AT ANY TIME DURING SITE PREPARATION, EXCAVATION, OR OTHER GROUND DISTURBANCE ASSOCIATED WITH THIS DEVELOPMENT, ANY ARTIFACT OR OTHER EVIDENCE OF AN HISTORIC ARCHEOLOGICAL RESOURCE OR A NATIVE AMERICAN CULTURAL SITE IS DISCOVERED. THE RESPONSIBLE PERSONS SHALL IMMEDIATELY CEASE AND DESIST FROM ALL FURTHER SITE EXCAVATION AND NOTIFY THE SHERIFF-CORONER IF THE DISCOVERY CONTAINS HUMAN REMAINS, OR THE PLANNING DIRECTOR IF THE DISCOVERY CONTAINS NO HUMAN REMAINS. THE PROCEDURES ESTABLISHED IN SECTIONS 16.40.010 AND 16.42.100, SHALL BE OBSERVED.

17. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE VERIFICATION OF CONSTRUCTION QUANTITIES PRIOR TO BIDDING ON ANY ITEM. QUANTITY REFERENCES SHOWN ON THESE PLANS. OR ENGINEER'S ESTIMATES ARE FOR ESTIMATING PURPOSES ONLY AND SHALL NOT BE CONSIDERED AS A BASIS FOR CONTRACTOR PAYMENT. CONSULTANT SHALL NOT BE RESPONSIBLE FOR ANY FLUCTUATIONS IN SUCH QUANTITIES AND ESTIMATES.

18. THE ENGINEER PREPARING THESE PLANS WILL NOT BE RESPONSIBLE FOR, OR LIABLE FOR, UNAUTHORIZED CHANGES TO, OR USES OF, THESE PLANS. ALL CHANGES MUST BE IN WRITING AND MUST BE APPROVED BY THE ENGINEER PRIOR TO CONSTRUCTION.

19. UNDERGROUND UTILITY LOCATIONS SHOWN ARE COMPILED FROM INFORMATION SUPPLIED BY UTILITY AGENCIES, AND ARE APPROXIMATE ONLY. CONTRACTOR SHALL VERIFY LOCATIONS OF AFFECTED UTILITY LINES PRIOR TO ANY TRENCHING OR EXCAVATING AND POTHOLE THOSE AREAS WHERE POTENTIAL CONFLICTS EXIST OR WHERE DATA IS OTHERWISE INCOMPLETE. FOR LOCATION, CALL USA 1-800-624-1444.

20. ANY EXISTING UTILITIES THAT ARE REQUIRED TO BE RELOCATED AS A PART OF THIS CONSTRUCTION SHALL BE RELOCATED AT THE OWNER'S EXPENSE.

EARTHWORK AND GRADING

1. WORK SHALL CONSIST OF ALL CLEARING, GRUBBING, STRIPPING, PREPARATION OF LAND TO BE FILLED, EXCAVATION, SPREADING, COMPACTION AND CONTROL OF FILL, AND ALL SUBSIDIARY WORK NECESSARY TO COMPLETE THE GRADING TO CONFORM TO THE LINES, GRADES, AND SLOPES, AS SHOWN ON THE APPROVED PLANS.

2. ALL GRADING OPERATIONS SHALL CONFORM TO SECTION 19 OF THE CALTRANS STANDARD SPECIFICATIONS, AND SHALL ALSO BE DONE IN CONFORMANCE WITH THE REQUIREMENTS OF THE COUNTY OF SANTA CRUZ. THE MOST STRINGENT GUIDELINE SHALL PREVAIL.

3. REFERENCE IS MADE TO THE GEOTECHNICAL INVESTIGATIONS BY PACIFIC CREST ENGINEERING, INC., ENTITLED "GEOTECHNICAL INVESTIGATION OF 379 BEACH DRIVE APTOS, CALIFORNIA FOR JIM & SUE VAUDAGNA, SARATOGA, CALIFORNIA," DATED NOVEMBER 30TH, 2017. THE CONTRACTOR SHALL MAKE A THOROUGH REVIEW OF THIS REPORT AND SHALL FOLLOW ALL RECOMMENDATIONS THEREIN. THE CONTRACTOR SHALL CONTACT PACIFIC CREST ENGINEERING. FOR ANY CLARIFICATIONS NECESSARY PRIOR TO PROCEEDING WITH THE WORK.

4. THE CONTRACTOR SHALL GRADE TO THE LINE AND ELEVATIONS SHOWN ON THE PLAN AND SHALL SECURE THE SERVICES OF A LICENSED LAND SURVEYOR OR REGISTERED CIVIL ENGINEER TO PROVIDE STAKES FOR LINE AND GRADE.

5. THE GEOTECHNICAL ENGINEER SHOULD BE NOTIFIED AT LEAST FOUR (4) DAYS PRIOR TO ANY SITE CLEARING AND GRADING OPERATIONS.

6. STRIPPED AREAS SHOULD BE SCARIFIED TO A DEPTH OF ABOUT 8", WATER-CONDITIONED TO BRING THE SOILS WATER CONTENT TO ABOUT 1%-3% ABOVE THE OPTIMUM, AND COMPACTED TO A DENSITY EQUIVALENT TO AT LEAST 90% OF THE MAXIMUM DRY DENSITY OF THE SOIL ACCORDING TO ASTM D1557 (LATEST EDITION). SUBGRADES AND AGGREGATE BASE ROCK FOR PAVEMENTS SHOULD BE COMPACTED TO A MINIMUM OF 95%.

7. ENGINEERED FILL SHOULD BE PLACED IN THIN LIFTS NOT EXCEEDING 8" IN LOOSE THICKNESS, MOISTURE CONDITIONED, AND COMPACTED TO AT LEAST 90% RELATIVE COMPACTION.

8. MATERIAL USED FOR ENGINEERED FILL SHALL MEET THE REQUIREMENTS OF THE AFOREMENTIONED REPORTS BY PACIFIC CREST ENGINEERING, INC..

9. IMPORTED FILL MATERIAL USED AS ENGINEERED FILL FOR THE PROJECT SHALL MEET THE REQUIREMENTS OF THE AFOREMENTIONED GEOTECHNICAL INVESTIGATION.

10. ALL FILL MATERIAL SHALL BE APPROVED BY THE PROJECT GEOTECHNICAL ENGINEER PRIOR TO JOBSITE DELIVERY AND PLACEMENT. NO EARTHWORK OPERATIONS SHALL BE PERFORMED WITHOUT THE DIRECT OBSERVATION AND APPROVAL OF THE GEOTECHNICAL ENGINEER.

11. BARE GROUND WITHIN 10' OF FOUNDATIONS SHALL BE SLOPED AWAY @ 5% MINIMUM OR 2% MINIMUM FOR PAVED SURFACES.

PLANNING SUBMITTAL

△ SANTA CRUZ COUNTY COMMENTS, APRIL 23, 2018

RICHARD J. IRISH
No. 45820
Exp. 12-31-18
CIVIL

RI Engineering, Inc.

303 Potrero St., Suite 42-202, Santa Cruz, CA 95060
831-425-3901 www.riengineering.com

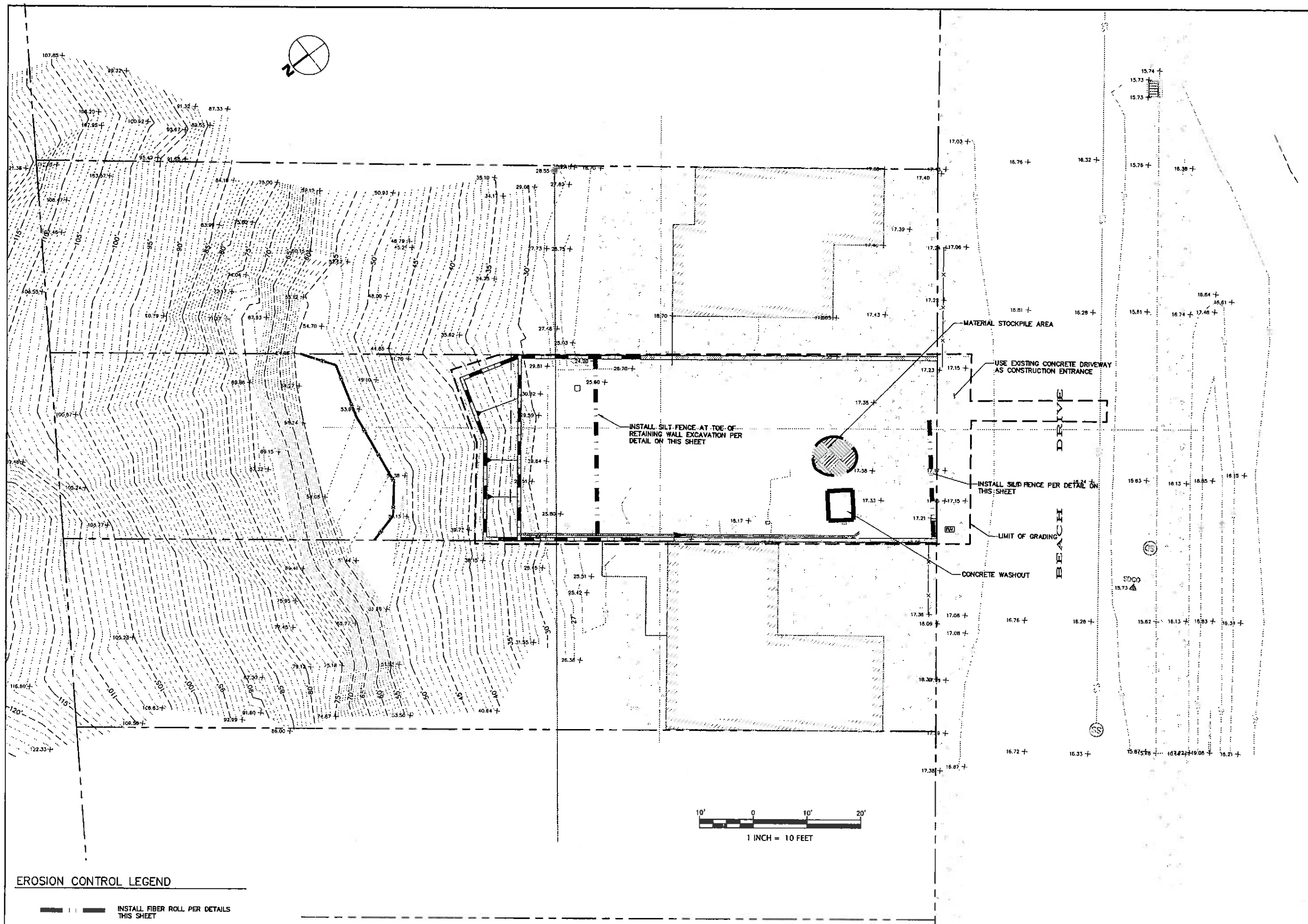
NEW RESIDENCE
FOR
JIM & SUE VAUDAGNA
379 BEACH DRIVE
APTOS, CALIFORNIA
APN 043-095-14

DETAILS

project no.
17-133-1
date
SEPTEMBER 2018
scale
AS SHOWN
dwg name
CIVIL1.DWG

C-2

EXHIBIT D



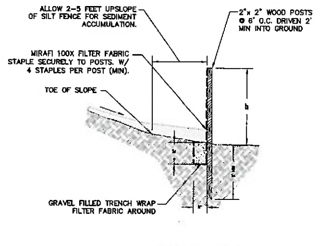
EROSION CONTROL LEGEND

▬ ▬ INSTALL FIBER ROLL PER DETAILS THIS SHEET

▨ PROPOSED STOCKPILE AREA

EXPOSED SLOPE MEASURES

1. COVER ALL EXPOSED SLOPES
2. STRAW 2 TONS/ACRE ON SLOPES ≤ 20% WITH SOIL BINDER
3. USE NORTH AMERICAN GREEN C125 OR EQUAL ON SLOPES >20%



SILT FENCE DETAIL

NTS

TOTAL AREA OF DISTURBANCE = 0.072 ACRES

SITE HOUSEKEEPING REQUIREMENTS

CONSTRUCTION MATERIALS

1. ALL LOOSE STOCKPILED CONSTRUCTION MATERIALS THAT ARE NOT ACTIVELY BEING USED (I.E. SOIL, SPOILS, AGGREGATE, FLY-ASH, STUCCO, HYDRATED LIME, ETC.) SHALL BE COVERED AND BERMED.
2. ALL CHEMICALS SHALL BE STORED IN WATERTIGHT CONTAINERS (WITH APPROPRIATE SECONDARY CONTAINMENT TO PREVENT ANY SPILLAGE OR LEAKAGE) OR IN A STORAGE SHED (COMPLETELY ENCLOSED).
3. EXPOSURE OF CONSTRUCTION MATERIALS TO PRECIPITATION SHALL BE MINIMIZED. THIS DOES NOT INCLUDE MATERIALS AND EQUIPMENT THAT ARE DESIGNED TO BE OUTDOORS AND EXPOSED TO ENVIRONMENTAL CONDITIONS (I.E. POLES, EQUIPMENT PADS, CABINETS, CONDUCTORS, INSULATORS, BRICKS, ETC.).
4. BEST MANAGEMENT PRACTICES TO PREVENT THE OFF-SITE TRACKING OF LOOSE CONSTRUCTION AND LANDSCAPE MATERIALS SHALL BE IMPLEMENTED.

LANDSCAPE MATERIALS

1. CONTAIN STOCKPILED MATERIALS SUCH AS MULCHES AND TOPSOIL WHEN THEY ARE NOT ACTIVELY BEING USED.
2. CONTAIN FERTILIZERS AND OTHER LANDSCAPE MATERIALS WHEN THEY ARE NOT ACTIVELY BEING USED.
3. DISCONTINUE THE APPLICATION OF ANY ERODABLE LANDSCAPE MATERIAL WITHIN 2 DAYS BEFORE A FORECASTED RAIN EVENT OR DURING PERIOD OF PRECIPITATION.
4. APPLY ERODABLE LANDSCAPE MATERIAL AT QUANTITIES AND APPLICATION RATES ACCORDING TO MANUFACTURE RECOMMENDATIONS OR BASED ON WRITTEN SPECIFICATIONS BY KNOWLEDGEABLE AND EXPERIENCED FIELD PERSONNEL.
5. STACK ERODABLE LANDSCAPE MATERIAL ON PALLETS AND COVERING OR STORING SUCH MATERIALS WHEN NOT BEING USED OR APPLIED.

VEHICLE STORAGE AND MAINTENANCE

1. MEASURES SHALL BE TAKEN TO PREVENT OIL, GREASE, OR FUEL TO LEAK IN TO THE GROUND, STORM DRAINS OR SURFACES WATERS.
2. ALL EQUIPMENT OR VEHICLES, WHICH ARE THE BE FUELED, MAINTAINED AND STORED ONSITE SHALL BE IN A DESIGNATED AREA FITTED WITH APPROPRIATE BMP'S.
3. LEAKS SHALL BE IMMEDIATELY CLEANED AND LEAKED MATERIALS SHALL BE DISPOSED OF PROPERLY.

WASTE MANAGEMENT

1. DISPOSAL OF ANY RINSE OR WASH WATERS OR MATERIALS ON IMPERVIOUS OR PERVIOUS SITE SURFACES OR INTO THE STORM DRAIN SYSTEM SHALL BE PREVENTED.
2. SANITATION FACILITIES SHALL BE CONTAINED (E.G., PORTABLE TOILETS) TO PREVENT DISCHARGES OF POLLUTANTS TO THE STORM WATER DRAINAGE SYSTEM OR RECEIVING WATER, AND SHALL BE LOCATED A MINIMUM OF 20 FEET AWAY FROM AN INLET, STREET OR DRIVEWAY, STREAM, RIPARIAN AREA OR OTHER DRAINAGE FACILITY.
3. SANITATION FACILITIES SHALL BE INSPECTED REGULARLY FOR LEAKS AND SPILLS AND CLEANED OR REPLACED AS NECESSARY.
4. COVER WASTE DISPOSAL CONTAINERS AT THE END OF EVERY BUSINESS DAY AND DURING A RAIN EVENT.
5. DISCHARGES FROM WASTE DISPOSAL CONTAINERS TO THE STORM WATER DRAINAGE SYSTEM OR RECEIVING WATER SHALL BE PREVENTED.
6. STOCKPILED WASTE MATERIAL SHALL BE CONTAINED AND SECURELY PROTECTED FROM WIND AND RAIN AT ALL TIMES UNLESS ACTIVELY BEING USED.
7. PROCEDURES THAT EFFECTIVELY ADDRESS HAZARDOUS AND NON-HAZARDOUS SPILLS SHALL BE IMPLEMENTED.
8. EQUIPMENT AND MATERIALS FOR CLEANUP OF SPILLS SHALL BE AVAILABLE ON SITE AND THAT SPILLS AND LEAKS SHALL BE CLEANED UP IMMEDIATELY AND DISPOSED OR PROPERLY; AND
9. CONCRETE WASHOUT AREAS AND OTHER WASHOUT AREAS THAT MAY CONTAIN ADDITIONAL POLLUTANTS SHALL BE CONTAINED SO THERE IS NO DISCHARGE INTO THE UNDERLYING SOIL AND ONTO THE SURROUNDING AREAS.

EROSION CONTROL MEASURES

1. EROSION IS TO BE CONTROLLED AT ALL TIMES ALTHOUGH SPECIFIC MEASURES SHOWN ARE TO BE IMPLEMENTED AT A MINIMUM BY OCTOBER 15.
2. UNLESS SPECIFIC MEASURES ARE SHOWN OR NOTED ON THIS PLAN, ALL COLLECTED RUNOFF SHALL BE CARRIED TO DRAINAGE COURSES IN LINED CONDUITS. DISCHARGE SHALL BE IN THE LOCATIONS SHOWN ON THE PLANS.
3. THE DESIRED END RESULT OF THESE MEASURES IS TO CONTROL SITE EROSION AND PREVENT SEDIMENT TRANSPORT OFF THE SITE. IT SHALL BE THE DEVELOPER'S RESPONSIBILITY TO SEE THAT ANY ADDITIONAL MEASURES NECESSARY TO MEET THIS GOAL ARE IMPLEMENTED. IF FAILED INSPECTIONS BY COUNTY STAFF SHOW THIS GOAL IS NOT BEING MET, ADDITIONAL MEASURES MAY BE REQUIRED.
4. ALL DISTURBED AREAS NOT CURRENTLY BEING USED FOR CONSTRUCTION SHALL BE SEEDED WITH THE FOLLOWING SEED MIXTURE:

   WINTER BARLEY 25#/ACRE
5. AFTER SEEDING, STRAW MULCH WILL BE APPLIED IN 4" (AVG.) LAYERS.
6. AMMONIUM PHOSPHATE FERTILIZER, 6-3-3, SHALL BE APPLIED AT A RATE OF 30 LBS. PER ACRE. ON SLOPES GREATER THAN 20% EROSION CONTROL BLANKET (NORTH AMERICAN GREEN) SHALL BE APPLIED.
7. SILT BARRIERS SHALL BE PLACED END TO END AND STAKED DOWN ALONG THE BOTTOM OF ALL GRADED SLOPES.

ALL EROSION CONTROL MEASURES INCLUDING BUT NOT LIMITED TO SILT FENCES, FIBER ROLLS AND SLOPE PROTECTION SHALL BE IN PLACE BY OCTOBER 15TH. THE ENGINEER OF RECORD SHALL INSPECT ONCE EROSION CONTROL MEASURES HAVE BEEN INSTALLED.

PLANNING SUBMITTAL

△1 SANTA CRUZ COUNTY COMMENTS, APRIL 23, 2018

RICHARD J. IRISH
No. 45820
Exp. 12-31-18
CIVIL

RI Engineering, Inc.
303 Potrero St., Suite 42-202, Santa Cruz, CA 95060
831-425-3901 www.riengineering.com

NEW RESIDENCE
FOR
JIM & SUE VAUDAGNA
379 BEACH DRIVE
APTOS, CALIFORNIA
APN 043-095-14

STORMWATER POLLUTION
CONTROL PLAN

project no.
17-133-1
date
SEPTEMBER 2018
scale
AS SHOWN
dwg name
CIVIL1.DWG

C-3

EXHIBIT D

Layout Tab Name: 24x36, Images: , Xrefs:
S:\PROJECTS\2017\17023 379 Beach Drive, Aptos\DWG\379 Beach Drive, Aptos C3D.dwg, Plotted By: paul, Plotted: May 11, 2017 – 3:02pm

NOTE:
A portion of the subject property is located within Zone VE with a base flood elevation of 24 feet (NAVD88) per FEMA Flood Insurance Rate Map;
Community Number 060353
Panel Number 0359 E
Map Revision Date May 16, 2012

Basis of Elevation
County Benchmark 477 being a 2"brass disk stamped "Santa Cruz County Surveyor BM 477". Elevation = 17.19 feet NAVD88
The contour interval is 1 foot.

Basis of Bearings
The basis of bearing for this map is S 51° 53' E between found monuments on the northeasterly line of Beach Drive per record map 024-M-26.

JOB NO.	17023
SHEET	SU-1
OF	2 SHEETS
A.P.N.	043-095-14
DATE	5-10-2017
DESIGN	
SCALE	1" = 10'
DRAWN	N. Pasquini

Topographic Map, The Lands Of:
Jim and Sue Vaudagna
379 Beach Drive, Aptos, CA 95003

HANAGAN LAND SURVEYING, INC.
305-C SOQUEL AVE., SANTA CRUZ, CA 95062
PHONE 831-469-3428

REVISION
APPROVED
Paul Hanagan LS 7797

PAUL J. HANAGAN
NO. 7797
EXP. 12-31-17

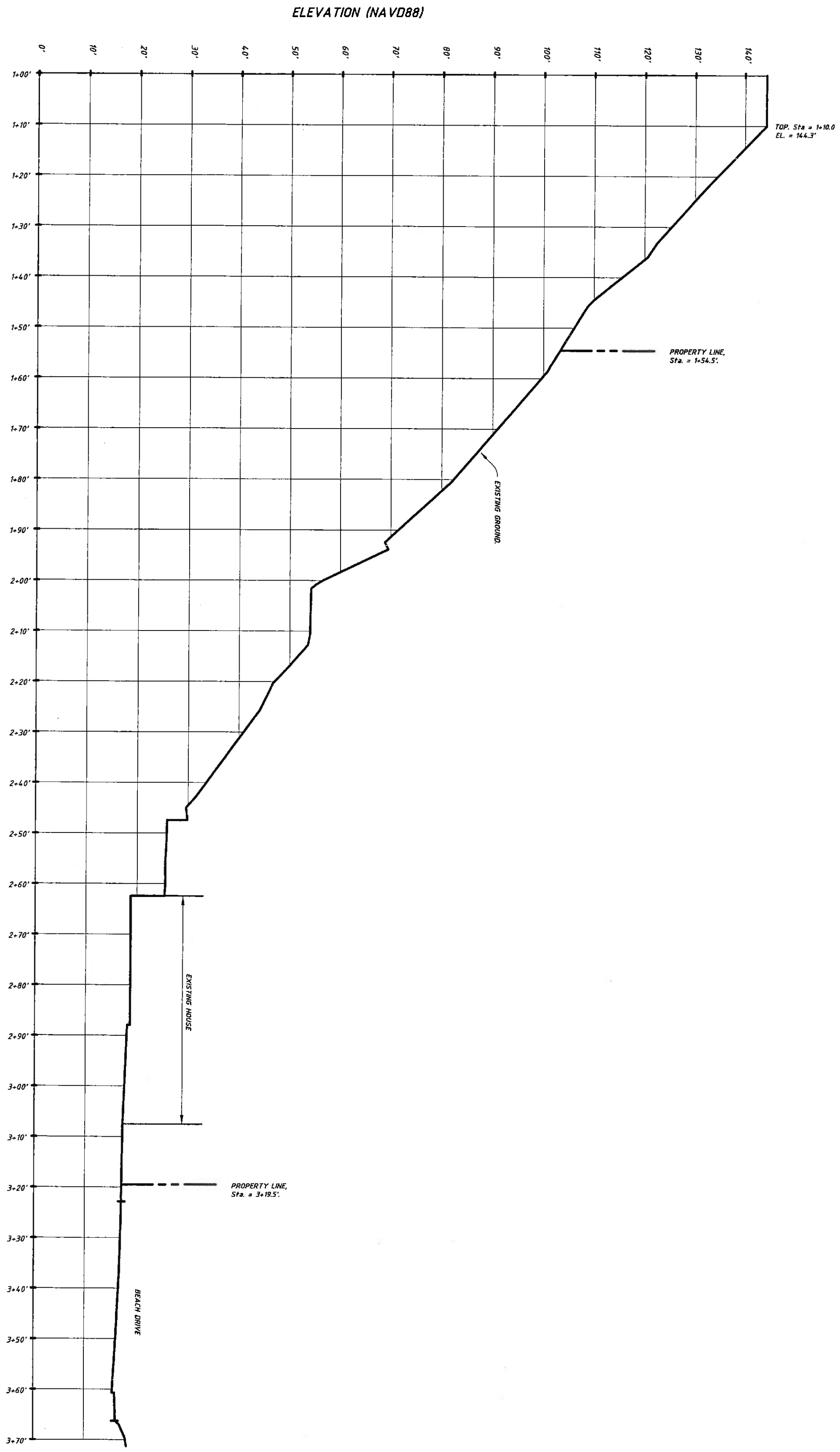
EXHIBIT D

Layout Tab Name: 24x36 PROFILE (2), Images: , Xrefs:
S:\PROJECTS\2017\17023 379 Beach Drive, Aptos\DWG\379 Beach Drive, Aptos C3D.dwg, Plotted By: paul, Plotted: May 11, 2017 – 3:03pm

SEE SHEET 1,

SECTION A

HORZ. SCALE: 1" = 10'
VERT. SCALE: 1" = 10'



Basis of Elevation

County Benchmark 477 being a 2"brass disk stamped "Santa Cruz County Surveyor BM 477".
Elevation = 17.19 feet NAVD88

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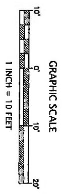


EXHIBIT D

JOB NO. 17023
SHEET SU-2 OF 2 SHEETS
A.P.N. 043-095-14
DATE 5-10-2017
DESIGN
SCALE 1" = 10'
DRAWN N. Pasquini

Topographic Map, The Lands Of:

Jim and Sue Vaudagna

379 Beach Drive, Aptos, CA 95003

HANAGAN LAND SURVEYING, INC.
305-C SOQUEL AVE., SANTA CRUZ, CA
95062
PHONE 831-469-3428

REVISION

APPROVED

Paul Hanagan LS 7797

PAUL J. HANAGAN
NO. 7797
EXP. 12-31-17

VAUDAGNA RESIDENCE

379 BEACH DRIVE
APTOS, CA 95003
A.P.N. 043-095-14

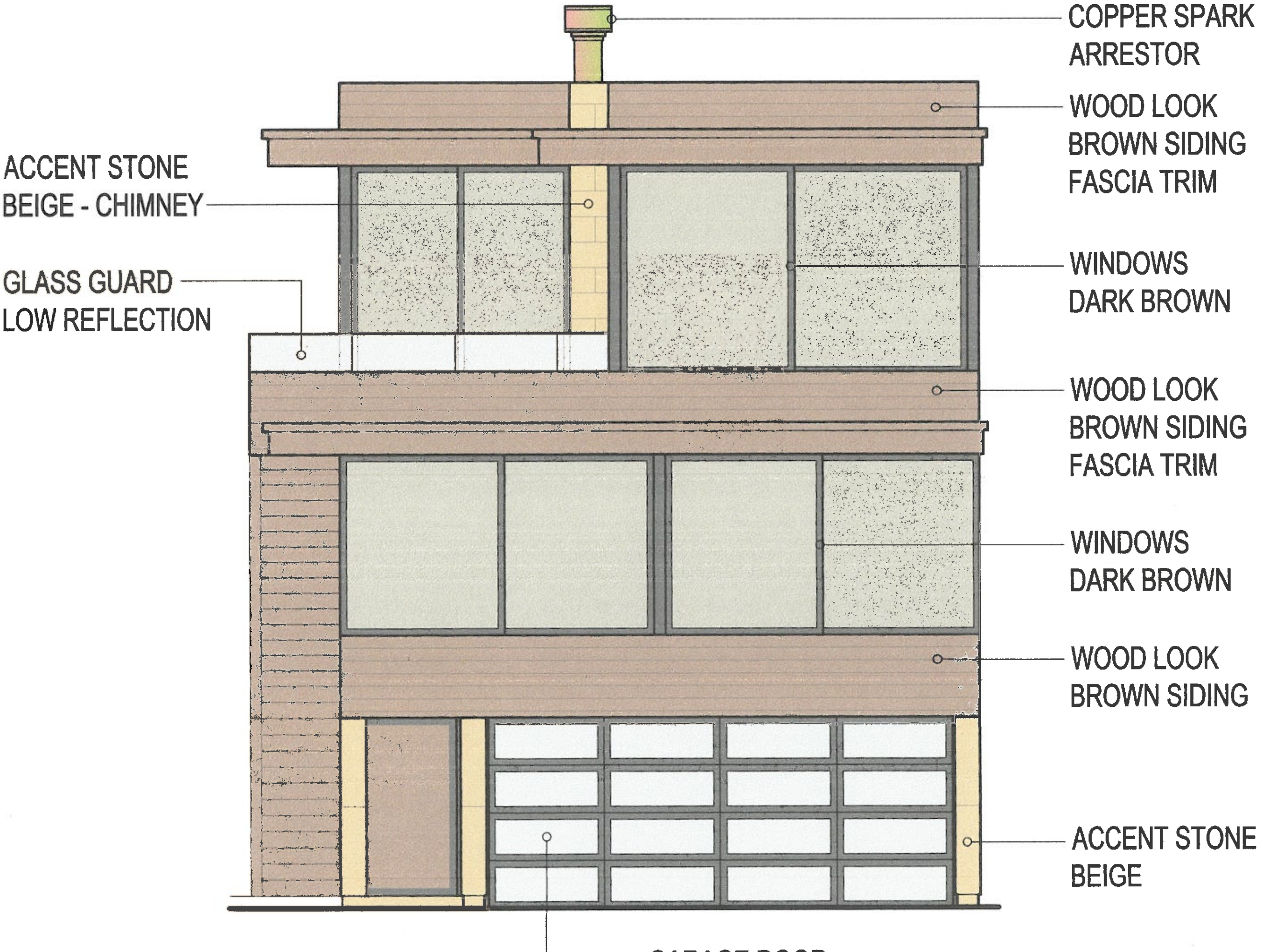
COLOR & MATERIALS

BRITTON
MATSON
ARCHITECTS

728 N BRANCIFORTE
SANTA CRUZ
CA 95062
831-425-0544

WOOD LOOK SIDING
BROWN

ACCENT STONE
BEIGE



PROPOSED FRONT
SOUTH ELEVATION

EXHIBIT D

Before

Vaudagna-Residence

APN 043-095-14

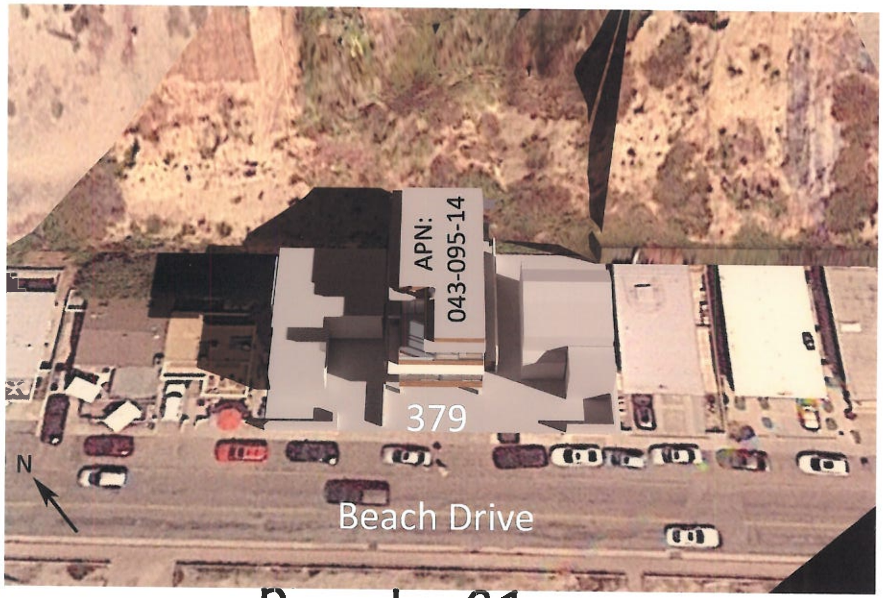
Proposed Vaudagna Residence

APN 043-095-14


nimatehouse
DIGITAL ARCHITECTURAL IMAGERY
N
Camera Position
APN: 043-095-14
379 Beach Dr

June 21 10am

June 21 2pm



December 21 10am

December 21 2pm

MATSON BRITTON ARCHITECTS

Shadow Study

Proposed Vaudagna Residence - A.P.N.: 043-095-14

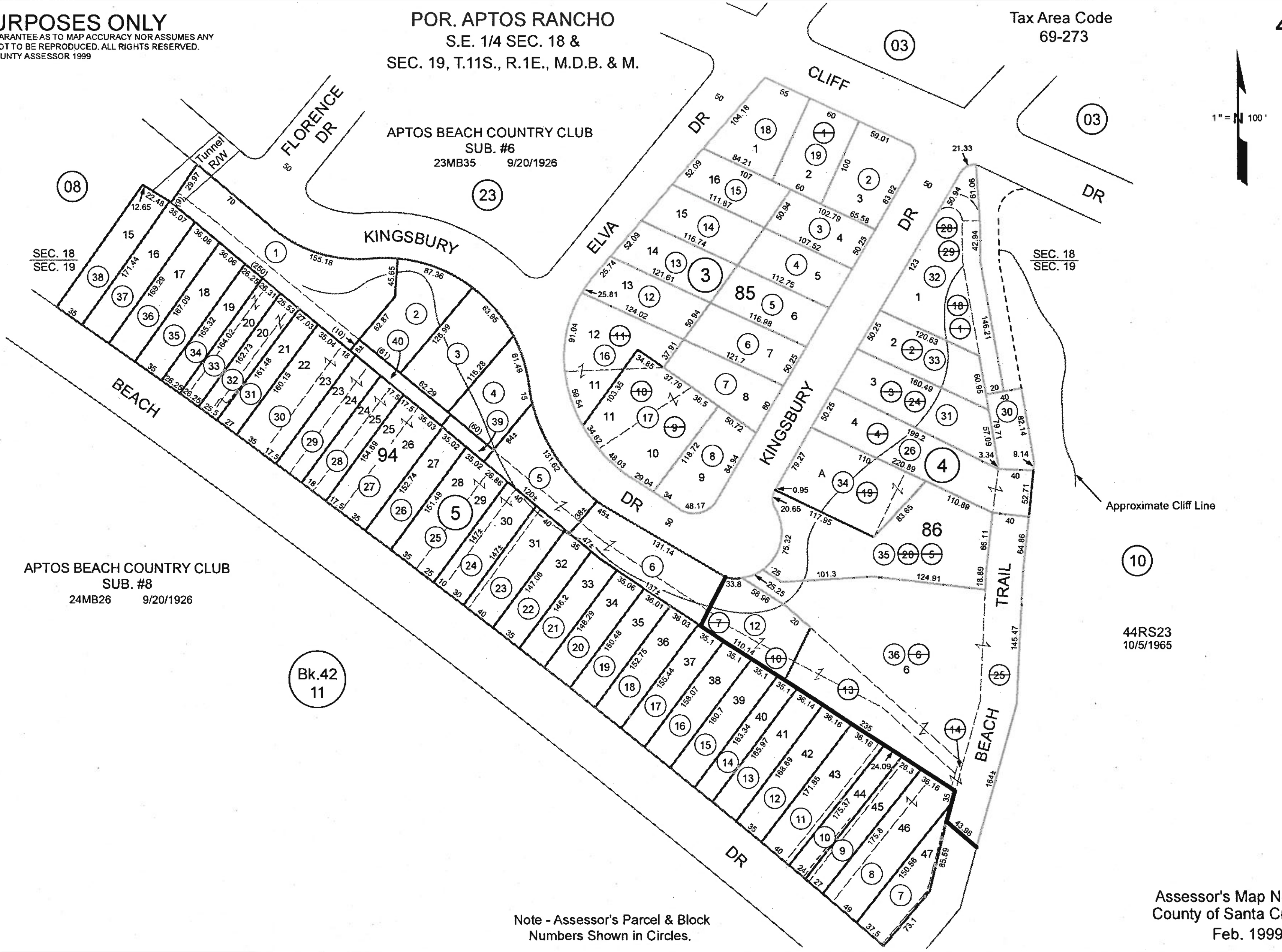
FOR TAX PURPOSES ONLY

POR. APTOS RANCHO
S.E. 1/4 SEC. 18 &
SEC. 19, T.11S., R.1E., M.D.B. & M.

Tax Area Code
69-273

43-09

EXHIBIT E



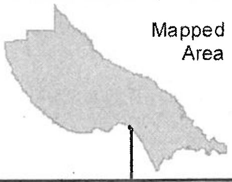
Note - Assessor's Parcel & Block Numbers Shown in Circles.

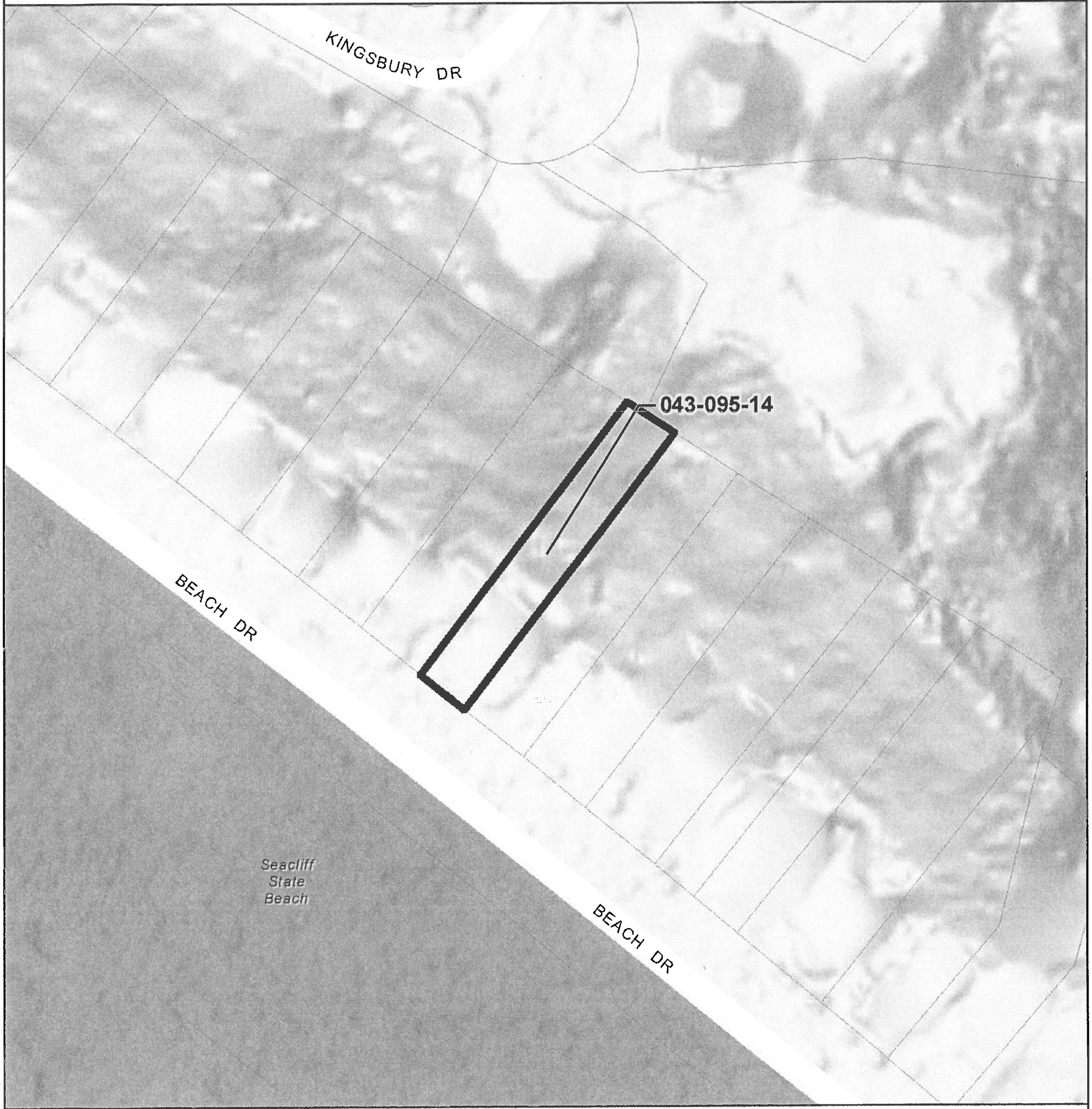
Assessor's Map No. 43-09
County of Santa Cruz, Calif.
Feb. 1999

Electronically Redrawn 2/10/99 rw
Rev. 2/10/99 (Por. to pg. 23) rw
Rev. 5/4/99 CB (Added MB ref's)
Rev. 5/9/00 CB (Added Blk line)
Rev. 5/25/01 mvm (changed page refs.)
Rev. 12/10/02 CB (2-0068226, Sp 4-32 & 33)
Rev. 3/31/05 DD (4-0089640, lba 4-34 & 35)
Rev. 6/3/15 AR (14-0019936, comb. 4-36)

SANTA CRUZ COUNTY PLANNING DEPARTMENT

Parcel Location Map





Parcel: 04309514

▭ Study Parcel
☐ Assessor Parcel Boundary
▨ Existing Park

N

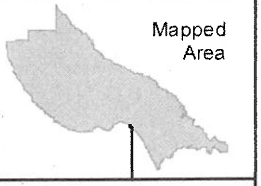
0 20 40
Feet

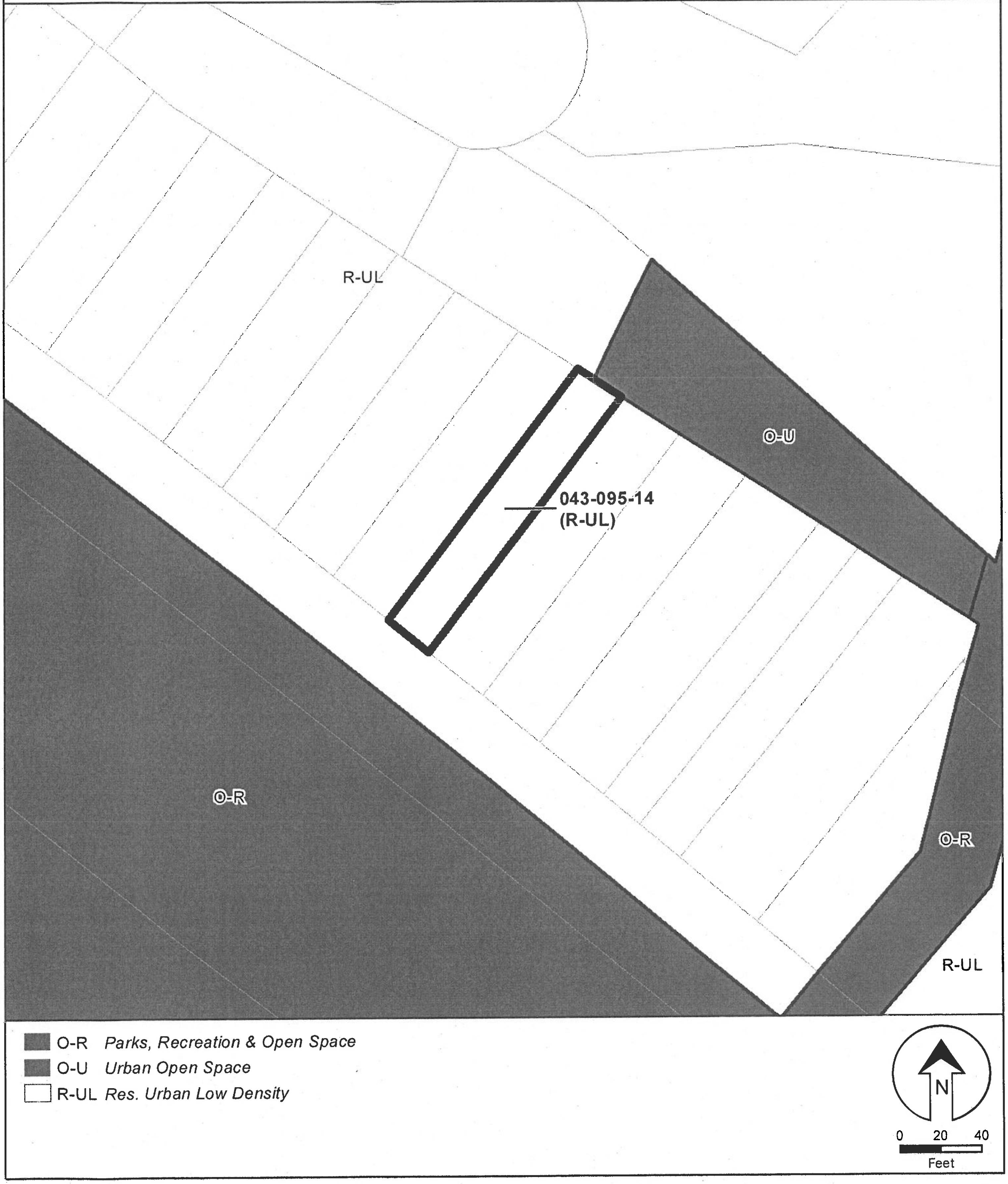
Map printed: 2 Mar. 2020

EXHIBIT E

SANTA CRUZ COUNTY PLANNING DEPARTMENT

Parcel General Plan Map



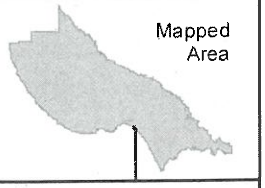


▨ O-R *Parks, Recreation & Open Space*
▨ O-U *Urban Open Space*
☐ R-UL *Res. Urban Low Density*

EXHIBIT E

SANTA CRUZ COUNTY PLANNING DEPARTMENT

Parcel Zoning Map



▨ PR *Parks, Recreation, & Open Space*

☐ R-1 *Single-Family Residential*

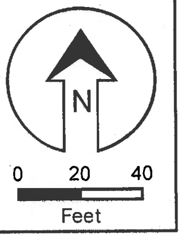


EXHIBIT E

GEOTECHNICAL INVESTIGATION

379 BEACH DRIVE
APTOS, CALIFORNIA

FOR
JIM AND SUE VAUDAGNA
SARATOGA, CALIFORNIA

Pacific Crest
ENGINEERING INC

CONSULTING GEOTECHNICAL ENGINEERS

1738-SZ70-B44
NOVEMBER 2017
www.4pacific-crest.com

EXHIBIT F

Pacific Crest
ENGINEERING INC

GEOTECHNICAL | ENVIRONMENTAL | CHEMICAL | MATERIAL TESTING | SPECIAL INSPECTIONS

November 30, 2017

Project No. 1738-SZ70-B44

Jim and Sue Vaudagna
19501 Scotland Drive
Saratoga, CA 95070

Subject: **Geotechnical Investigation – Design Phase**
379 Beach Drive
APN 043-095-14
Aptos, California

Dear Mr. and Mrs. Vaudagna,

In accordance with your authorization, we have performed a geotechnical investigation for your proposed residence at 379 Beach Drive in Aptos, California.

The accompanying report presents our findings, conclusions and recommendations for the subject project. If you have any questions concerning the information presented in this report, please call our office.

Very truly yours,

PACIFIC CREST ENGINEERING INC.

Prepared by:

Soma Goresky

REGISTERED PROFESSIONAL ENGINEER
SOMA GORESKY
2252
EXP. 6/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

Soma Goresky
Associate Engineer
GE 2252
Expires 6/30/19

Reviewed by:

Elizabeth Mitchell

REGISTERED PROFESSIONAL ENGINEER
ELIZABETH MAE MITCHELL
2718
Exp. 12/31/18
GEOTECHNICAL
STATE OF CALIFORNIA

Elizabeth M. Mitchell, GE
President/Principal Geotechnical
GE 2718
Expires 12/31/18

Copies: 3 to Client

EXHIBIT F

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I. INTRODUCTION

PURPOSE AND SCOPE

This report describes the geotechnical investigation and presents our conclusions and recommendations for a proposed residence at 379 Beach Drive in Aptos, California.

Our scope of services for this project has consisted of:

1. Site reconnaissance to observe the existing conditions.
2. Review of the following published maps:
    - Geologic Map of Santa Cruz County, California, Brabb, 1997.
    - Preliminary Map of Landslide Deposits in Santa Cruz County, California, Cooper-Clark and Associates, 1975.
    - Map Showing Geology and Liquefaction Potential of Quaternary Deposits in Santa Cruz County, California, Dupré, 1975.
    - Map Showing Faults and Their Potential Hazards in Santa Cruz County, California, Hall, Sarna-Wojcicki, Dupré, 1974.
    - Geographic Information System – Santa Cruz County, "GISWEB Interactive Mapping Application" http://gis.co.santa-cruz.ca.us/internet/wwwgisweb/ viewer.htm
3. The drilling and logging of 2 test borings.
4. Laboratory analysis of retrieved soil samples.
5. Engineering analysis of the field and laboratory test results.
6. Review of "Preliminary Geological Findings and Recommendations" prepared by Zinn Geology, dated November 12, 2017.
7. Discussion regarding the geologic hazards, proposed development and alternative mitigation measures with Matson Britton Architects and the project geologist Mr. Erik Zinn.
8. Preparation of this report documenting our investigation and presenting geotechnical recommendations for the design and construction of the project.

PROJECT LOCATION

The subject site is located at 379 Beach Drive in Aptos, California. Please refer to the Regional Site Map, Figure No. 1, in Appendix A for the general vicinity of the project site, which is located by the following coordinates:

Latitude = 36.965190 degrees
Longitude = -121.898822 degrees

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PROPOSED IMPROVEMENTS

A proposed residence will replace an existing residence using essentially the same building footprint. We understand that the new residence will be a three story structure with a garage on the first floor and living space on the 2nd and 3rd floors. Details of construction are not available at this time but we anticipate that the building will have a first floor elevation roughly equal to existing grade (elev. 19) and the 2nd story being roughly 10 feet above (elev. 29). Building loads are expected to be typical of a residential structure of this size.

We understand that a FEMA base flood elevation (BFE) of 21 feet NAVD has been calculated for this site and thus the front portion of the building site is mapped as being susceptible to coastal erosion and flooding. The understructure area below the BFE is assumed to be inundated by coastal flooding at some point during the next 100 years, and will be used for storage and parking only. The lower story will be enclosed by breakaway walls to allow the projected coastal flooding to flow through and under the structure.

II. INVESTIGATION METHODS

FIELD INVESTIGATION

One, 4-inch diameter test boring was drilled on the hillside behind the house on May 17, 2017 and a second boring was drilled in front of the existing garage on May 24, 2017. The approximate location of the test borings is shown on Figure No. 2, in Appendix A. The drilling method B-1 was a portable "minute man" rig and B-2 was performed with a truck mounted drill rig. Both methods employed continuous flight augers. An engineer from Pacific Crest Engineering Inc. was present during the drilling operations to log the soil encountered and to choose sampler type and locations.

Relatively undisturbed soil samples were obtained at various depths by driving a split spoon sampler 18 inches into the ground. For B-1 a 70 pound, hand operated hammer was used to drive the sampler due to the remote location of the drill hole. For B-2 this samplers were driven by dropping a 140 pound hammer a vertical height of 30 inches using a wire winch. The number of blows required to drive the sampler each 6 inch increment and the total number of blows required to drive the last 12 inches was recorded by the field engineer. The outside diameter of the samplers used was 3 inch or 2 inch and is designated on the Boring Logs as "L" or "T", respectively.

The field blow counts in 6 inch increments are reported on the Boring Logs adjacent to each sample as well as the standard penetration test data. All standard penetration test data has been normalized to a 2 inch O.D. sampler and is reported on the Boring Logs as SPT "N" values. The normalization method used was derived from the second edition of the Foundation Engineering Handbook (H.Y. Fang, 1991). We note that no correction for the 70 lb. hammer has been incorporated into the reported "N" values on B-1.

The soils encountered in the borings were continuously logged in the field and visually described in accordance with the Unified Soil Classification System (ASTM D2488) as described in the Boring Log Explanation, Figures No. 3 and 4, in Appendix A. The soil classification was verified upon completion of laboratory testing in accordance with ASTM D2487.

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Appendix A contains the site plan showing the locations of the test borings, our borings logs and an explanation of the soil classification system used. Stratification lines on the boring logs are approximate as the actual transition between soil types may be gradual.

LABORATORY TESTING

The laboratory testing program was developed to aid in evaluating the engineering properties of the materials encountered at the site. Laboratory tests performed include:

- Moisture Density relationships in accordance with ASTM D2937.
- Gradation testing in accordance with ASTM D1140 and D422.

The results of the laboratory testing is presented on the boring logs opposite the sample tested and/or presented graphically in Appendix A.

III. FINDINGS AND ANALYSIS

GEOLOGIC SETTING

For a detailed presentation of the geologic setting please refer to the geologic report for the site prepared by Zinn Geology (forthcoming). A brief summary is presented below.

The property is located at the base of a coastal bluff. The bluff and the entire property is mapped as being underlain by Purisima sandstone. Immediately upslope and northeast of the site, at the top of the bluff, the area is mapped as coastal marine terrace. Immediately southwest of the site the area is underlain by beach sand.

The bedrock encountered during our field investigation is consistent with the mapped bedrock description and the native soils overlaying the bedrock are consistent with beach sands as well as residual soils typically derived from the Purisima formation.

SURFACE CONDITIONS

The subject property is a narrow, 35 foot wide lot bordered by Beach Drive and the ocean beach to the southwest, a 120 foot high coastal bluff to the northeast and developed narrow lots to the remaining two sides. Beach houses in this area all have similarly narrow lots and existing buildings are separated horizontally by as little as 5 feet. Existing ground surface elevation across the building site ranges between 17 and 25 feet.

Based on the survey of the site prepared by Hanagan Land Surveying (titled Jim and Sue Vaudagna, dated May 10, 2017) the upper 40 feet of the bluff stands at inclinations between 45 and 50 degrees. Pacific Crest Engineering was involved in the observation and testing during installation of a Geobrugg Tecco slope stabilization system that covers the upper portion of the bluff beyond the subject property. This work was completed in 2016. Below this, natural slopes stand at between 33 and 40 degrees. It appears that a roughly 10 foot wide bench has been graded about 1/3 of the way up the slope by cutting about 6 feet and filling about 3 to 4 feet. Cut slopes on the uphill side of the bench stand at about 60 degrees and fills slopes below stand at about 45 degrees.

The bluff is partially vegetated with portions that have undergone recent erosion/sliding exposing bare soil.

SUBSURFACE CONDITIONS

Our subsurface exploration consisted of a single test boring (B-1) drilled with a portable rig at the base of the coastal bluff behind the building and one boring (B-2) drilled between the garage and Beach Drive. The borings extended between 28½ and 30 feet below existing grade. The soil profiles and classifications, laboratory test results and groundwater conditions encountered for each test boring are presented in the Logs of Test Borings, in Appendix A. The general subsurface conditions are described below.

Subsurface conditions encountered within B-1 consisted of about 19 feet fill/colluvial soil composed of a silty sand to sand with silt. The upper 7 feet of soil is loose in density and then becomes medium dense. This material is underlain by Purisima sandstone bedrock at about 19 feet below ground surface. The sandstone is fine grained and very soft in rock hardness.

B-2 encountered about 4 feet of sand and clayey sand which we interpret to be a fill material. From 4 feet to 16½ feet below ground surface we encountered a loose to medium dense clean beach sand. Purisima sandstone bedrock similar in composition to that encountered in B-1 underlies the sand at 16½ feet.

Groundwater was encountered at 22 feet in B-1 and at 12 feet in B-2. It should be noted that the groundwater level was not allowed to stabilize for more than a few hours; therefore, the actual groundwater level may be higher or lower than initially encountered. The groundwater conditions described in this report reflect the conditions encountered during our drilling investigation in May 2017 at the specific locations drilled. Groundwater levels at this site are primarily influenced by fluctuating ocean tides and therefore can be expected to vary widely. It is reasonable to expect that coastal flooding and seasonal high tides may cause the groundwater level to rise to the ground surface at certain times during the design life of the project.

Please refer the Logs of Test Borings in Appendix A, for a more detailed description of the subsurface conditions encountered in each of our test borings at the subject site.

FAULTING AND SEISMICITY

Faulting

Mapped faults which have the potential to generate earthquakes that could significantly affect the subject site are listed in Table No. 1. The fault distances are approximate distances based the U.S. Geological Survey and California Geological Survey, Quaternary fault and fold database, accessed on March 2017 from the USGS website (http://earthquake.usgs.gov/hazards/qfaults/) and overlaid onto Google Earth.

Table No. 1 - Distance to Significant Faults

Fault Name	Distance (miles)	Direction
Zayante-Vergeles	5½	East
San Andreas	7½	Northeast
Sargent	11½	Northeast
Monterey Bay-Tularcitos	12	Southwest
San Gregorio	16	West

<u>Seismic Shaking and CBC Design Parameters</u>

Due to the proximity of the site to active and potentially active faults, it is reasonable to assume the site will experience high intensity ground shaking during the lifetime of the project. Structures founded on thick soft soil deposits are more likely to experience more destructive shaking, with higher amplitude and lower frequency, than structures founded on bedrock. Generally, shaking will be more intense closer to earthquake epicenters. Thick soft soil deposits large distances from earthquake epicenters, however, may result in seismic accelerations significantly greater than expected in bedrock.

Selection of seismic design parameters should be determined by the project structural designer. The site coefficients and seismic ground motion values shown in the table below were developed based on CBC 2016 incorporating the ASCE 7-10 standard, and the project site location.

Table No. 2 - 2016 CBC Seismic Design Parameters [1]

Seismic Design Parameter	ASCE 7-10 Value
Site Class[3]	D
Spectral Acceleration for Short Periods	$S_S = 1.5g$
Spectral Acceleration for 1-second Period	$S_1 = 0.6g$
Short Period Site Coefficient	$F_a = 1.0$
1-Second Period Site Coefficient	$F_v = 1.5$
MCE Spectral Response Acceleration for Short Period	$S_{MS} = 1.5g$
MCE Spectral Response Acceleration for 1-Second Period	$S_{M1} = 0.9g$
Design Spectral Response Acceleration for Short Period	$S_{DS} = 1.0g$
Design Spectral Response Acceleration for 1-Second Period	$S_{D1} = 0.60g$
Seismic Design Category [2]	D

Note 1: Design values have been obtained by using the Ground Motion Parameter Calculator available on the USGS website at http://earthquake.usgs.gov/hazards/designmaps/usdesign.php.

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Note 2: The Seismic Design Category assumes a structure with Risk Category I, II or III occupancy as defined by Table 1604.5 of the 2016 CBC. Pacific Crest Engineering Inc. should be contacted for revised Table 2 seismic design parameters if the proposed structure has a different occupancy rating than that assumed.
Note 3: The site would normally be Site Class F because it is underlain by potentially liquefiable soils If the fundamental period of vibration of the structures is less than 0.5 seconds, the site class can be determined by assuming there is no liquefaction (ASCE 7-05 Section 20.3.1). Therefore, Site Class D was selected for the project site.

The recommendations of this report are intended to reduce the potential for structural damage to an acceptable risk level, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs. It should be assumed that exterior improvements such as pavements or sidewalks may need to be repaired or replaced following strong seismic shaking.

GEOTECHNICAL HAZARDS

A quantitative analysis of geotechnical hazards was beyond our scope of services for this project. In general however, the geotechnical hazards associated with the project site include seismic shaking (discussed above), ground surface fault rupture, coastal flooding and erosion, liquefaction, lateral spreading, landsliding and expansive soils. A qualitative discussion of these hazards is presented below.

Ground Surface Fault Rupture

Pacific Crest Engineering Inc. has not performed a specific investigation for the presence of active faults at the project site. Based upon our review of the Santa Cruz County GIS Hazard Maps, the project site is not mapped within a fault hazard zone.

Ground surface fault rupture typically occurs along the surficial traces of active faults during significant seismic events. Since the nearest known active, or potentially active fault trace is mapped approximately 5½ miles from the site, it is our opinion that the potential for ground surface fault rupture to occur at the site should be considered low.

Coastal Flooding and Erosion

FEMA has established a base flood elevation (BFE) for this site of 21 feet NAVD, and based on this the habitable portions of the structure must be placed above this elevation. The lower portion of the structure lying below the BFE may be used for storage and parking, however the projected coastal flood must be allowed to flow through the structure with minimum obstruction. Areas located below the BFE may be enclosed by breakaway walls. It should be understood that the contents therein, including parked vehicles, may be inundated by coastal flooding and lost, damaged or destroyed.

Consistent with the recommendations of the project geologist, we anticipate that the beach sand layer will be completely scoured down to the bedrock platform at roughly +1.5 feet mean seal level (NAVD 88). As a result, foundation design should consider that the lower story of the building will be inundated and the upper 16 feet of the foundation system will lose all vertical and lateral support.

Based on hydraulic data presented in the "Intermediate Data Submittal #3" (prepared by BakerAECOM, dated 11/24/14), on the preliminary geologic report from Zinn Geology (dated 11/12/17) and using design procedures presented in the Coastal Construction Manual (FEMA P-55, August 2011), the following parameters were used in our flood load analysis:

FEMA Base Flood Elevation (BFE): (Transect 76, includes wave setup)	21 feet NAVD 88
Lowest Eroded Ground Elevation:	1.5 feet NAVD 88
FEMA Stillwater Elevation	14.7 feet NAVD 88
Design Stillwater Depth (ds):	13.2 feet
Flood Velocity:	21 feet per second

Based upon FEMA P-55 our geotechnical recommendations for design flooding, wave and debris forces are provided in the Recommendations section of this report

Liquefaction and Lateral Spreading

Liquefaction is a phenomenon that can occur in saturated soil that has restricted drainage and is subject to seismic shaking. Liquefaction occurs when the soil grains are cyclically accelerated such that they begin to loose contact, allowing pressurized pore water to flow between soil particles. The soil, which derives its strength from point-to-point contact between grains, can become fluidized, resulting in significantly lower shear strengths. When the cyclic accelerations cease, the water pressure dissipates and the soil grains settle, regaining contact. Settlement can be differential due to the presence of non-homogeneous earth materials and due to differential densification and dewatering processes. Liquefaction can result in bearing failure and differential ground settlement, which can be highly damaging to structures, pavements and utilities.

Based on our field and laboratory data we infer that the beach sand that extends from the ground surface down to the bedrock platform (roughly 16 feet below ground surface) is highly liquefiable. However, for liquefaction to occur these materials would need to be submerged below water (such as would occur during a winter storm) at the same time as a major seismic event. There is a lower probability that high ground water conditions would occur simultaneously with a major earthquake and so the hazard of liquefaction can also be inferred to be somewhat lower.

Further evaluation of liquefaction hazards are not warranted for this site as the coastal flooding and erosion requirements dictate the entire beach sand layer be ignored for soil support. Design for these conditions essentially addresses potential impacts due to soil liquefaction.

Landsliding

The project geologist has identified a strong possibility for debris flows and/or shallow earth flows from the nearby bluff to impact the subject residence. These landsliding hazards can be divided into two basic categories: those originating on the subject property and those originating on the neighboring properties. The preliminary findings provided by Zinn Geology state debris flow volumes for a single event could be on

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the order of 260 cubic yards. Flows could move with a peak velocity of 38 feet per second and be 3 to 6 feet in height, with a deposition height of up to 8 feet.

Several options for reducing debris flow hazards were discussed with the project team including stabilization of the slope, installation of debris flow barriers and designing the lower story of the house with "break away" walls allowing debris flows to flow through the non-habitable first floor area. This latter option for reducing the hazard of debris flow at the site (break away first story walls) was selected by the project owner. For further discussion regarding how the risk of damage and loss of life due to debris flows can be reduced please refer to the following section.

Expansive Soils

The subject site is underlain by coarse grained soils that in our opinion have a low expansion potential.

IV. DISCUSSION AND CONCLUSIONS

GENERAL

1. The results of our investigation indicate that the proposed residential development is feasible from a geotechnical engineering standpoint, provided our recommendations and those provided by Zinn Geology are included in the design and construction of the project.

2. At the time we prepared this report, the grading plans had not been completed and the structure foundation details had not been finalized. We request an opportunity to review these items during the design stages to determine if supplemental recommendations will be required.

3. The structural design for the residence should include the guidelines outlined in the 2011 FEMA Coastal Construction Manual.

4. Pacific Crest Engineering Inc. should be notified at least four (4) working days prior to any site clearing and grading operations on the property in order to observe the stripping and disposal of unsuitable materials, and to coordinate this work with the grading contractor. During this period, a pre-construction conference should be held on the site, with at least the client or their representative, the grading contractor, a County representative and one of our engineers present. At this meeting, the project specifications and the testing and inspection responsibilities will be outlined and discussed.

5. Field observation and testing must be provided by a representative of Pacific Crest Engineering Inc., to enable them to form an opinion as to the degree of conformance of the exposed site conditions to those foreseen in this report, the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. Any work related to grading or foundation excavation that is performed without the full knowledge and direct observation of Pacific Crest Engineering Inc., the Geotechnical Engineer of Record, will render the recommendations of this report invalid, unless the Client hires a new Geotechnical Engineer who agrees to take over complete responsibility for this report's findings, conclusions and recommendations. The new Geotechnical Engineer must agree to prepare a Transfer of Responsibility letter. This may require

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additional test borings and laboratory analysis if the new Geotechnical Engineer does not completely agree with our prior findings, conclusions and recommendations.

PRIMARY GEOTECHNICAL CONSIDERATIONS

6. Based upon the results of our investigation, it is our opinion that the primary geotechnical issues associated with the design and construction of the proposed project at the subject site are the following:

a. *Coastal Flooding and Scour*: The habitable portion of the residence will be elevated above the FEMA BFE of 21 feet NAVD. The lower portion of the residence below the BFE will be enclosed by breakaway walls and used only for parking and storage. The area of the property below the BFE can be expected to be inundated by coastal flooding and impacted by wave forces and heavy debris. Flooding events could impact the first floor contents and all adjacent improvements with all contents being lost, damaged or destroyed. Future occupants of the property should be informed of the coastal flooding hazard and the potential for loss of items below the BFE, including parked vehicles. Damage to surrounding patios, decks, etc. should also be anticipated.

b. *Debris Flow Hazards:* The lower story and surrounding area may also be subject to rapidly moving debris flows originating from the bluff above. Debris flows could be up to 8 feet in thickness with up to 260 cubic yards of material flowing rapidly off of the hillside and impacting the lower story of the house and the surrounding area. In order to accommodate a flow through path for the anticipated debris we understand that the following design measures will be incorporated into the project plans:

- The northeast wall of the proposed house will be sited in essentially in the same location as the existing house and the entire first story will be "uninhabitable" space constructed with "break away" walls.
- The second habitable floor will be at roughly elevation 29 feet - well above the expected maximum height of debris (8 feet).
- The back yard will be excavated down to about elevation 19 and a new retaining wall will be constructed about 7 feet northeast of the existing retaining wall (see Figure 2). This wall is expected to be about 15 feet in height. This proposed grading will allow storage of a good portion of the design debris flow volume as well as providing a clear flow path off of the slope and down through the first story.

All occupants should be aware that there is still a high hazard of debris flows occurring at the site. The design measures and recommendations provided in this report will lower the risk but not the hazard of occurrence itself. Damage to improvements and contents in the first floor are within the debris flow path should be expected. The expected volume and height assumptions for debris flows provided in the geologic report should be incorporated into the project design and made known to the owner and all occupants.

c. *Foundation Design for Geologic Hazards:* The new residential structure will be supported by a grid foundation with drilled piers bearing into competent sandstone bedrock. The beach sand stratum overlying the bedrock between the ground surface and the historic scour line at elevation +1.5 feet NAVD should be neglected for all vertical and lateral soil support in the design of the pier

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foundation. The number of vertical piers and the extent of horizontal bracing should be minimized to avoid obstructing water and debris movement and the projected extent of flooding below the residence. Pier design will include lateral wave and debris forces due to logs and objects as presented in the following sections of this report.

d. *Liquefaction and Settlement:* Seismically-induced settlements within the beach sand layer above the historic scour elevation can be expected to occur during the design life of the structure. Provided our recommendations are incorporated into the design and construction of the residence the effects of such settlement is expected to be limited to exterior improvements or ground floor slabs which may require repair or replacement following a seismic event.

e. *Lateral Impact Forces on Foundations:* Portions of the first story of the residence may be subject to impacts from earth flows issuing from the coastal bluff located to the northeast of the property. In our opinion there is a low probability of a debris flow impact occurring simultaneously with the design wave forces; therefore the wave impact forces will govern the pier design.

f. *Strong Seismic Shaking:* The project site is located within a seismically active area and strong seismic shaking is expected to occur within the design lifetime of the project. Improvements should be designed and constructed in accordance with the most current CBC and the recommendations of this report to minimize reaction to seismic shaking. Structures built in accordance with the latest edition of the California Building Code have an increased potential for experiencing relatively minor damage which should be repairable, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs.

V. RECOMMENDATIONS

EARTHWORK

Clearing and Stripping

1. The initial preparation of the site may consist of demolition of the existing structures and their foundations and removal of designated trees and debris. All foundation elements from existing structures must be completely removed from the building areas. Tree removal should include the entire stump and root ball. Septic tanks and leaching lines, if found, must be completely removed. The extent of this soil removal will be designated by a representative of Pacific Crest Engineering Inc. in the field. This material must be removed from the site.

2. Any voids created by the removal of old structures and their foundations, tree and root balls, septic tanks, and leach lines must be backfilled with properly compacted engineered fill which meets the requirements of this report.

3. Surface vegetation, tree roots and organically contaminated topsoil should then be removed ("stripped") from the area to be graded. In addition, any remaining debris or large rocks must also be removed (this includes asphalt or rocks greater than 2 inches in greatest dimension). This material may be stockpiled for future landscaping.

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4. It is anticipated that the depth of stripping may be 2 to 4 inches. Final required depth of stripping must be based upon visual observations of a representative of Pacific Crest Engineering Inc., in the field. The required depth of stripping will vary based upon the type and density of vegetation across the project site and with the time of year.

Subgrade Preparation

5. It is possible that there are areas of man-made fill at the site that our field investigation did not detect. Areas of man-made fill, if encountered, will need to be completely excavated to undisturbed native material. The excavation process should be observed and the extent designated by a representative of Pacific Crest Engineering Inc., in the field. Any voids created by fill removal must be backfilled with properly compacted engineered fill.

6. After clearing and stripping the exposed soils in areas to receive exterior/interior concrete slabs-on-grade and pavements should be subexcavated to a minimum depth of 12 inches below bottom of slab. Final depth of subexcavation should be determined by a representative of Pacific Crest Engineering Inc., in the field.

7. Subexcavations should extend at least 3 feet horizontally beyond concrete slabs-on-grade, pavements and flatwork.

8. Care must be taken not to undermine the foundation system beneath the neighboring improvements. Excavations made adjacent to existing footings must not extend below a line drawn outward at a gradient of 3:1 (H:V) from the bottom outside edge of the footing.

9. Following clearing, stripping and any necessary subexcavations, the exposed subgrade soil that is to support concrete slabs-on-grade, foundations, pavements should then be scarified 8 inches, and the soil moisture conditioned and compacted as outlined below. The moisture conditioning procedure will depend upon the time of year that the work is done, but it should result in the soils being 1 to 3 percent over optimum moisture content at the time of compaction.

Material for Engineered Fill

10. Native or imported soil proposed for use as engineered fill should meet the following requirements:

    a. free of organics, debris, and other deleterious materials,
    b. free of "recycled" materials such as asphaltic concrete, concrete, brick, etc.,
    c. granular in nature, well graded, and contain sufficient binder to allow utility trenches to stand open,
    d. free of rocks in excess of 2 inches in size.

11. In addition to the above requirements, import fill should have a Plasticity Index between 4 and 12, and a minimum Resistance "R" Value of 30, and be non-expansive.

12. Samples of any proposed imported fill planned for use on this project should be submitted to Pacific Crest Engineering Inc. for appropriate testing and approval not less than ten (10) working days before the anticipated jobsite delivery. This includes proposed import trench sand, drain rock and for aggregate base materials. Imported fill material delivered to the project site without prior submittal of samples for appropriate testing and approval must be removed from the project site.

Engineered Fill Placement and Compaction

13. Following the subexcavation and subgrade preparation, areas to support concrete slabs-on-grade or pavements should be brought up to design grades with engineered fill that is moisture conditioned and compacted according to the recommendations of this report. **This should result in a minimum of 12 inches of engineered fill beneath slabs-on-grade floors and pavements.** Recompacted sections should extend at least 3 feet horizontally beyond all footings, slabs and pavement areas, where possible.

14. Engineered fill should be placed in maximum 8 inch lifts, before compaction, at a water content which is within 1 to 3 percent of the laboratory optimum value.

15. All soil on the project should be compacted to a minimum of 90% of its maximum dry density. The upper 8 inches of the soil subgrade in the pavement areas, and all aggregate subbase and aggregate base should be compacted to a minimum of 95% of its maximum dry density.

16. The maximum dry density will be obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557. This test will also establish the optimum moisture content of the material. Field density testing will be performed in accordance with ASTM Test #D6938 (nuclear method).

17. We recommend field density testing be performed in maximum 2 foot elevation differences. In general terms, we recommend at least one compaction test per 200 linear feet of utility trench or retaining wall backfill, and at least one compaction test per 2,000 square feet of building or structure area. This is a subjective value and may be changed by the geotechnical engineer based on a review of the final project layout and exposed field conditions.

Cut and Fill Slopes

18. No permanent cut or fill slopes are anticipated. Should cut or fill slopes be proposed, supplemental geotechnical engineering recommendations will be required.

Soil Moisture and Weather Conditions

19. If earthwork activities are done during or soon after the rainy season, the on-site soils and other materials may be too wet in their existing condition to be used as engineered fill. These materials may require a diligent and active drying and/or mixing operation to reduce the moisture content to the levels required to obtain adequate compaction as an engineered fill. If the on-site soils or other materials are too dry, water may need to be added. In some cases the time and effort to dry the on-site soil may be considered excessive, and the import of aggregate base may be required.

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Utility Trench Backfill

20. Utility trenches that are parallel to the sides of the building should be placed so that they do not extend below a line sloping down and away at a 3:1 (horizontal to vertical) slope from the bottom outside edge of all footings.

21. Utility pipes should be designed and constructed so that the top of pipe is a minimum of 24 inches below the finish subgrade elevation of any road or pavement areas. Any pipes within the top 24 inches of finish subgrade should be concrete encased, per design by the project civil engineer.

22. For the purpose of this section of the report, backfill is defined as material placed in a trench starting one foot above the pipe, and bedding is all material placed in a trench below the backfill.

23. Unless concrete bedding is required around utility pipes, free-draining clean sand should be used as bedding. Sand bedding should be compacted to at least 95 percent relative compaction. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

24. Approved imported clean sand or native soil should be used as utility trench backfill. Backfill in trenches located under and adjacent to structural fill, foundations, concrete slabs and pavements should be placed in horizontal layers no more than 8 inches thick. This includes areas such as sidewalks, patios, and other hardscape areas. Each layer of trench backfill should be water conditioned and compacted to at least 95 percent relative compaction

25. All utility trenches beneath perimeter footing or grade beams should be backfilled with controlled density fill (such as 2-sack sand\cement slurry) to help minimize potential moisture intrusion below interior floors. The length of the plug should be at least three times the width of the footing or grade beam at the building perimeter, but not less than 36 inches. A representative from Pacific Crest Engineering Inc. should be contacted to observe the placement of slurry plugs. In addition, all utility pipes which penetrate through the footings, stemwalls or grade beams (below the exterior soil grade) should also be sealed water-tight, as determined by the project civil engineer or architect.

26. Utility trenches which carry "nested" conduits (stacked vertically) should be backfilled with a control density fill (such as 2-sack sand\cement slurry) to an elevation one foot above the nested conduit stack. The use of pea gravel or clean sand as backfill within a zone of nested conduits is not recommended.

27. A representative from our firm should be present to observe the bottom of all trench excavations, prior to placement of utility pipes and conduits. In addition, we should observe the condition of the trench prior to placement of sand bedding, and to observe compaction of the sand bedding, in addition to any backfill planned above the bedding zone.

28. Jetting of the trench backfill is not recommended as it may result in an unsatisfactory degree of compaction.

29. Trenches must be shored as required by the local agency and the State of California Division of Industrial Safety construction safety orders.

Excavations and Shoring

30. It should be understood that on-site safety is the *sole responsibility* of the Contractor, and that the Contractor shall designate a *competent person* (as defined by CAL-OSHA) to monitor the slope excavation prior to the start of each work day, and throughout the work day as conditions change. The competent person designated by the Contractor shall determine if flatter slope gradients are more appropriate, or if shoring should be installed to protect workers in the vicinity of the slope excavation. Refer to Title 8, California Code of Regulations, Sections 1539-1543.

31. All excavations must meet the requirements of 29 CFR 1926.651 and 1926.652 or comparable OSHA approved state plan requirements.

32. The "top" of any temporary cut slope and excavations should be set-back at least ten feet (measured horizontally) from any nearby structure or property line. Any excavations which cannot meet this requirement will need to have a shoring system designed to support steeper sidewall gradients.

33. Temporary shoring is not currently anticipated for this project. Should these requirements change, please contact our office for additional recommendations.

FOUNDATIONS – DRILLED PIERS

General

34. At the time we prepared this report the grading plans had not been completed and the structure location and foundation details had not been finalized. We request an opportunity to review these items during the design stages to determine if supplemental recommendations will be required.

35. The residence will be supported by a grid-type foundation system, consisting of drilled piers that will penetrate the overlying beach sand stratum and extend a minimum depth of 10 feet into dense sandstone bedrock. The piers should be designed to develop their load carrying capacity through skin friction between the pier bottom and the underlying bedrock. The bedrock is very dense and will require specialized equipment to ensure that the piers extend to the full depth as outlined in the geotechnical report and the project plans and specifications.

36. Because the final pier depths are dependent upon the historic scour elevation of +1.5 feet NGVD, we recommend establishing a benchmark elevation at the site prior to pier drilling. Pier depths will be determined from the benchmark elevation rather than depth below existing grades.

37. The number of vertical piers and horizontal structural bracing should be minimized to allow maximum flood flow area. Horizontal bracing should be oriented parallel to the flow direction where possible to reduce flow obstructions.

38. We anticipate that that the pier excavations will need to be completely cased to keep the pier excavations from caving before the concrete can be poured. We also anticipate that the pier excavations will need to be cleaned out and pumped of water prior to placing concrete.

39. If the casing is pulled during the concrete pour, it must be pulled slowly with a minimum of 4 feet of casing remaining embedded within the concrete at all times.

40. If concrete is placed via a tremie, the end of the tube must remain embedded a minimum of 4 feet into the concrete at all times.

41. All piers must be constructed within ½ percent of a vertically plumb condition.

42. The drilling contractor should be experienced with drilling in coastal conditions with flowing sands. The contractor must assume responsibility for his work procedures, and therefore, needs to be proficient in performing the work he is contracted to do. Pier drilling is expected to be cumbersome for this project and the drilling contractor should be experienced with construction of piers in a flowing sand condition.

43. All pier construction must be observed by a Pacific Crest Engineering Inc. Any piers constructed without the full knowledge and continuous observation of a representative from Pacific Crest Engineering Inc., will render the recommendations of this report invalid.

Vertical Bearing Capacity

44. Minimum pier embedment should be 10 feet below the historic scour elevation; this will necessitate a *minimum* pier bottom elevation of -8.5 feet NAVD. Minimum pier depths are expected to be on the order of at least 26 feet below existing grades. Actual depths could depend upon a lateral force analysis performed by your structural engineer.

45. The piers should be a minimum of 24 inches in diameter. All pier holes must be free of loose material on the bottom.

46. Piers constructed to the above criteria may be designed for an allowable skin friction capacity of 600 pounds per square foot between the pier shaft and **sandstone bedrock**. The allowable bearing capacity may be increased by 1/3rd for short-term wind or seismic loading.

47. An allowable skin friction due to the bedrock stratum of 400 psf per square foot of pier surface area may be used to resist uplift forces. Skin friction should be neglected from the ground surface to +1.5 feet NAVD.

48. Passive resistance due to competent bedrock of 400 pcf equivalent fluid pressure (EFP) acting over two pier diameters may be used. Passive resistance should be neglected from the ground surface to +1.5 feet NGVD (approximately the upper 16 feet below existing ground surface).

49. If the structural designer wishes to include seismic forces in their design, the wall may be designed using the above active soil pressures plus a horizontal seismic force of 12H^2 pounds per lineal foot (where H is the height of retained material). The resultant seismic force should be applied at a point 1/3rd above the base of the wall. This force has been estimated using the Mononobe-Okabe method of analysis as

EXHIBIT F

modified by Whitman (1990) and Lew and Sitar (2010). A reduced factor of safety for overturning and sliding may be used in seismic design as determined by the structural designer.

Lateral/Wave Forces

50. The foundation system should be designed to resist an active lateral force of 30 pcf (EFP) due to lateral spreading of beach sand above the historic scour line. This load may be assumed to act over 1½ pier diameters.

51. We recommend a breaking wave load (F_{brkp}) on the pier of 5.8 kips per foot of pier diameter. The wave force should be assumed to act at the still water elevation (elevation +14.7 NAVD).

52. Hydrodynamic loads (F_{dyn}) imposed by moving flood waters of 6.7 kips per foot of pile diameter, acting at +8.1 NAVD (halfway between the scour elevation and the design stillwater level)

53. Wave-borne debris can be expected to impact the foundation system during its design life. Storm waves commonly carry large logs and other debris toward shore, it is recommended that the flood velocity of 21 feet per second be used when calculating debris impact loads (F_i). The force can be assumed to act at the design stillwater elevation (14.7 feet NAVD).

54. The structural engineer should refer to Chapter 8 -11 of the 2011 FEMA Coastal Construction Manual for guidance in determining the flood load combinations for this particular project.

55. Although not suggested by FEMA, in our opinion the potential exists for wave uplift forces to exert pressure upon horizontal structural members at or below the BFE. We recommend an uplift pressure of 500 psf. be considered.

RETAINING WALLS

56. Retaining walls are proposed on the sides of the property and between the house and the bluff. The tallest wall proposed roughly 22 northeast of the existing residence is anticipated to have a base elevation of about 18 feet and be about 15 feet in height. This wall is not intended to stop debris flows but instead to allow a free flowing path beneath the habitable portion of the structure. Assuming no tie-backs are required, the following parameters may be used for design of this wall. We request the opportunity to review proposed retaining wall designs to verify that these parameters apply.

Table No. 3, Active Earth Pressure Values, Equivalent Fluid Pressures

(Assuming no Tie Backs)

Maximum Backfill Slope (H:V)	Active Earth Pressure (psf/ft of depth)
Level	35
2:1	50
1½:1	75

57. Should the slope behind the retaining walls be other than shown in Table 3, supplemental design criteria will be provided for the active earth for the particular slope angle.

58. Active earth pressure values may be used when walls are free to yield an amount sufficient to develop the active earth pressure condition (about ½% of height). The effect of wall rotation should be considered for areas behind the planned retaining wall (pavements, foundations, slabs, etc.). When walls are restrained at the top or to design for minimal wall rotation, at-rest earth pressure values should be used.

59. Retaining walls should be supported on drilled pier foundation systems as outlined in the Foundations section of this report with the following exceptions:
    - Minimum pier embedment should be 10 feet into bedrock. The bedrock elevation is inferred to be steeply sloping in this location. We roughly estimate that bedrock will be encountered between about elevation 1.5 and 13 NAVD, depending on the wall location. Final embedment depth should be determined by the project structural engineer.
    - Skin friction and lateral passive resistance of all beach sand and colluvium should be ignored. Where the bedrock is steeply sloping, the upper 5 feet of rock should also be ignored for lateral passive resistance.

Tie-Back Anchors

60. If tiebacks are required, tieback retaining walls should be designed for apparent lateral earth pressure. The appropriate apparent earth pressure diagram will depend on the location of the tieback anchors with respect to the vertical wall face. Refer to the Apparent Earth Pressure Diagram, Figure 9, in Appendix A for details. If tie backs are not used the active equivalent fluid pressures provided in the previous section should be used. Please note that these earth pressure diagrams assume fully drained conditions. If fully drained conditions are not provided behind the wall, an additional hydrostatic load of 62.4 psf/ft should be included in the wall design.

61. The tie-back wall design should incorporate all geotechnical design criteria outlined within the foundation section, including seismic design criteria, if appropriate. Tie-back design and the construction techniques for installing them are the responsibility of the specialty tie-back contractor.

62. Preliminary design of the tie-backs should be based on an ultimate rock/grout bond value of 2000 psf. Frictional resistance of all soil should be ignored. Final bonded length should be based on field conditions and pull out tests. Actual strengths developed will depend upon the actual material in which the tie-backs are embedded, diameter of the tie-back hole, roughness of the hole grouting technique, grout strength and other construction factors. It is the Contractor's responsibility to construct tie-backs which develop the required tie-back capacity.

63. Tie-backs should be installed at an inclination of about 10 to 20 degrees below horizontal.

64. The bonded length of the tie-back anchor should begin outside the "active" soil wedge behind the wall. The active wedge is estimated as a 1:1 (h:v) plane measured from the base of the wall, and extending upward

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towards the ground surface beneath the road. We recommend a minimum bonded length extending 15 feet beyond the active wedge or 10 feet into rock, whichever is greater.

65. All spoils from the tie-back drilling work must be removed from the site. These materials may not be placed on the slope area below the retaining wall.

66. All tie-backs must be proof tested by the Contractor in the presence of the Geotechnical Engineer to 133% of their design load, with 25% of the anchors performance tested to 133% of the design load. Any tie-backs that fail during testing must be removed, reconstructed and retested at the Contractor's expense. Testing and acceptance criteria should be based on that presented by Post Tensioning Institute ("Recommendations for Presetressed Rock and Soil Anchors, 2014").

67. Tie-back anchors should be locked off at a value of at least 80 to 90 percent of the design load for the tie-back anchor, or as determined by the project structural engineer.

68. Tie-back designs, construction details and corrosion protection systems must be submitted to the Civil Engineer and the Geotechnical Engineer a minimum of three weeks in advance of the commencement of tie-back construction for review and approval.

69. All tie-back anchor construction and testing must be observed by a representative from Pacific Crest Engineering Inc. Any tie-back anchors constructed without the full knowledge and continuous observation of Pacific Crest Engineering Inc., will render the recommendations of this report invalid. The Contractor and drilling subcontractor should be notified regarding this requirement.

Retaining Wall Drainage

70. The above design criteria are based on fully drained conditions. Therefore, we recommend that permeable material meeting the State of California Standard Specification Section 68-1.025, Class 1, Type A, be placed behind the wall, with a minimum width of 12 inches and extending for the full height of the wall to within 1 foot of the ground surface. The top of the permeable material should be covered with Mirafi 140N filter fabric or equivalent and then compacted native soil placed to the ground surface. A 4-inch diameter perforated rigid plastic drain pipe should be installed within 3 inches of the bottom of the permeable material and be discharged to a suitable, approved location. The perforations should be placed downward; oriented along the lower half of the pipe. Neither the pipe nor the permeable material should be wrapped in filter fabric. Please refer to the Typical Retaining Wall Drain Detail, Figure 10, in Appendix A for details.

71. The area behind the wall and beyond the permeable material should be compacted with approved material to a minimum relative compaction of 90%.

SLAB-ON-GRADE CONSTRUCTION

72. Slab-on-grade may be used for ground level construction on native soil or engineered fill. It should be clearly understood that slab floors and/or patios and walkways will need to be replaced following severe coastal flooding or debris flow impacts.

73. All concrete slab-on-grade should be designed in conformance with FEMA's recommendations as outlined in the Coastal Construction Manual. Slabs should not be structurally integrated with the footings.

SURFACE DRAINAGE

74. Surface water drainage is the responsibility of the project civil engineer. The following should be considered by the civil engineer in design of the project.

75. Surface water must not be allowed to pond or be trapped adjacent to foundations, or on building pads and parking areas.

76. All roof eaves should be guttered, with the outlets from the downspouts provided with adequate capacity to carry the storm water away from structures to reduce the possibility of soil saturation and erosion. The connection should be in a closed conduit which discharges at an approved location away from structures and graded areas.

77. Slope failures can also occur where surface drainage is allowed to concentrate onto unprotected slopes. Appropriate landscaping and good control of surface drainage around the project area becomes very important to reduce potential for shallow slumping of slopes. Erosion control measures should be implemented and maintained. Under no circumstances should surface runoff be directed toward, or discharged upon, any topographic slopes.

78. Final grades should be provided with positive gradient away from all foundation elements. Soil grades should slope away from foundations at least 5 percent for the first 10 feet. Impervious surfaces should slope away from foundations at least 2 percent for the first 10 feet. Concentrations of surface runoff should be handled by providing structures, such as paved or lined ditches, catch basins, etc.

EROSION CONTROL

79. The surface soils are classified as having a high potential for erosion. Therefore, the finished ground surface should be planted with ground cover and continually maintained to minimize surface erosion. For specific and detailed recommendations regarding erosion control on and surrounding the project site, the project civil engineer or an erosion control specialist should be consulted.

PLAN REVIEW

80. We respectfully request an opportunity to review the project plans and specifications during preparation and before bidding to ensure that the recommendations of this report have been included and to provide additional recommendations, if needed. These plan review services are also typically required by the reviewing agency. Misinterpretation of our recommendations or omission of our requirements from the project plans and specifications may result in changes to the project design during the construction phase, with the potential for additional costs and delays in order to bring the project into conformance with the requirements outlined within this report. Services performed for review of the project plans and specifications are considered "post-report" services and billed on a "time and materials" fee basis in accordance with our latest Standard Fee Schedule.

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VI. LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. This Geotechnical Investigation was prepared specifically for Jim and Sue Vaudagna and for the specific project and location described in the body of this report. This report and the recommendations included herein should be utilized for this specific project and location exclusively. This Geotechnical Investigation should not be applied to nor utilized on any other project or project site. Please refer to the ASFE "Important Information about Your Geotechnical Engineering Report" attached with this report.

2. 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 provided.

3. 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.

4. 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 process or 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 of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes. This report should not be considered valid after a period of two (2) years without our review.

5. This report was prepared upon your request for our services in accordance with currently accepted standards of professional geotechnical engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.

6. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

EXHIBIT F

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/THE BEST PEOPLE ON EARTH exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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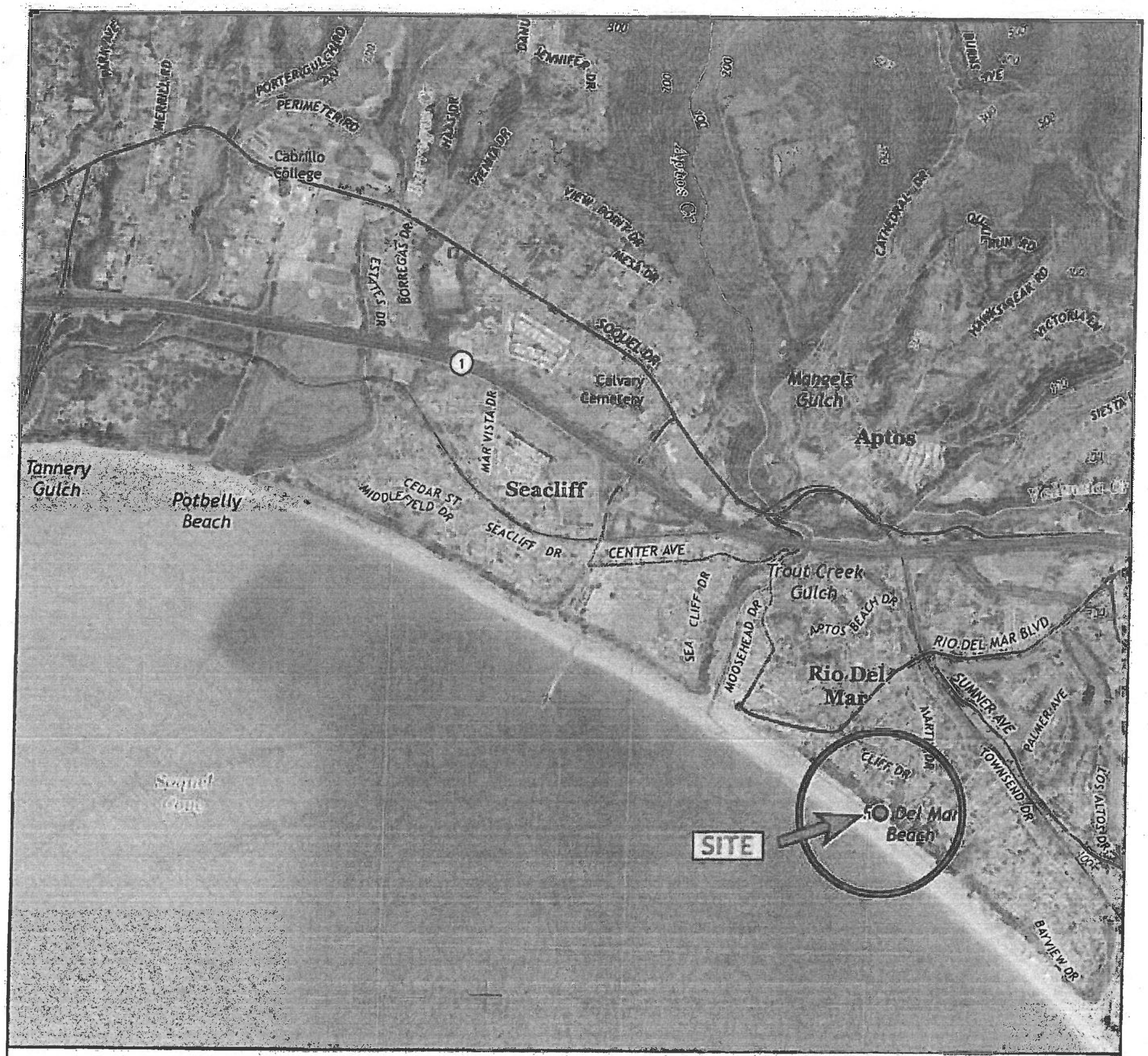
EXHIBIT F

APPENDIX A

Regional Site Map
Site Map Showing Test Borings
Key to Soil Classification
Log of Test Borings
Apparent Earth Pressure Diagram for Tie Back Walls

EXHIBIT F



0 2000 ft.

Scale

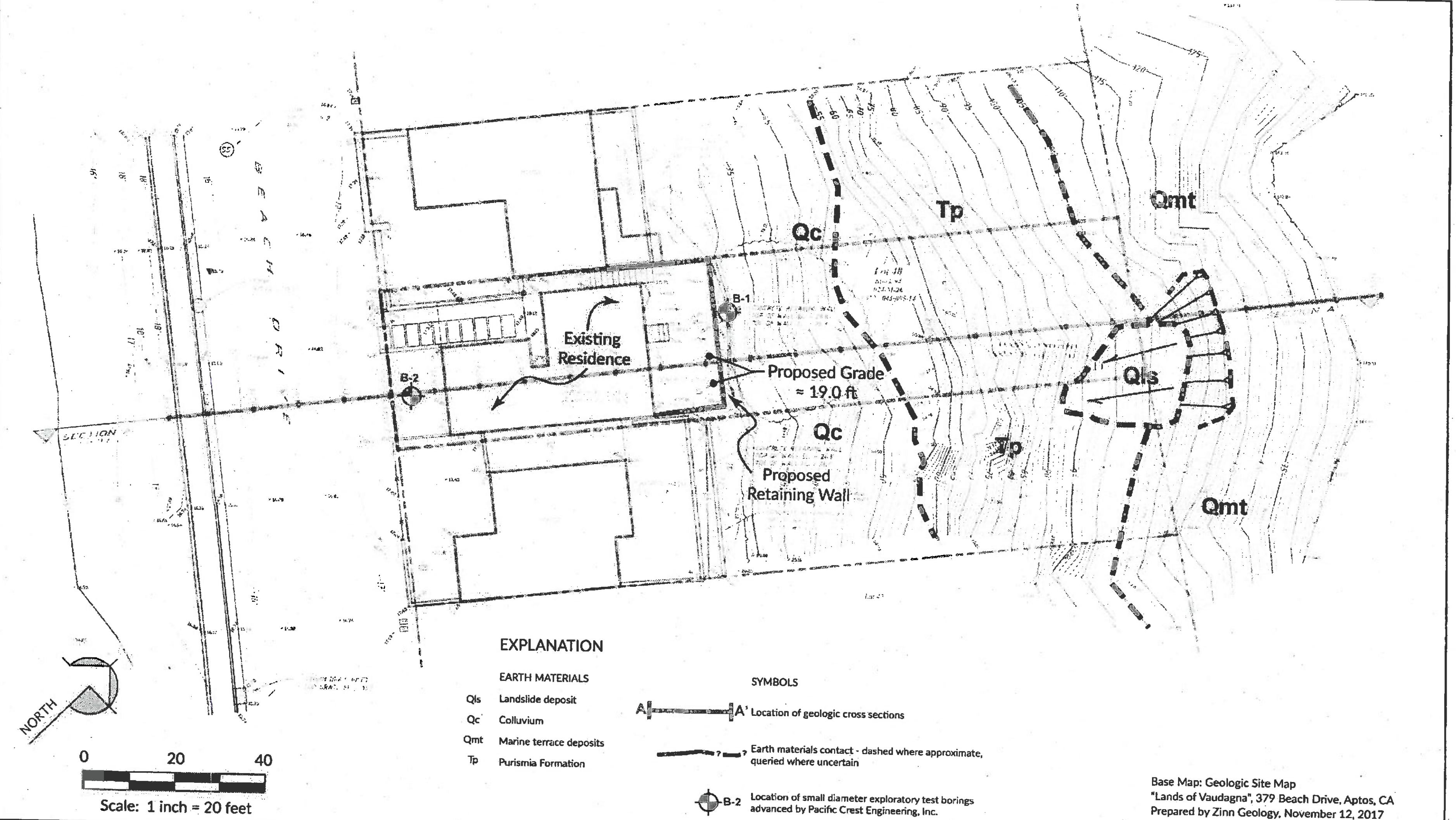
N

Base Map: United States Geological Survey
Soquel Quadrangle, California - Santa Cruz County
7.5 Minute Series, Soquel, CA 2015

Pacific Crest ENGINEERING INC	**Regional Site Map** 379 Beach Drive Aptos, California	Figure No. 1 Project No. 1738 Date: 11/30/17

EXHIBIT F

EXHIBIT F



EXPLANATION

EARTH MATERIALS

Qls	Landslide deposit
Qc	Colluvium
Qmt	Marine terrace deposits
Tp	Purismia Formation

SYMBOLS

A——A' Location of geologic cross sections

—?— Earth materials contact - dashed where approximate, queried where uncertain

B-2 Location of small diameter exploratory test borings advanced by Pacific Crest Engineering, Inc.

Base Map: Geologic Site Map
"Lands of Vaudagna", 379 Beach Drive, Aptos, CA
Prepared by Zinn Geology, November 12, 2017

Pacific Crest
ENGINEERING INC

Site Map Showing Test Boring Locations

379 Beach Drive
Aptos, California

Figure No. 2
Project No. 1738
Date: 11/30/17

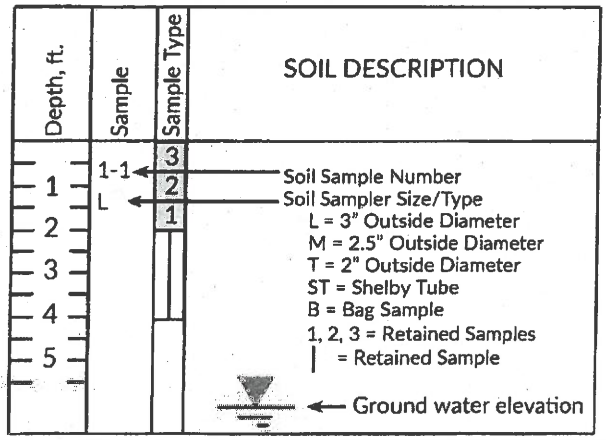
KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS (FGS)
UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2487 (Modified)

MAJOR DIVISIONS		SYMBOL	FINES	COARSENESS	SAND/GRAVEL	GROUP NAME
SILT AND CLAY	*LL < 35% Low Plasticity	CL Lean Clay PI > 7 Plots Above A Line -OR- ML Silt PI > 4 Plots Below A Line	<30% plus No. 200	<15% plus No. 200		Lean Clay / Silt
				15-30% plus No. 200	% sand ≥ % gravel	Lean Clay with Sand / Silt with Sand
					% sand < % gravel	Lean Clay with Gravel / Silt with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Lean Clay / Sandy Silt
					≥ 15% gravel	Sandy Lean Clay with Gravel / Sandy Silt with Gravel
				% sand < % gravel	< 15% sand	Gravelly Lean Clay / Gravelly Silt
					≥ 15% sand	Gravelly Lean Clay with Sand / Gravelly Silt with Sand
		CL - ML 4 < PI < 7	<30% plus No. 200	<15% plus No. 200		Silty Clay
				15-30% plus No. 200	% sand ≥ % gravel	Silty Clay with Sand
					% sand < % gravel	Silty Clay with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Silty Clay
					≥15% gravel	Sandy Silty Clay with Gravel
				% sand < % gravel	< 15% sand	Gravelly Silty Clay
					≥15% sand	Gravelly Silty Clay with Sand
	35%≤ *LL < 50% Intermediate Plasticity	CI	<30% plus No. 200	<15% plus No. 200		Clay
				15-30% plus No. 200	% sand ≥ % gravel	Clay with Sand
					% sand < % gravel	Clay with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Clay
					≥ 15% gravel	Sandy Clay with Gravel
				% sand < % gravel	< 15% sand	Gravelly Clay
					≥ 15% sand	Gravelly Clay with Sand
	*LL > 50% High Plasticity	CH Fat Clay Plots Above A Line -OR- MH Elastic Silt Plots Below A Line	<30% plus No. 200	<15% plus No. 200		Fat Clay or Elastic Silt
				15-30% plus No. 200	% sand ≥ % gravel	Fat Clay with Sand Elastic Silt with Sand
					% sand < % gravel	Fat Clay with Gravel / Elastic Silt with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Fat Clay / Sandy Elastic Silt
					≥ 15% gravel	Sandy Fat Clay with Gravel / Sandy Elastic Silt with Gravel
				% sand < % gravel	< 15% sand	Gravelly Fat Clay / Gravelly Elastic Silt
					≥ 15% sand	Gravelly Fat Clay with Sand / Gravelly Elastic Silt with Sand

* LL = Liquid Limit

* PI = Plasticity Index

BORING LOG EXPLANATION



MOISTURE

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp, but no visible water
WET	Visible free water, usually soil is below the water table

CONSISTENCY

DESCRIPTION	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 - 0.5	2 - 4
FIRM	0.5 - 1.0	5 - 8
STIFF	1.0 - 2.0	9 - 15
VERY STIFF	2.0 - 4.0	16 - 30
HARD	> 4.0	> 30

Pacific Crest
ENGINEERING INC

Boring Log Explanation - FGS
379 Beach Drive
Aptos, California

Figure No. 3
Project No. 1738
Date: 11/30/17

EXHIBIT F

KEY TO SOIL CLASSIFICATION - COARSE GRAINED SOILS
UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2487 (Modified)

MAJOR DIVISIONS		FINES	GRADE/TYPE OF FINES	SYMBOL	GROUP NAME *
GRAVEL	More than 50% of coarse fraction is larger than No. 4 sieve size	<5%	Cu ≥ 4 and 1 ≤ Cc ≤ 3	**GW**	Well-Graded Gravel / Well-Graded Gravel with Sand
			Cu < 4 and/or 1 > Cc > 3	**GP**	Poorly Graded Gravel/Poorly Graded Gravel with Sand
		5-12%	ML or MH	**GW - GM**	Well-Graded Gravel with Silt / Well- Graded Gravel with Silt and Sand
				GP - GM	Poorly Graded Gravel with Silt / Poorly Graded Gravel with Silt and Sand
			CL, Cl or CH	**GW - GC**	Well-Graded Gravel with Clay / Well-Graded Gravel with Clay and Sand
				GP - GC	Poorly Graded Gravel with Clay / Poorly Graded Gravel with Clay and Sand
		>12%	ML or MH	**GM**	Silty Gravel / Silty Gravel with Sand
			CL, Cl or CH	**GC**	Clayey Gravel/ Clayey Gravel with Sand
			CL - ML	**GC - GM**	Silty, Clayey Gravel/Silty, Clayey Gravel with Sand
SAND	50% or more of coarse fraction is smaller than No. 4 sieve size	<5%	Cu ≥ 6 and 1 ≤ Cc ≤ 3	**SW**	Well-Graded Sand / Well-Graded Sand with Gravel
			Cu < 6 and/or 1 > Cc > 3	**SP**	Poorly Graded Sand / Poorly Graded Sand with Gravel
		5-12%	ML or MH	**SW - SM**	Well-Graded Sand with Silt / Well- Graded Sand with Silt and Gravel
				SP - SM	Poorly Graded Sand with Silt / Poorly Graded Sand with Silt and Gravel
			CL, Cl or CH	**SW - SC**	Well-Graded Sand with Clay / Well-Graded Sand with Clay and Gravel
				SP - SC	Poorly Graded Sand with Clay / Poorly Graded Sand with Clay and Gravel
		>12%	ML or MH	**SM**	Silty Sand / Silty Sand with Gravel
			CL, Cl or CH	**SC**	Clayey Sand / Clayey Sand with Gravel
			CL - ML	**SC - SM**	Silty, Clayey Sand / Silty, Clayey Sand with Gravel

* The term "with sand" refers to materials containing 15% or greater sand particles within a gravel soil, while the term "with gravel" refers to materials containing 15% or greater gravel particles within a sand soil.

US STANDARD SIEVE SIZE: 3 inch | ¾ inch | No. 4 | No. 10 | No. 40 No. 200 | 0.002 µm

	COARSE	FINE	COARSE	MEDIUM	FINE		
COBBLES AND BOULDERS	GRAVEL		SAND			SILT	CLAY

RELATIVE DENSITY

DESCRIPTION	STANDARD PENETRATION (BLOWS/FOOT)
VERY LOOSE	0 - 4
LOOSE	5 - 10
MEDIUM DENSE	11 - 30
DENSE	31 - 50
VERY DENSE	> 50

MOISTURE

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp, but no visible water
WET	Visible free water, usually soil is below the water table

Pacific Crest ENGINEERING INC	**Boring Log Explanation - CGS** 379 Beach Drive Aptos, California	Figure No. 4 Project No. 1738 Date: 11/30/17

EXHIBIT F

LOGGED BY CLA DATE DRILLED 5/17/17 BORING DIAMETER 4" SS BORING NO. 1

DRILL RIG Cenozoic Portable HAMMER TYPE 70 lb Hand Hammer

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	% Passing #200 Sieve	Dry Density (pcf)	Moisture Content (%)	Additional Lab Results
1 2	1-1 L	1	**FILL: SILTY:** Dark yellowish brown (10YR 4/4) and yellowish brown (10YR 5/4), fine grained with trace medium grains, poorly graded, quartz rich, poorly indurated, trace rootlets, charcoal noted at 2½ feet, slightly damp	SM	3 5 10	15		15	86	9	
3 4	1-2 T		Mica flakes scattered throughout the sample		3 3 2	5					
5			Encountered chain link fencing at 5 feet, moved bore hole 5 feet north								
6 7	1-3 L		**FILL: SAND WITH SILT:** Dark yellowish brown (10YR 4/4) and yellowish brown (10YR 5/4), fine grained with trace medium grains, poorly graded, clean, quartz rich, trace sub-angular shaped conglomerate (wall backfill), slightly moist	SP	30/6"			11	100	10	
8 9	1-4 T		**NATIVE: SAND WITH SILT:** Dark yellowish brown (10YR 4/4) and yellowish brown (10YR 5/4), fine grained with trace medium grains, poorly graded, clean, quartz rich, rounded chert gravels up to 1 inch in diameter, slightly moist to moist	SP	4 5 7	12					
10 11	1-5 L	2 1	Decrease in gravel content, trace coarse to very coarse grained chert and quartz sand, mica flakes scattered thoughout the sample, slightly moist		6 8 10	18		7	104	9	
12 13 14											
15 16	1-6 T		Trace silt, slightly more indurated than the previous sample		8 10 19	29					
17 18 19											
20 21	1-7 L 1-8 T		Lack of split, lack of coarse to very coarse grained sand, trace fine to medium grained chert sand ▽ Trace rounded chert pebbles up to ¼ inch in diameter		30/3" 30/5"			8		9	
22 23											

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 379 Beach Drive Aptos, California	Figure No. 5 Project No. 1738 Date: 11/30/17

EXHIBIT F

LOGGED BY CLA DATE DRILLED 5/17/17 BORING DIAMETER 4" SS BORING NO. 1

DRILL RIG Cenozoic Portable HAMMER TYPE 70 lb Hand Hammer

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	% Passing #200 Sieve	Dry Density (pcf)	Moisture Content (%)	Additional Lab Results
24			**PURISMA SANDSTONE BEDROCK:** Dark yellowish brown (10YR 4/4), weathered to a fine grained sand, trace medium to coarse grains, poorly graded, quartz rich with trace chert grains, bedded, friable, clean, wet, very soft rock hardness								
25	1-9 T	▯			30/4"						
26											
27											
28	1-10 T	▯	Dark olive brown (2.5Y 3/3), very fine to fine grained, poorly graded, quartz rich with trace chert grains, clean, massive		30/3"			8		26	
29											
30			Boring terminated at 28½ feet. Groundwater encountered at 22 feet. Note: Samples advanced with hand driven 70 lb. hammer - SPT values not corrected for hammer type								
31											
32											
33											
34											
35											
36											
37											
38											
39											
40											
41											
42											
43											
44											
45											
46											

Pacific Crest Engineering Inc.
444 Airport Blvd., Suite 106
Watsonville, CA 95076

Log of Test Borings
379 Beach Drive
Aptos, California

Figure No. 6
Project No. 1738
Date: 11/30/17

EXHIBIT F

LOGGED BY CLA DATE DRILLED 5/24/17 BORING DIAMETER 8" HS BORING NO. 2

DRILL RIG EGI Truck Mounted Mobile B53 Red HAMMER TYPE 140 lb Down-Hole Safety Hammer

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	% Passing #200 Sieve	Dry Density (pcf)	Moisture Content (%)	Additional Lab Results
			CONCRETE: 4"								
1 2	2-1 L	2 1	**FILL: SAND:** Olive brown (2.5Y 4/3) and light gray (2.5Y 7/2), fine to medium grained, sub-angular to sub-rounded shaped, poorly graded, clean, quartz rich, slightly moist, loose	SP	5 8 5	7		21	96	11	
3 4	2-2 T		**FILL: CLAYEY SAND:** Yellowish brown (10YR 5/6), fine grained with trace medium grains, poorly graded, quartz rich, scattered mica flakes, trace rootlets, slightly moist, loose	SC	5 9 15	24					
5 6	2-3 L	2 1	**SAND:** Light olive brown (2.5Y 5/4) and light gray (2.5Y 7/2), fine to medium grained, sub-angular to sub-rounded shaped, poorly graded, clean, quartz rich, poorly indurated, dry, medium dense Loose	SP	9 10 10	10		2	100	4	
7 8	2-4 T		Increase in drilling resistance at 7 feet Thin magnitite interbeds from 7 to 8 feet		5 5 4	9					
9 10	2-5 L	1	Slight increase in coarseness of sand, medium grained		6 8 10	9			97	5	
11	2-6 T		Approximately ½ inch thick magnitite bed at 11½ feet, moist, medium dense		7 8 12	20					
12			▽								
13 14 15	2-7 L	2 1	Dark grayish brown (2.5Y 4/2) and light gray, increase in coarseness of sand, medium to coarse grained with trace very coarse grains, quartz and chert sand, moist, dense, (trace shell fragments)		24 33 40	38		4	112	15	
16			Increase in drilling resistance at 16 feet								
17 18			**PURISMA SANDSTONE BEDROCK:** Dark grayish brown (2.5Y 4/2), weathered to a sand, fine grained, poorly graded, quartz rich, massive, friable, scattered mica flakes, moist, very soft rock hardness								
19 20	2-8 T				23 33 50/6"	50/6"					
21											
22											
23											

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 379 Beach Street Aptos, California	Figure No. 7 Project No. 1738 Date: 11/30/17

EXHIBIT F

LOGGED BY CLA DATE DRILLED 5/24/17 BORING DIAMETER 4" SS BORING NO. 2

DRILL RIG EGI Truck Mounted Mobile B53 Red HAMMER TYPE 140 lb DH Safety

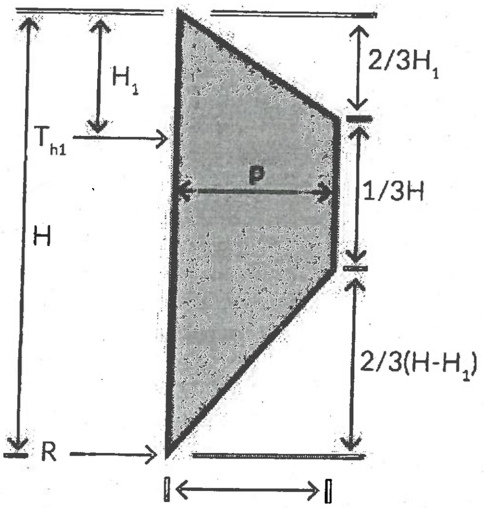
Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	% Passing #200 Sieve	Dry Density (pcf)	Moisture Content (%)	Additional Lab Results
24	2-9 L	▨	**PURISMA SANDSTONE BEDROCK:** Dark grayish brown (2.5Y 4/2), fine grained, poorly graded, quartz rich, massive, friable, weathered to a sand, mica flakes, wet, very soft rock hardness		50/4"	50/4"					
25											
26											
27											
28											
29	2-10 L	▨	Dark yellowish brown (10YR 4/4), fine to medium grained with trace coarse grains, trace very coarse grained rounded chert sand Trace rounded chert pebbles up to ¼ inch in diameter		50/5"	50/5"		3	106	20	
30	2-11 T				24 50/5"						
31			Boring terminated at 30 feet. Groundwater initially encountered at 12 feet. Measured at 14½ feet at the end of drilling activities.								
32											
33											
34											
35											
36											
37											
38											
39											
40											
41											
42											
43											
44											
45											
46											

Pacific Crest Engineering Inc.
444 Airport Blvd., Suite 106
Watsonville, CA 95076

Log of Test Borings
379 Beach Drive
Aptos, California

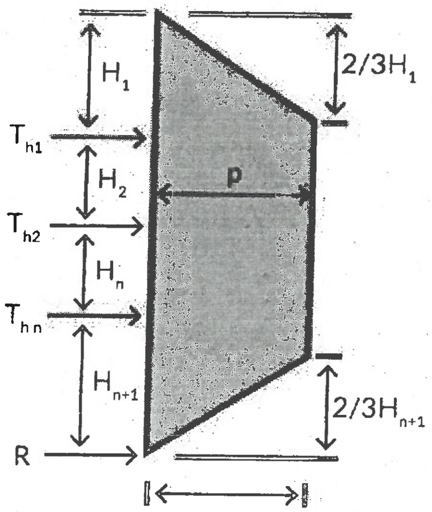
Figure No. 8
Project No. 1738
Date: 11/30/17

EXHIBIT F



$$p = \frac{\text{TOTAL LOAD}}{2/3H} \approx K_A \gamma H$$

Walls with one level of ground anchors



$$p = \frac{\text{TOTAL LOAD}}{H - 1/3H_1 - 1/3H_{n+1}}$$

Walls with multiple levels of ground anchors

TOTAL LOAD = $0.65\ K_A \gamma H^2$

H_1 = Distance from ground surface to uppermost ground anchor
H_{n+1} = Distance from base of excavation to lowermost ground anchor
T_{h1} = Horizontal load in ground anchor 1
R = Reaction force to be resisted by subgrade (i.e.; below base of excavation)
p = Maximum ordinate of diagram

Recommended Soil Parameters:

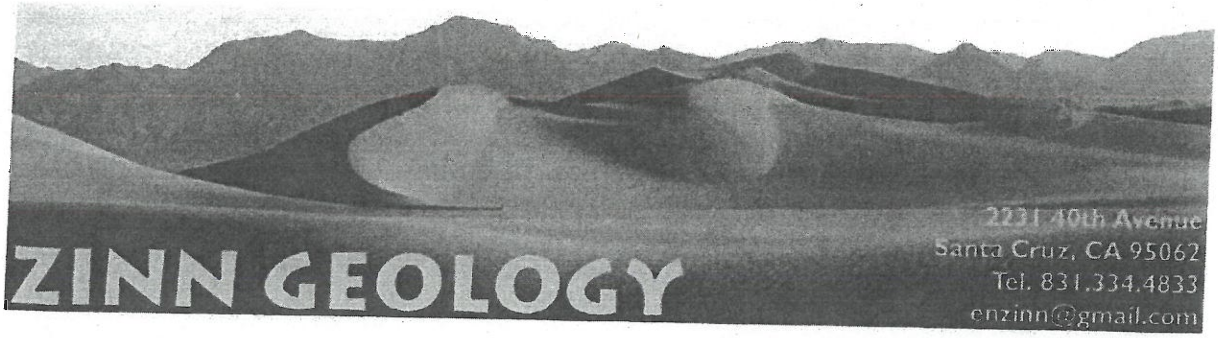
Level Backslope	K_A = 0.30
2:1 Backslope Angle:	K_A = 0.47
1.5:1 Backslope Angle:	K_A = 0.85

γ = 115 pcf

Based on: FHWA-IF-015 "Ground Anchors and Anchored Systems"

Pacific Crest ENGINEERING INC	**Apparent Earth Pressure Diagram for Tie Back Walls** 379 Beach Drive, Aptos, California	Figure No. 9 Project No. 1738 Date: 11/30/17

EXHIBIT F



ZINN GEOLOGY
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Santa Cruz, CA 95062
Tel. 831.334.4833
enzinn@gmail.com

COASTAL GEOLOGIC INVESTIGATION

Lands of Vaudagna
379 Beach Drive
Aptos, California
County of Santa Cruz APN 043-095-14

Job #2017011-G-SC
11 February 2018

Engineering Geology ⚒ Coastal Geology ⚒ Fault & Landslide Investigations

EXHIBIT G

ZINN GEOLOGY
2231 40th Avenue
Santa Cruz, CA 95062
Tel. 831.334.4833
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11 February 2018

Job #2017011-G-SC

Jim and Sue Vaudagna
19501 Scotland Drive
Saratoga, CA 95070

Re: Coastal geologic investigation
379 Beach Drive
Aptos, California
County of Santa Cruz APN 043-095-14

Dear Mr. And Ms. Vaudagna:

Our geologic report on the property referenced above is attached. This report documents geologic conditions on the subject property and addresses potential hazards to the proposed construction of a single-family residence, slated to replace the existing residence, such as coastal flooding, erosion, seismic shaking, landsliding and liquefaction.

Based on the information gathered and analyzed, it is our opinion that the proposed residence will be geologically suitable, provided our recommendations are adequately adhered to by the design team. The proposed residence will be subject to "ordinary" risks as defined in Appendix B, provided our recommendations are followed. Appendix B should be reviewed in detail by the developer and all property owners to determine whether an "ordinary" risk as defined in the appendix is acceptable. If this level of risk is unacceptable to the developer and the property owners, then the geologic hazards in question should be mitigated to reduce the corresponding risks to an acceptable level.

The most recent issue of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) Community Panel number 066087C0359F portrays the property as being within the limit of the floodway flood zone VE. FEMA has calculated a coastal base flood elevation of +21.0 feet above mean sea level (NAVD88) for this zone. Since the ground surface is no higher than about +17½ feet NAVD 88 in the developable portions of the property, the risk to any new structures constructed at or near the existing grade due to coastal flooding is clearly greater than ordinary for that particular flood elevation.

The contact between the beach sand and the underlying bedrock is about 1.5 feet above mean sea level (NAVD88) and this contact marks the former scour elevation for the property. Future

EXHIBIT G

storms may scour out the beach sand to that depth again, particularly when considering the impacts of continuing rising sea levels and intensity and frequency of large storms. It is also important to note that such an extreme scour depth will expose the foundation elements embedded in the loose sandy soils to battering by objects caught up in breaking waves such as logs. For the sake of simplicity and conservatism, we recommend that the future structures be designed for a scour depth of +1.5 feet NAVD88 across the lower portion of the property, **where warranted or required**. If design for scour is not required for the structures being considered (such as the case of the building something high up on the coastal bluff), this finding and accompanying recommendation can be ignored.

Seismic shaking at the subject site will be intense during the next major earthquake along one of the local fault systems. It is important that the recommendations regarding seismic shaking be considered in the design for the proposed developments where applicable. The proposed development will be geologically suitable, if it is designed and constructed in conformance with the seismic parameters issued in the PCEI report, where warranted.

It is our opinion that the proposed residence will be subject to a greater than ordinary risk related to the debris flow and landslide hazard. In our opinion the risk to the proposed residence can be adequately reduced to ordinary if a debris flow impact wall or barrier is constructed up slope of the residence. Alternatively, the lower floor of the residence can be designed as non-habitable with break-away walls in order to allow both debris flows and coastal flooding to pass through the ground floor of the residence. If our debris flow parameters are utilized in the design and construction of the proposed residence, the risk related to that hazard can be mitigated to an ordinary level.

Based upon our qualitative analysis, we conclude that liquefaction and lateral spreading may occur during the lifetime of the proposed deck and will create a greater than ordinary risk if is not adequately mitigated. We hasten to add, however, that our analysis is qualitative in nature. If the Project Geotechnical Engineer concludes that liquefaction is not a potential hazard, we will defer to that finding.

RECOMMENDATIONS

1. FEMA has determined that the base flood elevation is +21.0 feet NAVD88 and therefore the elevation of the bottom of the lowest horizontal structural members of new structures should be at or above this elevation **where warranted**. A wave force analysis should be performed for the project in order to evaluate the effect of coastal flooding on the proposed developments and the results should be used to establish design criteria. The structural elements below the habitable portion of the residence should be designed to withstand the impact of coastal waves, as well as the impact of battering objects caught up in the waves, such as large logs. The lower structural elements should also be

EXHIBIT G

designed for uplift forces from wave action in the event that sand accumulates under the residence.

2. Foundations for designed structures should be designed to resist the forces generated by liquefaction and lateral spreading where warranted, unless the project geotechnical engineer indicates that this is unnecessary.
3. All structures for the proposed development should be designed for a scour depth of +1.5 feet mean sea level (NAVD88), as portrayed upon Plate 2.
4. The proposed residence and appurtenant structures (decks, upgraded retaining walls, etc.) should consider the debris flow parameters listed under the "Landsliding" section of this report.
5. The owners or occupants of the residence should be prepared to accept the loss of all items stored on the ground floor and parked in the driveway, including vehicles. Additionally, they should be prepared to pay for replacement of the break-away walls on the lower story, since our analysis indicates that the property will be inundated by coastal waves and possibly by debris flows.
6. We recommend that our firm be provided the opportunity to review the final design and specifications in order that our recommendations may be properly interpreted and implemented in the design and specification. If our firm is not accorded the privilege of making the recommended review we can assume no responsibility for misinterpretation of our recommendations.

Further elaboration and supporting details accompanying the above findings and recommendations can be found in the body of the report. This report is issued with the understanding that it is the duty and responsibility of the owner or his representative or agent to ensure that the recommendations contained in this report are brought to the attention of the architect and engineer for the project, incorporated into the plans and specifications, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

If you have any questions or comments regarding this report, please contact us at your earliest convenience.

Sincerely,
ZINN GEOLOGY

Erik N. Zinn
Principal Geologist
P.G. #6854, C.E.G. #2139

CERTIFIED ENGINEERING GEOLOGIST
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STATE OF CALIFORNIA

EXHIBIT G

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PLATES - In pockets at back of report.
NOTE: Plates must accompany text of report in order for report to be considered complete.

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INTRODUCTION

This report presents the results of our coastal geologic investigation on the property located at 379 Beach Drive, Aptos, California (Figures 1 and 2). The purpose of our investigation was to evaluate the potential geologic hazards relevant to the construction of a new single-family residence intended to replace the existing residence on the subject property. Our investigation focused on the hazards and attendant risks primarily associated with the impacts of landsliding, seismic shaking, liquefaction, flooding due to coastal wave run-up and deep scour due to coastal wave erosion.

SCOPE OF INVESTIGATION

Work performed during this study included:

1. A review of published and unpublished literature relevant to proposed development on the subject property.

2. Examination and interpretation of eleven sets of historical stereo-pair vertical aerial photographs to assess the past effects of earthquakes and storms on the subject property.

3. Co-logging of small diameter borings advanced by the project geotechnical engineer, Pacific Crest Engineering, Inc. [PCEI]. The reader should refer to the Pacific Crest Engineering report to view a graphic depiction of the soil and bedrock encountered in the borings.

4. Construction of a geologic map and cross section for the property.

5. Meetings and telephone conversations with the design team, including the Project Geotechnical Engineer, Elizabeth Mitchell and Soma Goresky of PCEI, the Project Civil Engineer, Richard Irish of R.I. Engineering, and the project architect, Cove Britton of Matson-Britton Architects to discuss the preliminary results of our investigation.

6. Analysis and interpretation of the geologic data and preparation of this report.

REGIONAL GEOLOGIC SETTING

The subject property lies at the toe of a steep coastal bluff, straddling the intersection of the bluff with the broad beach fronting it near Rio Del Mar (Figures 1 and 2). The bluff behind the existing residence is one of many such bluffs along the northern coast of Monterey Bay, characterized by gently dipping, late Tertiary marine sedimentary rocks that are overlain by nearly horizontal, Quaternary terrace deposits chiefly of marine origin, as well as recently deposited beach sands and artificial fill.

The northern Monterey Bay coastal bluffs are generally vertical, ranging between 20 and 120 feet high. This indicates that the bluffs are subject to wave erosion periodically, since the Tertiary marine sedimentary rocks exposed in the bluff face are incapable of holding a vertical slope over geologic time, particularly considering the frequency of intense rainfall and seismic shaking events (which cause the slope to erode and fail to a less steep pitch).

A relatively wide and continuous beach between New Brighton State Beach (to the north) and La Selva Beach (to the south) appears to protect the coastal bluff from wave erosion most of the time. However, we can conclude, with a fair amount of certainty, that the waves attack the base of the bluff periodically (over geologic time), based upon the information cited in the previous paragraph and upon prior historical events (Griggs and Johnson, 1983). This dynamic geologic environment has had a severe impact in the past upon the integrity of structures that didn't take the frequency and magnitude of the sundry geologic threats (wave erosion, landsliding, seismic shaking and liquefaction) into account.

REGIONAL SEISMIC SETTING

California's broad system of strike-slip faulting has had a long and complex history. Some of these faults present a seismic hazard to the subject property. The most important of these are the San Andreas, Zayante(-Vergeles), San Gregorio and Monterey Bay-Tularcitos fault zones (Figures 3 and 4). These faults are either active or considered potentially active (Petersen et al., 1996; Working Group On Northern California Earthquake Potential [WGONCEP], 1996). Each fault is discussed below. Locations of epicenters associated with the faults are shown in Figure 3.

San Andreas Fault

The San Andreas fault is active and represents the major seismic hazard in northern California (Working Group on Northern California Earthquake Potential [NCEP], 1996). The main trace of the San Andreas fault trends northwest-southeast and extends over 700 miles from the Gulf of California through the Coast Ranges to Point Arena, where the fault extends offshore.

Geologic evidence suggests that the San Andreas fault has experienced right-lateral, strike-slip movement throughout the latter portion of Cenozoic time (the past 20 to 30 million years), with cumulative offset of hundreds of miles. Surface rupture during historical earthquakes, fault creep, and historical seismicity confirm that the San Andreas fault and its branches, the Hayward, Calaveras, and San Gregorio faults, are all active today.

Historical earthquakes along the San Andreas fault and its branches have caused significant seismic shaking in the Monterey Bay area. The two largest historical earthquakes on the San Andreas to affect the area were the moment magnitude (M_w) 7.9 San Francisco earthquake of 18 April 1906 (actually centered near Olema) and the M_w 6.9 Loma Prieta earthquake of 17 October 1989. The San Francisco earthquake caused severe seismic shaking and structural damage to many buildings in the Monterey Bay area. The Loma Prieta earthquake appears to have caused more intense seismic shaking than the 1906 event in localized areas of the Santa Cruz Mountains, even though its regional effects were not as extensive. There were also significant earthquakes in northern California along or near the San Andreas fault in 1838, 1865 and possibly 1890 (Sykes and Nishenko, 1984; NCEP, 1996).

Geologists have recognized that the San Andreas fault system can be divided into segments with "characteristic" earthquakes of different magnitudes and recurrence intervals (Working Group on California Earthquake Probabilities [WG], 1988 and 1990). A study by NCEP in 1996 has redefined the segments and the characteristic earthquakes for the San Andreas fault system in northern and central California. Two "locked" overlapping segments of the San Andreas fault system represent the greatest potential hazard to the property.

The first segment is defined by the rupture that occurred from Cape Mendocino to San Juan Bautista along the San Andreas fault during the great M_w 7.9 earthquake of 1906. The NCEP (1996) has hypothesized that this "1906 rupture" segment experiences earthquakes with comparable magnitudes at intervals of about two hundred years.

The second segment is defined by the rupture zone of the M_w 6.9 Loma Prieta earthquake. Although it is uncertain whether this "Santa Cruz Mountains" segment has a characteristic earthquake independent of great San Andreas fault earthquakes, the NCEP (1996) has assumed an "idealized" earthquake of M_w 7.0 with the same right-lateral slip as the 1989 Loma Prieta earthquake but having an independent segment recurrence interval of 138 years and a multi-segment recurrence interval of 400 years.

The 2002 WG (2003) segmentation model is largely similar to that adopted by NCEP in 1996, although they have added far more complexity to the model, and have reduced the forecasted magnitudes for the different segments. The 2002 California probabilistic seismic hazard maps issued by the California Geological Survey (Cao et al., 2003) appear to have largely adopted the earthquake magnitudes issued by the 2002 WG. The most significant change in modeling the San Andreas Fault Zone by Cao et al. (2003) is the elimination of a singular listing of the penultimate event, the 1906 Mw 7.9 earthquake (although such an event can be derived by looking at the aggregate probability of the individual segments rupturing together, as they did in 1906).

In spite of the increasing complexity of the models addressing different size earthquakes with different recurrence intervals on the sundry segments of this fault, it is undeniable that the 1906 M_w 7.9 earthquake still eclipses all the other events which have occurred on the San Andreas fault in this region. Keeping this in mind, it is important that any site-specific seismic analyses performed for development on the property take the 1906 event into account, particularly since the empirical evidence presented by field researchers indicates the 1906 event recurs every several centuries.

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Zayante (-Vergeles) Fault

The Zayante fault lies west of the San Andreas fault and trends about 50 miles northwest from the Watsonville lowlands into the Santa Cruz Mountains. The southern extension of the Zayante fault, known as the Vergeles fault, merges with the San Andreas fault south of San Juan Bautista.

The Zayante-Vergles fault has a long, well-documented history of vertical movement (Clark and Reitman, 1973), probably accompanied by right-lateral, strike-slip movement (Hall et al., 1974; Ross and Brabb, 1973). Stratigraphic and geomorphic evidence indicates the Zayante-Vergles fault has undergone late Pleistocene and Holocene movement and is potentially active (Buchanan-Banks et al., 1978; Coppersmith, 1979).

Some historical seismicity may be related to the Zayante-Vergles fault (Griggs, 1973). For instance, the Zayante-Vergles fault may have undergone sympathetic fault movement during the 1906 earthquake centered on the San Andreas fault, although this evidence is equivocal (Coppersmith, 1979). Seismic records strongly suggest that a section of the Zayante-Vergles fault approximately 3 miles long underwent sympathetic movement in the 1989 earthquake. The earthquake hypocenters tentatively correlated to the Zayante-Vergles fault occurred at a depth of 5 miles; no instances of surface rupture on the fault have been reported.

In summary, the Zayante-Vergles fault should be considered potentially active. The NCEP (1996) considers it capable of generating a magnitude 6.8 earthquake with an effective recurrence interval of 10,000 years. Alternatively, Cao et al. (2003) considers this fault capable of generating a maximum earthquake of Mw 7.0, with no stated recurrence interval.

San Gregorio Fault

The San Gregorio fault, as mapped by Greene (1977), Weber and Lajoie (1974), and Weber et al. (1995) skirts the coastline of Santa Cruz County northward from Monterey Bay, and trends onshore at Point Año Nuevo. Northward from Año Nuevo, it passes offshore again, to connect with the San Andreas fault near Bolinas. Southward from Monterey Bay, it may trend onshore north of Big Sur (Greene, 1977) to connect with the Palo Colorado fault, or continue southward through Point Sur to connect with the Hosgri fault in south-central California. Based on these two proposed correlations, the San Gregorio fault zone has a length of at least 100 miles and possibly as much as 250 miles.

The landward extension of the San Gregorio fault at Point Año Nuevo shows evidence of late Pleistocene (Buchanan-Banks et al., 1978) and Holocene displacement (Weber and Cotton, 1981). Although stratigraphic offsets indicate a history of horizontal and vertical displacements, the San Gregorio is considered predominantly right-lateral strike slip by most researchers (Greene, 1977; Weber and Lajoie, 1974; and Graham and Dickinson, 1978).

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In addition to stratigraphic evidence for Holocene activity, the historical seismicity in the region is partially attributed to the San Gregorio fault (Greene, 1977). Due to inaccuracies of epicenter locations, even the magnitude 6+ earthquakes of 1926, tentatively assigned to the Monterey Bay fault zone, may have actually occurred on the San Gregorio fault (Greene, 1977).

The NCEP (1996) has divided the San Gregorio fault into the "San Gregorio" and "San Gregorio, Sur Region" segments. The segmentation boundary is located west of the Monterey Bay, where the fault appears to have a right step-over. The San Gregorio fault has been assigned a slip rate that results in a M_w 7.3 earthquake with a recurrence interval of 400 years. This is based on the preliminary results of a paleoseismic investigation at Seal Cove by Lettis and Associates (see NCEP, 1996) and on regional mapping by Weber et al. (1995). The Sur Region segment has been assigned a slip rate that results in a M_w 7.0 earthquake with an effective recurrence interval of 400 years (coinciding with the recurrence interval for the other segment). The Sur Region earthquake was derived from an assumed slip rate similar to that of the Hosgri fault.

2002 WG and Cao et al. (2003) has adopted a model similar to the NCEP (1996), essentially renaming the San Gregorio segment the “San Gregorio North” segment, and downgrading the forecasted earthquake on this segment to a Mw 7.2, and renaming the San Gregorio, Sur Region segment the San Gregorio South segment, retaining the forecasted earthquake of Mw 7.0.

Monterey Bay-Tularcitos Fault Zone

The Monterey Bay-Tularcitos fault zone is 6 to 9 miles wide, about 25 miles long, and consists of many en échelon faults identified during shipboard seismic reflection surveys (Greene, 1977). The fault zone trends northwest-southeast and intersects the coast in the vicinity of Seaside and Ford Ord. At this point, several onshore fault traces have been tentatively correlated with offshore traces in the heart of the Monterey Bay-Tularcitos fault zone (Greene, 1977; Clark et al., 1974; Burkland and Associates, 1975). These onshore faults are, from southwest to northeast, the Tularcitos-Navy, Berwick Canyon, Chupines, Seaside, and Ord Terrace faults. Only the larger of these faults, the Tularcitos-Navy and Chupines, are shown on Figure 2. It must be emphasized that these correlations between onshore and offshore portions of the Monterey Bay-Tularcitos fault zone are only tentative; for example, no concrete geologic evidence for connecting the Navy and Tularcitos faults under the Carmel Valley alluvium has been observed, nor has a direct connection between these two faults and any offshore trace been found.

Outcrop evidence indicates a variety of strike-slip and dip-slip movement associated with onshore and offshore traces. Earthquake studies suggest the Monterey Bay-Tularcitos fault zone is predominantly right-lateral, strike-slip in character (Greene, 1977). Stratigraphically, both offshore and onshore fault traces in this zone have displaced Quaternary beds and, therefore, are considered potentially active (Buchanan-Banks et al., 1978). One offshore trace, which aligns with the trend of the Navy fault, has displaced Holocene beds and is therefore active by definition (Buchanan-Banks et al., 1978).

Seismically, the Monterey Bay-Tularcitos fault zone may be historically active. The largest historical earthquakes *tentatively* located in the Monterey Bay-Tularcitos fault zone are two events, estimated at 6.2 on the Richter Scale, in October 1926 (Greene, 1977). Because of possible inaccuracies in locating the epicenters of these earthquakes, it is possible that they actually occurred on the nearby San Gregorio fault zone (Greene, 1977). Another earthquake in April 1890 might be attributed to the Monterey Bay-Tularcitos fault zone (Burkland and Associates, 1975).

The NCEP (1996) has assigned an earthquake of M_w 7.1 with an effective recurrence interval of 2,600 years to the Monterey Bay-Tularcitos fault zone, based on Holocene offshore offsets. Petersen et al. (1996) have a similar earthquake magnitude, but for a recurrence interval of 2,841 years. Their earthquake is based on a composite slip rate of 0.5 millimeters per year (after Rosenberg and Clark, 1995).

Cao et al. (2003) has developed a model for the Monterey Bay fault zone that combines slip rates of the different segments, resulting in a composite slip rate of 0.5 mm per year and a forecasted earthquake of Mw 7.3, with no stated recurrence interval. The Cao et al. (2003) model adopted implicitly assumes that all the assessed segments in the Monterey Bay fault zone each have an independent slip rate of 0.1 mm per year (based upon the one slip rate developed by Rosenberg and Clark, 1995 for the Tularcitos segment), and essentially assigns the composite slip rate to the Tularcitos trace of the Monterey Bay fault zone.

SITE GEOLOGIC SETTING

The Geologic Site Map (Plate 1) and Geologic Cross Section (Plate 2) graphically depict relevant geologic information for the subject property. See also the Local Geology Map (Figure 5) for information of a more general nature.

Topography

The subject property mostly occupies a very steep coastal bluff (Figures 1 and 2; Plates 1 and 2). The southwestern end of the property just touches upon a very-gently seaward-dipping beach surface that is traversed by Beach Drive. The property is currently developed with an existing residence, which is notched into the coastal bluff at the intersection of the coastal bluff and the beach (Plates 1 and 2). The ground surface on and above the property slopes very steeply landward from the residence to the top of the coastal bluff, with a total vertical throw of approximately 120 feet.

The coastal bluff above the existing residence is overall a concave-in slope (i.e. bowl-shaped), but is cut by broad shallow drainage swales that are in turn pockmarked by multiple zero-order scallops and some landslide deposits. The best description overall for the topography of the coastal bluff above the residence is "hummocky".

The concave-in slope character of the coastal bluff creates a bowl shape that directs flow and drainage toward the existing and proposed residential location (the red stippled area labeled "Debris Flow/Landslide Source Area And Runout Area" on Plate 1). Drainage and debris flows within the bowl will move toward the residence and will strike the residence if the deposit travels far enough.

There is distinct change in slope gradient from west to east across the property between elevation 40 and 50 feet, marked by relatively lower slope gradients below the break. This topographic change marks the transition from past erosion and landsliding terrain above to the blanket of soil that has been deposited near the base of the bluff as a result of the erosion and landsliding coming to rest on the beach below.

The upper coastal bluff on the adjacent up slope property stabilized with Geobrugg Tecco System from approximately 105 feet to 145 feet. Our firm worked in conjunction with R.I. Engineering, PCEI and Haro, Kasunich & Associates to develop the layout and design of the system for the adjacent property owner.

Earth Materials

Brabb (1997, Figure 4; see also Plates 1 and 2) has mapped the subject property as being underlain by Purisima Formation bedrock, which is partially consistent with our findings. Based upon the data procured from our outcrop mapping and the small-diameter borings advanced by PCEI, the southwestern end of the property (the portion proposed for development) is underlain by about 16 to 22 ½ feet of relatively looser, well sorted, well rounded, fine-grained sand to coarse pebbly-sand. The package of relatively looser sand can be further subdivided into a separate layer of beach sand that is least 16 feet thick at the southwestern property line (see Plate 2). The two formations must interfinger with one another somewhere underneath the existing residence, based upon the results of the borings and the nature of the development of the colluvial wedge at the base of the bluff over geological time.

The bluff above the existing residence appears to be underlain by Purisima Formation sandstone, with an inconsistent blanket of colluvium that ranges from no coverage whatsoever to five feet. Near the top of the bluff, we have assumed that a deposit of marine terrace deposits of approximately 40 feet thick caps the Purisima Formation bedrock, based upon the results of our investigation for the property and the slope inflection present near elevation 105 that likely marks the contact between the two formations.

The Purisima Formation (Tp) is described by Brabb (1997) as consisting of very thick bedded, yellowish gray, tuffaceous and diatomaceous siltstone containing thick interbeds of bluish-gray, semi-friable, andesitic sandstone. We have also noted that the site is located very near the area where the Aromas Formation is depicted as lapping up onto the Purisima Formation, and that the underlying bedrock might possibly be the Aromas Formation. The Aromas Formation in this area is described by Brabb (1997) as consisting of moderately well sorted eolian sand with a highly variable degree of consolidation owing to differential weathering. The outcrops of the upper portion of the bluff and the small-diameter borings advanced by PCEI exposed thinly to very thinly bedded, cross bedded at times, nearly flat-lying, dense to very dense, interbedded and interfingering, well rounded, predominantly well sorted (poorly graded) fine- to coarse-grained sandstone and coarse pebbly sandstone. Based upon this observation, it is our opinion that the site is underlain by Purisima Formation bedrock. A predominant sub-vertical joint set on one- to two-foot spacing that is roughly parallel to the coastal bluff was observed in outcrops upcoast and downcoast of the property. It appears that the orientation and geometry of the coastal bluff portion of the property has largely been controlled by the predominant joint set a time long ago (thousands of years?) when the formation of the bluff was created predominantly by wave erosion.

The elevation and geometry of the contact between the bedrock and the overlying beach sand is one of the more important geologic parameters to be considered. The contact between the two units, portrayed upon Plate 2, was chosen solely upon the data from the small-diameter borings and appears to lie at one and half feet mean sea level (+1.5' msl) (NAVD88) . The contact between the two units was marked by a contrast in drilling consistency (more difficult within the Purisima Formation), grain size (the beach sand was slightly coarser grained than the Purisima Formation) and a change in standard penetration blow counts (higher blow counts recorded in the Purisima Formation correlative to a higher relative density).

As noted previously, the Purisima Formation is capped by marine terrace deposits above the property. We presume that the wave cut platform that marks the contact between the two units dips very gently seaward (several degrees) as it does elsewhere in this region, although it should be noted that we did not observe or measure it directly in outcrops above the site. Based upon some assumptions and our observation of the colluvium derived from the marine terrace deposits, they are composed of a flat-lying, sub-rounded, predominantly well sorted (poorly graded), fine- to coarse-grained sand and coarse pebbly sand. We assumed that the total thickness of this unit is approximately r0 feet, based upon our experience with project that lies above the subject property (above the site; see Plate 2).

The marine terrace deposits are typically overprinted by a laterally-discontinuous, weakly developed pedogenic soil, composed of an A-soil horizon (silty sand to sandy silt), underlain by a B-soil horizon (sand with clay). The geometry of the pedogenic soil typically mirrors the morphology of the site with an aggregate thickness of about five to eight feet. Please note that we did not map the distribution of the pedogenic soil on our Geologic Site Map (Plate 1) or our Geologic Cross Section (Plate 2), since it is not germane to any of our findings or recommendations regarding geological hazards.

Up to five feet of colluvium blankets both parent formations (Purisima Formation and the marine terrace deposits) on the coastal bluff in a patchy fashion and appears to thicken considerably near

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the base of the bluff, where it presumably interfingers with the buried beach sand deposits at the base of the bluff (see Plate 2). The colluvium exposed on the coastal bluff is composed of an unconsolidated mixture of sand of varying grain sizes, derived from mass wasting of marine terrace deposits and Purisima Formation sandstone out of the coastal bluff. It is thickest at near the base of the bluff and the top of the bluff, thinning on the steeper portions of the bluff in the middle where it appears to have been the source of past debris flows.

Although we haven't mentioned it thus far, an extremely broad sandy beach, Beach Drive and a seawall currently fronts the subject property and the coastal bluff. The beach appears to exceed several hundred feet in width, even during the winter time, based upon our aerial photo analysis. Although the presence of this broad beach will have no immediate impact upon the design of the residence, its' existence has a profound impact on the geometry and long-term retreat rate of the coastal bluff. We will discuss this impact in subsequent sections of the report.

Drainage and Groundwater

Drainage at the site is primarily by sheet flow to the west-southwest toward the Monterey Bay.

Groundwater levels encountered in the small diameter borings advanced by PCEI ranged between five feet above mean sea level at the southernmost edge of the property to eight feet above mean sea level behind the existing residence. We presume that the lower groundwater elevation is controlled by ocean tides and waves that drive daily attenuated fluctuations within the beach sand, while the higher ground water elevation is likely related to the interplay of regional groundwater moving westward and the aforementioned tidally-influenced groundwater.

It is important to note that there will be times, presumably during coastal flooding or seasonally high-tides when the groundwater will be virtually at the ground surface in the area of development. Any foundation design or liquefaction analysis should take this into account.

GEOLOGIC HAZARDS

In our opinion, the primary geologic hazards that could potentially affect the proposed residence are: 1) coastal flooding, 2) coastal erosion, 3) intense seismic shaking, 4)landsliding from the coastal bluff on the subject property and 5) liquefaction and lateral spreading. The controlling geologic hazards for the project are coastal flooding and erosion, which when adequately mitigated, will reduce the risk due to the other hazards to ordinary.

Coastal Flooding

The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) Community Panel number 06087C0359F portrays the property as being within the limit of the floodway flood zone VE. FEMA has calculated a coastal flood 100-year base flood elevation of

+21.0 feet (above mean sea level; NAVD 88) for this zone. Since the ground surface is no higher than about +17½ feet NAVD in the developable portions of the property, the risk to any new structures constructed at or near the existing grade due to coastal flooding is clearly greater than ordinary for that particular flood elevation.

We have not included a detailed discussion of the storm history for this region, since that is typically done for sites where a site-specific coastal wave run-up analysis is performed, and there is no need to perform such an analysis, since FEMA has effectively performed that analysis for this stretch of coastline. As noted above, FEMA has determined that the base flood elevation is +21.0 feet NAVD 88. FEMA requires that the elevation of the bottom of the lowest horizontal structural member of the lowest floor be at or above this elevation.

It is important to note that coastal flooding due to coastal wave run up will break away the walls on the lower story (below +21.0 feet NAVD 99) and will damage the contents therein.

Coastal Erosion

The existing grade for the proposed development area is protected from erosion by a sea wall of unknown design that lies southwest of the property, on the other side of Beach Drive. The embedment depth and foundation type for the wall is unknown at this stage of our investigation. As noted in the earth materials section, the proposed development area is underlain by a blanket of beach sand. The contact between the beach sand and the underlying bedrock is about one and half feet above mean (1.5' msl NAVD 88). This is important to note, because the contact between the beach sand and bedrock marks the former scour elevation for the property, which makes it a possibility that a large coastal storm could scour out the beach sand to that depth again, particularly when considering the impacts of continuing rising sea levels and intensity and frequency of large storms. If the sea wall, as well as the proposed foundation for the new residence are not designed to withstand that depth of scour, then the foundation elements will be undermined and will catastrophically collapse. It is also important to note that such an extreme scour depth will expose the foundation elements to battering by objects caught up in breaking waves such as logs. For the sake of simplicity and conservatism, we recommend that the project be designed for a scour depth of 1.5 feet above msl (NAVD 88) across the lower portion of the property.

Seismic Shaking Hazard

Seismic shaking at the subject site will be intense during the next major earthquake along one of the local fault systems. It is important that the recommendations regarding seismic shaking be considered in the design for the proposed developments where applicable. The proposed development will be geologically suitable, if it is designed and constructed in conformance with the seismic parameters issued in the PCEI report, where warranted.

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Landsliding

Numerous landslides have historically occurred in the bluff along Beach Drive, based upon our aerial photo analysis and experience with this area. The landslides issuing from the bluffs are typically fluid debris flows and occur within the marine terrace deposits at the top of the bluff, and the weathered and fractured "rind" forming within the bedrock in the face of the bluff. The triggering event is typically a strong winter storm accompanied by an intense rainfall event, such as occurred in this area in January 1982, the winter of 1995 and the winters of 1996-1997, 1997-1998 and 2016-2017. Homes constructed at the base of the bluff have been damaged by these events; in cases where there are no homes at the base of the bluff to act as impromptu impact walls, debris flows have run out as far as 100 feet from the base of the bluff before they come to rest (based upon our aerial photo research and file research).

In the case of the subject property, there are clearly developed debris flow scar zones above the existing residence. These scars are the result of multiple debris flow events that have occurred over geological time, resulting in broad, hummocky ill-defined drainage swales that represent a coalescence of the scars from past debris flows.

The source of future debris flows can lie anywhere on the bluff above the property where colluvium or marine terraces are exposed. A reasonably conservative approach to assessing the debris flow hazard and attendant risk is to design for debris flows that have issued from near the top of the bluff, where the colluvial soils and marine terrace deposits are exposed on a slope that is too steep for those soils under saturated conditions, and possibly when subjected to large-magnitude long duration earthquakes.

The existing walls and benches that lie up slope and behind the existing residence provide little to no protection from future debris flows in our opinion.

As noted in an earlier section, a portion of the upper bluff is stabilized with Geobrugg Tecco System on the up slope property. Although this does somewhat help improve the overall likelihood of future debris flows initiating from the up slope property and striking a residence on the subject property, it does not entirely eliminate the threat. This is because only a portion of the bluff above the subject property is protected and the bluff is bowl-shaped, resulting in a larger contributory area for potential debris flows than just the area immediately up slope. Plate 1 graphically depicts the portion of the slope above the residence that could potentially generate debris flows that could strike a residence on the subject property (stippled red and labeled "Debris Flow/Landslide Source Area And Runout Area).

It is our understanding that the design of the proposed residence is ongoing. As noted above, we have plotted the potential debris flow source and runout area upon our Geological Map (Plate 1). We have also plotted the area on the subject property that could potentially be stabilized with some form of an engineered slope stabilization system (shaded orange and labeled "Potential

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Slope Are To Be Stabilized). It is important to note that even if the entire slope on the property is stabilized with a system that ties into the existing Geobrugg Tecco System on the up slope property, there will still be debris flow source areas to either side of the property that will generate debris flows that will impact the proposed residence.

We have previously discussed these concepts with the design team and disseminated our draft map and cross section along with our recommendations. It was collectively decided that the most cost effective way to reduce the risk resulting from debris flow hazards was to make the ground story of the proposed residence non-habitable with break away walls. This design premise would also mitigate the risk due to coastal flooding.

Given the above observations, we have run through some potential debris flow/shallow landslide scenarios for the site, based upon our geological analysis of the bluff. The types of failure we have issued scenario parameters for are as follows:

1. Arcuate failure near elevation 112 feet above mean sea level, involving colluvium, marine terrace deposits and weathered Purisima Formation where it is not protected by Geobrugg Tecco;
2. 5' Thick Planar failure or translational slide along the bluff face during an earthquake - generating a debris flow/sand flow
3. Segmented Planar failure of the bluff face during intense rainfall - 10 feet thick in the marine terrace deposits and 5 feet thick in the Purisima Formation generating a debris flow 10'x10'x30' (marine terrace deposits) plus 5'x40'x20' initially (an aggregate of 259 cubic yards)

Arcuate Failure
Drop Height = 88 feet
Velocity At Impact = 38 feet per second
Area Of Soil At Impact Back Of House = 5' high by 25' wide = 125 square feet
Area Of Soil After Soil Stops Moving = 8' high by 25' wide = 200 square feet
Maximum Angle Between Direction Of Slide Movement With Respect To Back Of House = 90 degrees (seaward portions of the house that can be struck lie at an oblique angle that is less than 90 degrees)

5' Thick Planar Failure (seismic)
Drop Height = 88 feet
Velocity At Impact = 15 feet per second
Area Of Soil At Impact With Back Of House = 5' high by 25' wide = 125 square feet
Area Of Soil After Soil Stops Moving = 8' high by 25' wide = 200 square feet
Angle Between Direction Of Slide Movement With Respect To Back Of House = 90 degrees (seaward portions of the house that can be struck lie at an oblique angle that is less than 90 degrees)

Segmented Planar Failure (Saturated)
Drop Height = 88 feet
Velocity At Impact = 35 feet per second
Area Of Soil At Impact With Back Of House = 5' high by 25' wide = 125 square feet
Area Of Soil After Soil Stops Moving = 8' high by 25' wide = 200 square feet
Angle Between Direction Of Slide Movement With Respect To Back Of House = 90 degrees (seaward portions of the house that can be struck lie at an oblique angle that is less than 90 degrees)

We have reviewed recent debris flow research papers for some indication of the morphology of the debris flow train as it passes through a given spot. Most of the papers reviewed address very steep and high, hard bedrock landscapes with v-shaped channels carved into the mountains (in locations such as British Columbia, Switzerland, the upper elevations of the Rockies, the mountains surrounding the Los Angeles basin and the western range front of Big Sur). Additionally, the papers are also dealing with debris flows that occur in regions that are subject to flash flooding from snow melt or thunder showers. In our opinion, those conditions do not really applies to the coastal bluff landward of Beach Drive.

In this instance, we will need to utilize empirical data and our professional judgement born of our past experience, mostly based on the mud line and run out distances of the debris flows along Beach Drive.

The aggregate debris flow event typically occurs incrementally over short spans of time, due to the first increment of the debris flow event leaving a scar with steep margins that subsequently removes lateral support of the surrounding saturated soil. That is why the total remnant of fresh debris flow scars are uneven and have a "fluted" appearance after a debris flow event. If you look closely at the debris flow deposits that have been mapped in the Beach Drive area, you will notice that they are not perfectly smooth and conical, such as you might expect from a singular event. Instead, they are lumpy and unevenly lobate, which is indicative of deposition by multiple deposits.

Splash lines in the Beach Drive area are typically four to five feet lower than the deposit. This indicates a fairly fluid rheology of the debris flows sourced from colluvium derived from the underlying sandy soil formation belonging to the marine terrace deposits and the sandy soil derived from the weathering of the underlying Purisima Formation bedrock. Debris flow deposit mud lines of approximately eight to ten feet high are common for Beach Drive debris flows that travel from near the top of the bluff and strike the structures orthogonally.

The debris flows in the Beach Drive area do share some similarity to other debris flows from different geological terranes around the world. The debris flows are segmented and they all sort themselves the same, grading from coarse in the front of the flow mass to finer in a progressive

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fashion toward back of the flow mass. Overall the debris flow masses are tear-shaped in cross section and the snouts are typically the highest portion of the flow mass.

For the this project, the design debris flow for a hypothetical new proposed impact wall or debris flow fence will initiate at about 112 feet above mean sea level and travel approximately 88 vertical feet before it strikes the hypothetical house. In order to travel that distance, the flow will have to be very fluid and moving fast. In our opinion, the debris flows that would strike the house will have snouts that will be three to six feet high.

If a Geobrugg debris flow barrier will be used on the site, the following geological parameters should be used:

Total debris volume = 259 cubic yards
Debris flow peak velocity = 38 feet per second
Number of surges = 2
Volume of first surge = 172 cubic yards
Required retention volume = 259 cubic yards

In summary, it is our opinion that the proposed residence will be subject to a greater than ordinary risk related to the debris flow and landslide hazard. In our opinion the risk to the proposed residence can be adequately reduced to ordinary if a debris flow impact wall or barrier is constructed up slope of the residence. Alternatively, the lower floor of the residence can be designed as non-habitable with break-away walls in order to allow both debris flows and coastal flooding to pass through the ground floor of the residence.

Liquefaction And Lateral Spreading

The physical process of seismically induced liquefaction has been documented by numerous researchers (Youd, 1973; Seed and Idriss, 1982; National Research Council, 1985). During an earthquake seismic waves travel through the earth and vibrate the ground. In cohesionless, granular materials having low relative density (loose sands for example), this vibration can disturb the particle framework, thus leading to increased compaction of the material and reduction of pore space between the framework grains. If the sediment is saturated, water occupying the pore spaces resists this compaction and exerts pore pressure that reduces the contact stress between the sediment grains. With continued shaking, transfer of intergranular stress to pore water can generate pore pressures great enough to cause the sediment to lose its strength and change from a solid state to a liquefied state. This mechanical transformation can cause various kinds of ground failure at or near the ground surface.

The liquefaction process typically occurs at depths less than 50 feet below the ground surface, although liquefaction can occur at deeper intervals, given the right conditions. The most susceptible zone occurs at depths shallower than 30 feet below the ground surface. Diminished susceptibility as depth increases is due to the increased firmness of deeper sedimentary materials, which can be attributed mainly to two factors: 1) increased overburden pressure resulting from the load of overlying sediment layers, and 2) increased geologic age. These two factors tend to create a denser packing of sediment grains in the deeper sedimentary materials, which thus are less likely to experience the additional compaction and elevated pore pressures that are necessary to induce loss of shear strength and liquefaction during an earthquake.

Liquefaction can lead to several types of ground failure, depending on slope conditions and the geologic and hydrologic setting (Seed, 1968; Youd, 1973; Tinsley et al, 1985). The four most common types of ground failure are: 1) lateral spreads, 2) flow failures, 3) ground oscillation and 4) loss of bearing strength. Sand boils (injections of fluidized sediment) commonly accompany these different types of ground failure and form sand volcanoes at the ground surface or convolute layering and sand dikes in subsurface sediment layers.

Dupré (1975) has mapped the beach sand deposits in the Beach Drive area as having a high potential for liquefaction. As noted in our Earth Materials section, the entire property is blanketed by a layer of beach sand that ranges between 25 and 26 feet thick. There will be times in the future when groundwater on the property will nearly be at the ground surface and, as noted in the Seismic Shaking section, the property will likely be subjected to at least one or more large magnitude earthquakes on one of the nearby fault zones. Additionally, a geological consultant report for some nearby Beach Drive properties (Foxx, Nielsen and Associates, 1999) documented some evidence of minor ground cracking (likely due to liquefaction and lateral spreading) occurring within the beach sand in this region during the 1989 Loma Prieta earthquake.

Based upon this qualitative analysis, we conclude that liquefaction and lateral spreading may occur during the lifetime of the proposed residence and will create a greater than ordinary risk if is not adequately mitigated. We hasten to add, however, that our analysis is qualitative in nature. If the Project Geotechnical Engineer performs a more robust quantitative liquefaction analysis that concludes that liquefaction is not a potential hazard, we will defer to that conclusion.

FINDINGS

Based on the information gathered and analyzed in the steps outlined above, it is our opinion that the proposed residence will be geologically suitable, provided our recommendations are adequately adhered to by the design team. The proposed residence will be subject to "ordinary" risks as defined in Appendix B, provided our recommendations are followed. Appendix B should be reviewed in detail by the developer and all property owners to determine whether an "ordinary" risk as defined in the appendix is acceptable. If this level of risk is unacceptable to the developer and the property owners, then the geologic hazards in question should be mitigated to reduce the corresponding risks to an acceptable level.

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The most recent issue of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) Community Panel number 066087C0359F portrays the property as being within the limit of the floodway flood zone VE. FEMA has calculated a coastal base flood elevation of +21.0 feet above mean sea level (NAVD88) for this zone. Since the ground surface is no higher than about +17½ feet NAVD 88 in the developable portions of the property, the risk to any new structures constructed at or near the existing grade due to coastal flooding is clearly greater than ordinary for that particular flood elevation.

The contact between the beach sand and the underlying bedrock is about 1.5 feet above mean sea level (NAVD88) and this contact marks the former scour elevation for the property. Future storms may scour out the beach sand to that depth again, particularly when considering the impacts of continuing rising sea levels and intensity and frequency of large storms. It is also important to note that such an extreme scour depth will expose the foundation elements embedded in the loose sandy soils to battering by objects caught up in breaking waves such as logs. For the sake of simplicity and conservatism, we recommend that the future structures be designed for a scour depth of +1.5 feet NAVD88 across the lower portion of the property, **where warranted or required**. If design for scour is not required for the structures being considered (such as the case of the building something high up on the coastal bluff), this finding and accompanying recommendation can be ignored.

Seismic shaking at the subject site will be intense during the next major earthquake along one of the local fault systems. It is important that the recommendations regarding seismic shaking be considered in the design for the proposed developments where applicable. The proposed development will be geologically suitable, if it is designed and constructed in conformance with the seismic parameters issued in the PCEI report, where warranted.

It is our opinion that the proposed residence will be subject to a greater than ordinary risk related to the debris flow and landslide hazard. In our opinion the risk to the proposed residence can be adequately reduced to ordinary if a debris flow impact wall or barrier is constructed up slope of the residence. Alternatively, the lower floor of the residence can be designed as non-habitable with break-away walls in order to allow both debris flows and coastal flooding to pass through the ground floor of the residence. If our debris flow parameters are utilized in the design and construction of the proposed residence, the risk related to that hazard can be mitigated to an ordinary level.

Based upon our qualitative analysis, we conclude that liquefaction and lateral spreading may occur during the lifetime of the proposed deck and will create a greater than ordinary risk if is not adequately mitigated. We hasten to add, however, that our analysis is qualitative in nature. If the Project Geotechnical Engineer concludes that liquefaction is not a potential hazard, we will defer to that finding.

RECOMMENDATIONS

1. FEMA has determined that the base flood elevation is +21.0 feet NAVD88 and therefore the elevation of the bottom of the lowest horizontal structural members of new structures should be at or above this elevation **where warranted**. A wave force analysis should be performed for the project in order to evaluate the effect of coastal flooding on the proposed developments and the results should be used to establish design criteria. The structural elements below the habitable portion of the residence should be designed to withstand the impact of coastal waves, as well as the impact of battering objects caught up in the waves, such as large logs. The lower structural elements should also be designed for uplift forces from wave action in the event that sand accumulates under the residence.

2. Foundations for designed structures should be designed to resist the forces generated by liquefaction and lateral spreading where warranted, unless the project geotechnical engineer indicates that this is unnecessary.

3. All structures for the proposed development should be designed for a scour depth of +1.5 feet mean sea level (NAVD 88), as portrayed upon Plate 2.

4. The proposed residence and appurtenant structures (decks, upgraded retaining walls, etc.) should consider the debris flow parameters listed under the "Landsliding" section of this report.

5. The owners or occupants of the residence should be prepared to accept the loss of all items stored on the ground floor and parked in the driveway, including vehicles. Additionally, they should be prepared to pay for replacement of the break-away walls on the lower story, since our analysis indicates that the property will be inundated by coastal waves and possibly by debris flows.

6. We recommend that our firm be provided the opportunity to review the final design and specifications in order that our recommendations may be properly interpreted and implemented in the design and specification. If our firm is not accorded the privilege of making the recommended review we can assume no responsibility for misinterpretation of our recommendations.

EXHIBIT G

INVESTIGATIVE LIMITATIONS

1. Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology principles and practices. No warranty, expressed or implied including any implied warranty of merchantability or fitness for the purpose is made or intended in connection with our services or by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

2. The analysis and recommendations submitted in this report are based on the geologic information derived from the steps outlined in the scope of services section of this report. The information is derived from necessarily limited natural and artificial exposures. Consequently, the conclusions and recommendations should be considered preliminary.

3. The conclusions and recommendations noted in this report are based on probability and in no way imply the site will not possibly be subjected to ground failure or seismic shaking so intense that structures will be severely damaged or destroyed. The report does suggest that building structures at the subject site, in compliance with the recommendations noted in this report, is an "ordinary" risk as defined in Appendix B.

4. This report is issued with the understanding that it is the duty and responsibility of the owner or his representative or agent to ensure that the recommendations contained in this report are brought to the attention of the architect and engineer for the project, incorporated into the plans and specifications, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

5. The findings of this report are valid as of the present date. However, changes in the conditions of property and its environs can occur with the passage of time, whether they be due to natural processes or to the works of man. 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, the conclusions and recommendations contained in this report cannot be considered valid beyond a period of two years from the date of this report without review by a representative of this firm.

EXHIBIT G

REFERENCES

Aerial Photographs

DATE FLOWN	FLIGHT LINE	PHOTO NUMBERS	PRINTS
1928	SC	20, 21 & 22	Black & white
January 1935		95, 96 & 97	Black & white
05/14/48	CDF5-3	52 & 53	Black & white
05/11/65	USACE 1	53,54 & 55	Black & white
10/14/75	SCZCO 1	81 & 82	Black & white
05/06/78	CA Dept. P&R	113 & 114	Color
01/07/82	USGS 12	3 & 4	Black & white
10/26/89	USGS 11	3 & 4	Black & white
05/14/90	WAC 9	197 & 198	Black & white
08/29/98	USDA 10532	61 & 62	Black & white
06/26/03	AMBAG 323	11 & 12	Color

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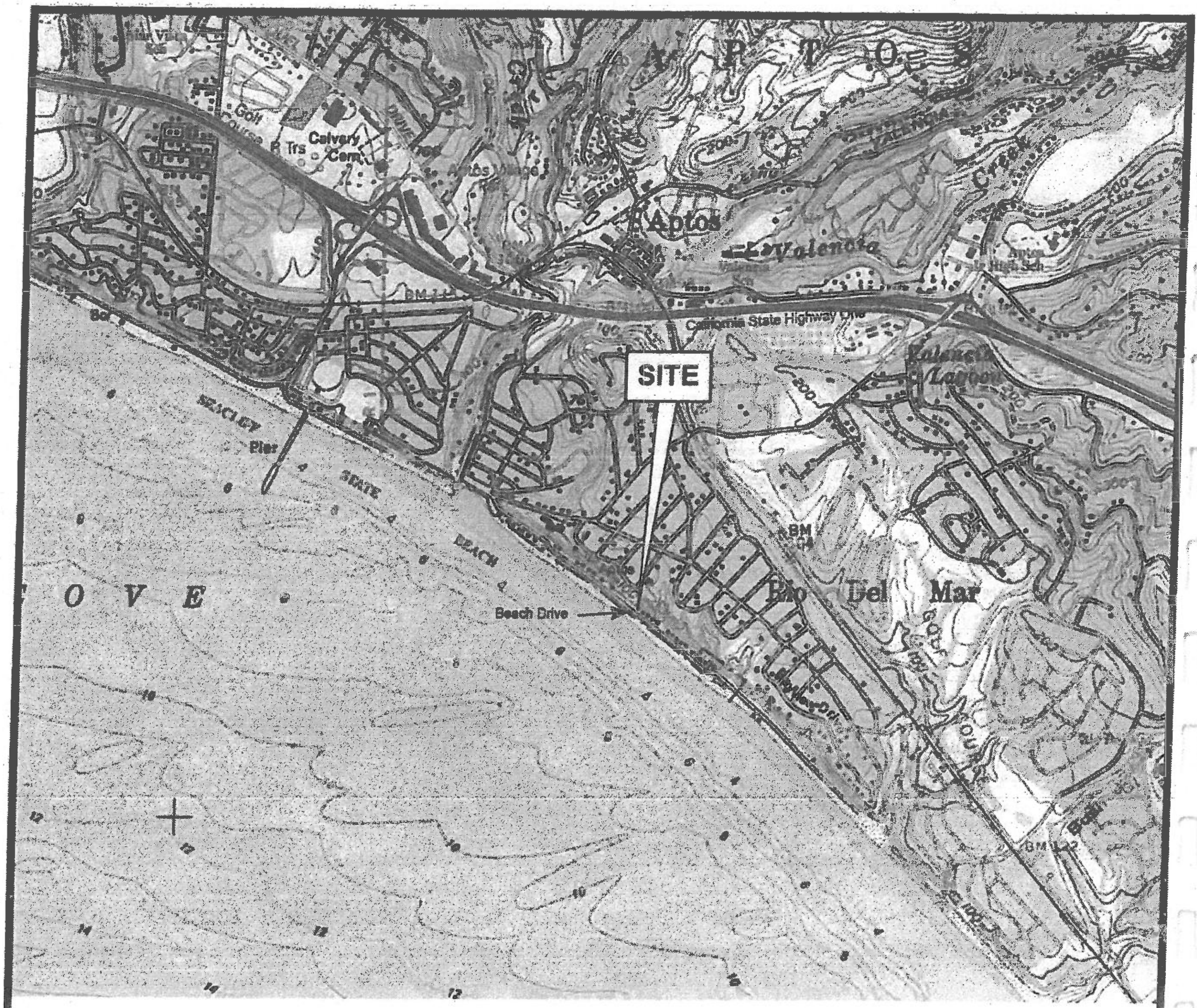
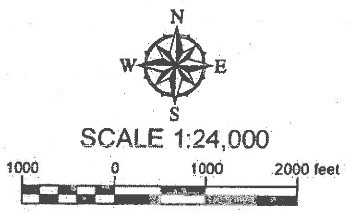
APPENDIX A

FIGURES



BASE MAP: U.S. Geological Survey, 1954 (photorevised 1980), Soquel quadrangle, California, 7.5' topographic series, scale 1:24,000.

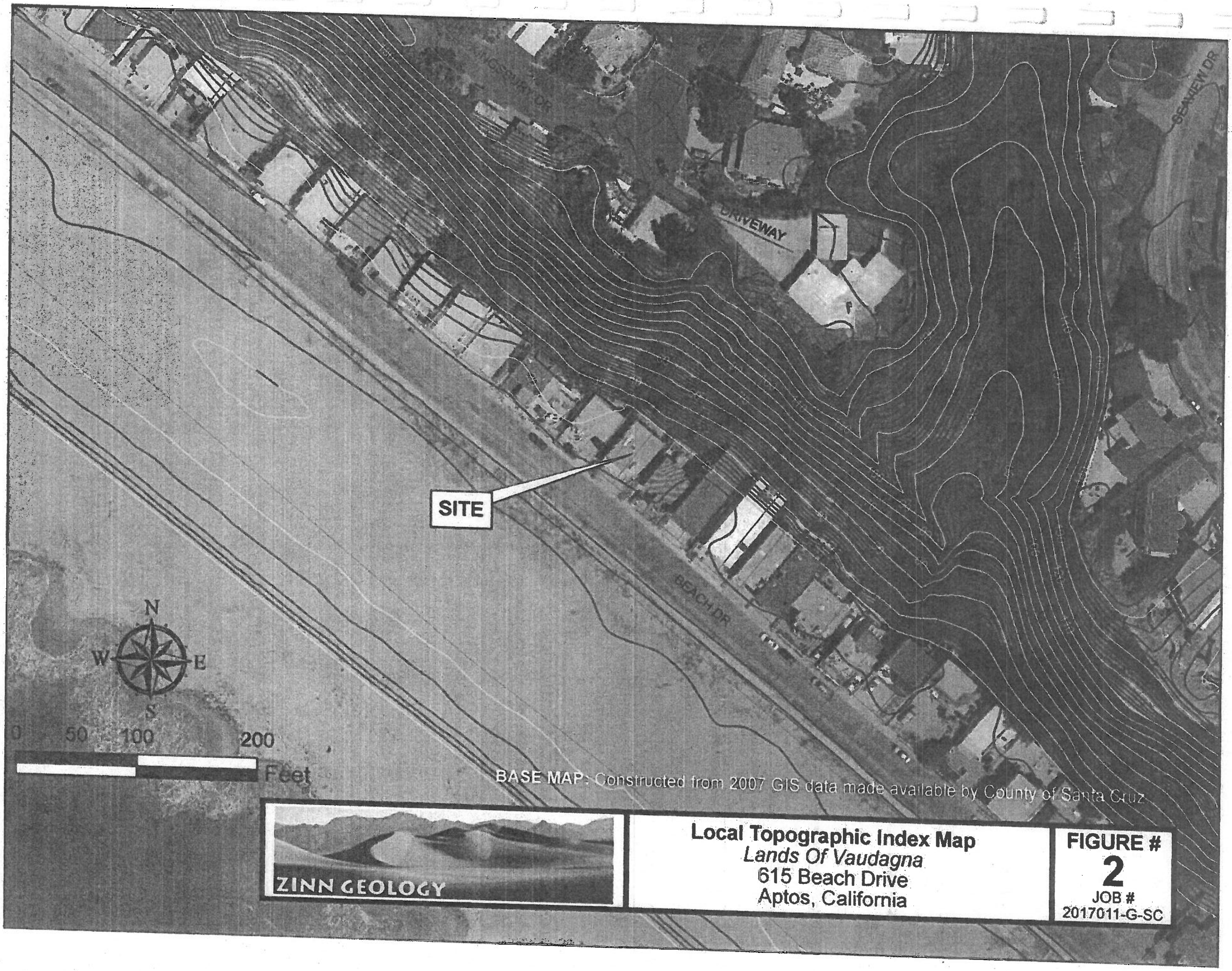


ZINN GEOLOGY

Topographic Index Map
Lands Of Vaudagna
379 Beach Drive
Aptos, California

FIGURE #
1
JOB #
2017011-G-SC

EXHIBIT G

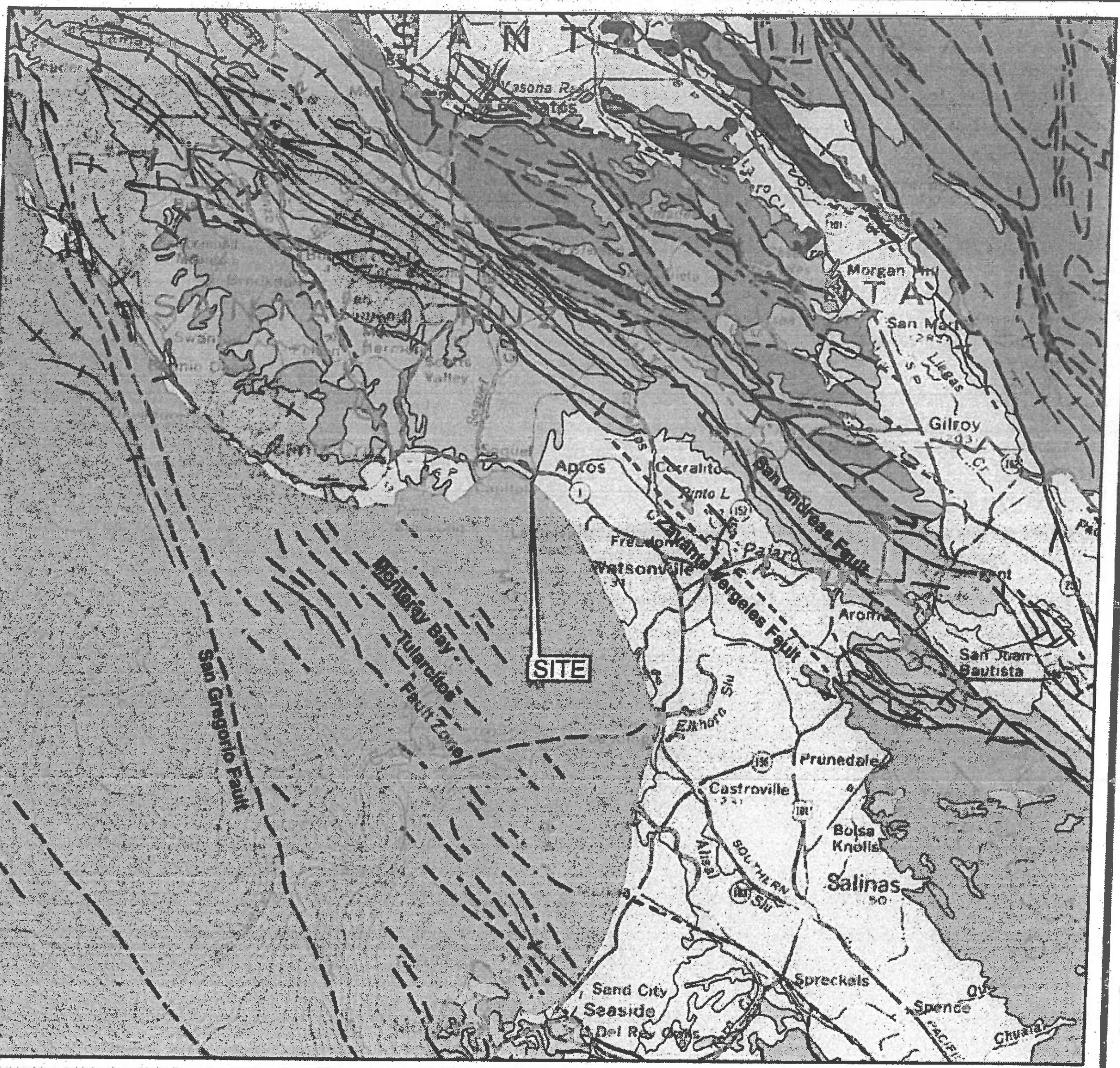


BASE MAP: Constructed from 2007 GIS data made available by County of Santa Cruz

Local Topographic Index Map
Lands Of Vaudagna
615 Beach Drive
Aptos, California

FIGURE #
2
JOB #
2017011-G-SC

EXHIBIT G



Reference: Jennings, C.W., 1977, Geologic Map of California: California Department of Conservation, Division of Mines and Geology, scale 1:750,000.
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EXPLANATION

Geologic Units

- ▭ Quaternary Deposits
- ▭ Quaternary Volcanics
- ▭ Tertiary Sedimentary Rocks
- ▭ Tertiary Volcanic Rocks
- ▭ Pre-Tertiary Sedimentary Rocks
- ▨ Pre-Tertiary Volcanic Rocks
- ▨ Grantic Intrusive Rocks
- ▨ Franciscan Complex
- ■ Ultramafic Rocks
- ▭ Pre-Tertiary Metamorphic Rock
- ▨ Pre-Cambrian Metamorphic and Igneous Rocks

Symbols

- ╲ contact
- ╲ fault, certain
- ╲ fault,approx. located
- ╲ fault, concealed or inferred
- ✕ anticline
- ✕ monocline
- ✕ syncline

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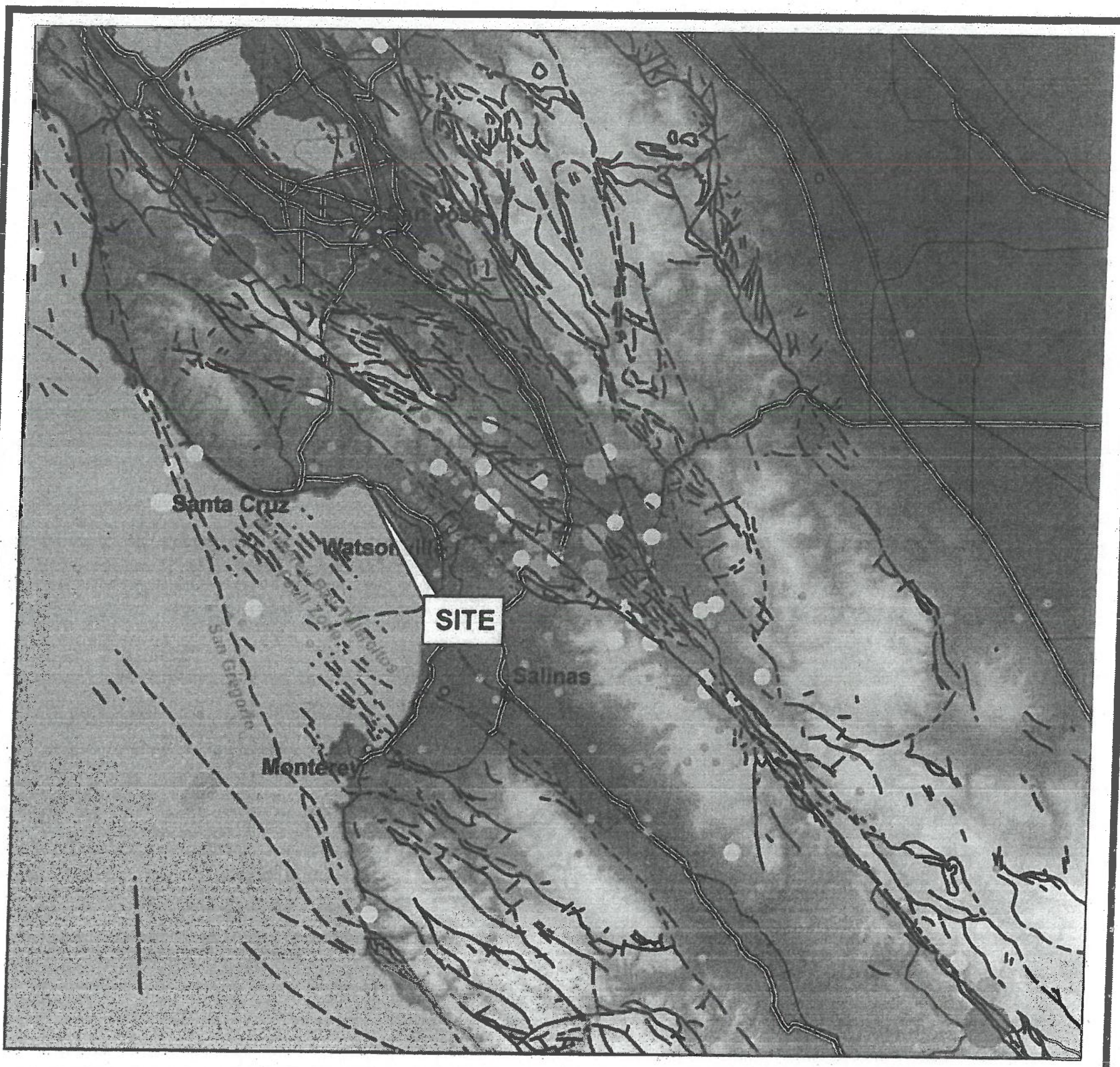
ZINN GEOLOGY

Regional Geologic Map
Lands Of Vaudagna
379 Beach Drive
Aptos, California

FIGURE #
3
JOB #
2017011-G-SC

EXHIBIT G



Seismicity Information: Magnitude 4 and greater earthquakes, compiled from various sources, 1769 to 2000; available at www.consrv.cagov/CGS/rghm/quakes/cgs2000_fnl.txt
Fault Information: Jennings, C.W., 1977, Geologic map of California: California Department of Conservation, Division of Mines and Geology, scale 1:750,000

EXPLANATION

Symbols

—— fault, certain
— — fault,approx. located
- - - - fault, concealed or inferred

Earthquake Magnitude

● 4.0 to 4.99
● 5.0 to 5.99
● 6.0 to 6.99
● 7.0 +

N
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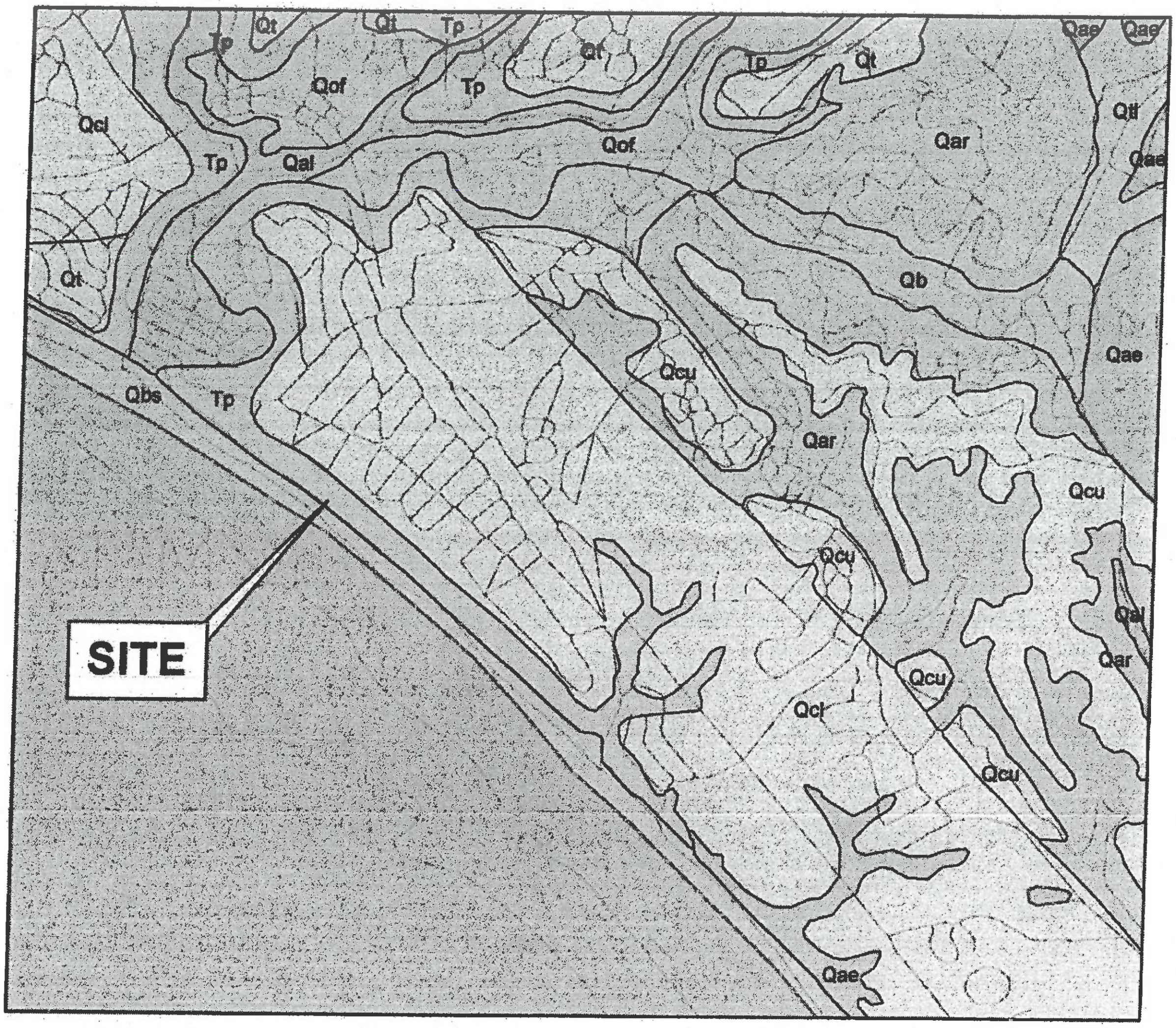
Miles

ZINN GEOLOGY

Regional Seismicity Map
Lands Of Vaudagna
379 Beach Drive
Aptos, California

FIGURE #
4
JOB #
2017011-G-SC

EXHIBIT G



BASE MAP: Brabb, E.E., 1997, Geologic map of Santa Cruz County, California: a digital database: U.S. Geological Sruvey, Open-File Report 97-489, scale 1:62,500.

Explanation

UNITS

- ▨ Qbs - Beach Sand
- ▨ Water
- ▨ Qal - Alluvium
- ▨ Qof - Older flood-plain deposits
- ▨ Qcl - First emergent coastal terrace deposits
- ▨ Qcu - Undifferentiated coastal terrace deposits
- ▨ Qt - Undifferentiated terrace deposits
- ▨ Qar - Undivided Aromas Sand
- ▨ Tp - Purisima Formation

SYMBOLS

- ——— contact, certain
- – – – water boundary

N
W E
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SCALE 1:24,000

1000 0 1000 2000 feet

ZINN GEOLOGY

Local Geologic Index Map
Lands Of Vaudagna
379 Beach Drive
Aptos, California

FIGURE #
5
JOB #
2017011-G-SC

EXHIBIT G

APPENDIX B

SCALE OF ACCEPTABLE RISKS FROM GEOLOGIC HAZARDS

SCALE OF ACCEPTABLE RISKS FROM SEISMIC GEOLOGIC HAZARDS		
Risk Level	Structure Types	Extra Project Cost Probably Required to Reduce Risk to an Acceptable Level
Extremely low[1]	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	No set percentage (whatever is required for maximum attainable safety).
Slightly higher than under "Extremely low" level.[1]	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	5 to 25 percent of project cost.[2]
Lowest possible risk to occupants of the structure.[3]	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	5 to 15 percent of project cost.[4]
An "ordinary" level of risk to occupants of the structure.[3,5]	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1 to 2 percent of project cost, in most cases (2 to 10 percent of project cost in a minority of cases).[4]

[1] Failure of a single structure may affect substantial populations.

[2] These additional percentages are based on the assumptions that the base cost is the total cost of the building or other facility when ready for occupancy. In addition, it is assumed that the structure would have been designed and built in accordance with current California practice. Moreover, the estimated additional cost presumes that structures in this acceptable risk category are to embody sufficient safety to remain functional following an earthquake.

[3] Failure of a single structure would affect primarily only the occupants.

[4] These additional percentages are based on the assumption that the base cost is the total cost of the building or facility when ready for occupancy. In addition, it is assumed that the structures would have been designed and built in accordance with current California practice. Moreover the estimated additional cost presumes that structures in this acceptable-risk category are to be sufficiently safe to give reasonable assurance of preventing injury or loss of life during and following an earthquake, but otherwise not necessarily to remain functional.

[5] "Ordinary risk": Resist minor earthquakes without damage: resist moderate earthquakes without structural damage, but with some non-structural damage; resist major earthquakes of the intensity or severity of the strongest experienced in California, without collapse, but with some structural damage as well as non-structural damage. In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. (Structural Engineers Association of California)

Source: *Meeting the Earthquake,* Joint Committee on Seismic Safety of the California Legislature, Jan. 1974, p.9.

SCALE OF ACCEPTABLE RISKS FROM NON-SEISMIC GEOLOGIC HAZARDS[6]

Risk Level	Structure Type	Risk Characteristics
Extremely low risk	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	1. Failure affects substantial populations, risk nearly equals nearly zero.
Very low risk	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	1. Failure affects substantial populations. Risk slightly higher than 1 above.
Low risk	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	1. Failure of a single structure would affect primarily only the occupants.
"Ordinary" risk	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1. Failure only affects owners /occupants of a structure rather than a substantial population. 2. No significant potential for loss of life or serious physical injury. 3. Risk level is similar or comparable to other ordinary risks (including seismic risks) to citizens of coastal California. 4. No collapse of structures; structural damage limited to repairable damage in most cases. This degree of damage is unlikely as a result of storms with a repeat time of 50 years or less.
Moderate risk	Fences, driveways, non-habitable structures, detached retaining walls, sanitary landfills, recreation areas and open space.	1. Structure is not occupied or occupied infrequently. 2. Low probability of physical injury. 3. Moderate probability of collapse.

[6] Non-seismic geologic hazards include flooding, landslides, erosion, wave runup and sinkhole collapse

COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT
701 OCEAN STREET, 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123
KATHLEEN MOLLOY PREVISICH, PLANNING DIRECTOR

26 March 2018

Matson Britton Architects
728 N. Branciforte Ave.
Santa Cruz, CA 95062

Subject: Review of the Geotechnical Investigation dated 30 November 2017 by Pacific Crest Engineering, Inc. – Project No. 1738-SZ70-B44

Review of the Coastal Geologic Investigation dated 11 February 2018 by Zinn Geology – Project No. 2017011-G-SC

Project Site: 379 Beach Drive
APN 043-095-14
Application No. REV181023

Dear Applicant:

The purpose of this letter is to inform you that the Planning Department has *not accepted* the subject reports for the following reasons:

1. The 379 Beach Drive project site is situated at the base of the coastal bluff. A new residence was recently constructed upon the blufftop above the project site at 340 Kingsbury Drive. To minimize the effects of bluff recession upon the blufftop development, a Tecco Mesh slope stabilization system was installed below the blufftop and a buried soil pin type retaining wall system was installed inboard of sections of the blufftop. A portion of the Tecco Mesh slope stabilization system failed in March 2016 and has been repaired. The recent re-development of the property at the top of the bluff has caused erosion and debris flow hazards on the subject parcels below. The upper bluff is no longer a natural slope, therefore the stability of the entire slope must be quantitatively evaluated by the geotechnical engineer given the current conditions.

   At a minimum, the stability of the bluff face shall be evaluated by: pseudostatic analysis of a wet winter/design seismic event with a minimum factor of safety of 1.1 or greater; and surficial stability analysis of the bluff face using an infinite slope model with saturation of the colluvium mantling the slope as well as seepage parallel to the slope surface with a minimum factor of safety of 1.5 or greater. The Tecco Mesh slope stabilization system should not be utilized in the slope stability models for the long term assessment of the stability of bluff face surficial soils.

EXHIBIT

2. The project geologist has qualitatively determined anticipated modes of slope failure above 379 Beach Drive as well as estimated debris impact velocities. After the quantitative slope analyses are complete, the project geologist and geotechnical engineer should work together to determine a project design slide debris mass volume.

   Once the design slide debris mass volume has been determined based on quantitative analyses, the project geologist should revise the project Geologic Cross Section using the current house design (Sheet P3 – MBA 10/11/17) and illustrating the post event configuration of the slide mass debris.

3. The 2016 California Building Code (CBC) Section 1808.7.2 requires a setback from descending slopes to the face of the structure that is equal to at least the smaller of half the height of the slope and 15 feet. The current proposal includes a second floor deck which is setback approximately 4 feet from the face of the slope (Sheet P6). As proposed, the project cannot be approved, as a reduction to this setback cannot be granted given the known and ongoing slope instability above the proposed home. In order to consider a reduced setback, the geotechnical engineer must demonstrate that the entire volume of the projected debris flow will solely pass under the home, without being scattered by the flow impacting portions of the home (i.e. second floor deck).

4. The project geotechnical engineer shall determine the design slide debris impact force striking the columns supporting the residence as well as the elevations of the anticipated impact zone(s).

5. An Alternatives Analysis must be completed that demonstrates the proposed design is as safe as other potential designs such as a bunker style residence set into the blufftoe (SCCC 16.10.070(H)(3)(c)).

Note: The project geology report does not address the issue of sea level rise either historic or related to anthropomorphic global warming. Given that our the County determines Base Flood Elevation based upon maps published through the National Flood Insurance Program, the County will not at this time ask for elaboration concerning these factors and the design of the new home. The County reserves the right to request additional information concern sea level rise should questions rise during the environmental review process.

Additional comments may be forthcoming pending review of the information requested above.

Please note that this determination may be appealed within 14 calendar days of the date of service. Additional information regarding the appeals process may be found online at: http://www.sccoplanning.com/html/devrev/plnappeal_bldg.htm

EXHIBIT H

Please contact Rick Parks at (831) 454-3168/email: Rick.Parks@santacruzcounty.us or Joe Hanna at (831) 454-3175/Joseph.Hanna@santacruzcounty.us if we can be of any further assistance.

Respectfully,

Rick Parks, GE 2603
Civil Engineer – Environmental Planning
County of Santa Cruz Planning Department

Joseph Hanna, CEG 1313
County Geologist– Environmental Planning
County of Santa Cruz Planning Department

Cc: Environmental Planning, Attn: Jessica deGrassi
Zinn Geology, Attn: Eric Zinn, CEG
Pacific Crest Engineering, Attn: Soma Goresky, GE

EXHIBIT H

Pacific Crest
ENGINEERING INC

GEOTECHNICAL | ENVIRONMENTAL | CHEMICAL | MATERIAL TESTING | SPECIAL INSPECTIONS

August 16, 2018

Project No. 1738-SZ70-B44

Jim and Sue Vaudagna
19501 Scotland Drive
Saratoga, CA 95070

Subject: **Response to Review of Geotechnical Investigation and Supplemental Analysis**
379 Beach Drive
APN 043-095-14
Aptos, California

References:

1) **Geotechnical Investigation, 379 Beach Drive, Aptos, California**, Project No. 1738-SZ70-B44, dated November 30, 2017, prepared by Pacific Crest Engineering, Inc.

2) **Review of the Geotechnical Investigation,** dated March 26, 2018, prepared by Planning Department, County of Santa Cruz, dated March 2018.

3) **Coastal Geologic Investigation, Lands of Vaudagna**, Job #2017011-G-SC, dated February 11, 2018 and prepared by Zinn Geology.

4) **Draft Site Retaining Wall Section,** prepared by RI Engineering

5) **Response to County of Santa Cruz Comments** prepared by Zinn Geology and dated August 16, 2018

Dear Mr. and Mrs. Vaudagna,

As requested, we have reviewed the County of Santa Cruz's review of our geotechnical investigation for 379 Beach Drive (Reference 2 listed above), met with representatives of the County of Santa Cruz Planning Department to discuss the site and discussed the project with your design team . Our response to Reference 2 and supplemental analyses are presented below. For ease of reference the numbering scheme in Reference 2 is repeated below.

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Comment 1, Paragraph 1 - Tecco System

To address Comment #1, we would first like to preface our response by addressing the Planning Department's wording for that comment. First, Pacific Crest Engineering provided geotechnical engineering assistance in the design and construction of a Geobrugg Tecco stabilization system for the property located immediately upslope of the Vaudagna property (the Meyerhoff property). It was the primary objective of the upslope property owner (Meyerhoff) to increase the stability of the coastal bluff contained within confines of their property, to lower the likelihood and magnitude of the coastal bluff failing and striking the unprotected residences below. The Tecco stabilization measure was NOT constructed to protect the Meyerhoff residence, because it was located seaward of the 100-year bluff retreat line, which did not take any bluff protection into account. Nonetheless, we do recognize that while the Tecco system did in part serve to "minimize the effects of bluff recession" that was not the primary objective.

Second, we strongly disagree that the "recent re-development of the Meyerhoff property at the top of the bluff has caused erosion and debris flow hazards on the subject parcels below". In fact, we consider the opposite to be true; now that the Meyerhoff residential project has been completed and the engineered drainage improvements have been fully implemented, the risk of erosion and debris flows hazards emanating from the Meyerhoff property and impacting the properties below has been significantly reduced.

Third, the statement that a "portion of the Tecco stabilization system failed in March 2016" requires additional clarification when describing events during the strong storms that occurred in early March 2016 while the Meyerhoff residence was under construction. A debris flow occurred on a portion of the bluff adjacent to the Tecco system when concentrated construction storm water was improperly directed to the southeast side of the property and allowed to cascade onto the unprotected bluff face. The lateral margins of the erosion and subsequent debris flow undermined a portion of the Tecco system perimeter, but the system stayed in place and continued to retain soil underneath, which means that the system did not "fail". The contractor subsequently implemented a repair plan and the Tecco system was re-secured to the bluff after hand grading and erosion control measures were employed to improve sheet flow conditions.

It is important to note that erosion was a component of the soil that moved downslope during the storm event. The rill and gully that developed on the scar and within the debris flow deposit clearly demonstrated that concentrated water continued to flow and scour the sandy soil underlying the mesh. The primary function of the Tecco system is to migigate debris flows and erosion driven by direct rainfall. As clearly revealed by the event of March 5, 2016, these systems are not designed to handle concentrated storm water runoff.

Therefore, given our understanding and experience with the events of March 2016 and the resulting measures to counteract the contractor's actions that led to the debris flow that damaged (but did not fail) the Tecco system, we are of the opinion that the approximately 4300 square feet of Tecco stabilization system continues to perform as designed and has significantly reduced the risk of debris flow and erosion hazards to impact the downlope properties. As this is an engineered system that continues to perform, we saw no reason why it should not be modeled as such when assessing the slope stability hazard on the Vaudagna property. As described below we have therefore developed a

EXHIBIT I

quantitative slope stability model which appropriately includes the engineered Tecco system on the adjacent, upslope property but assumes up to 8 feet of soil loss around its perimeter. To omit the upper bluff improvements completely would, in our opinion, be overly conservative and does not adequately model the existing bluff configuration.

Comment 1, Paragraph 2 - Slope Stability Analysis

As requested we have performed a slope stability analysis of the bluff face above the subject site with the purpose of substantiating the design volume of a debris flow event that potentially could impact the proposed improvements. We note that the design of the proposed improvements assume that portions of the bluff will fail in heavy rainfall and debris flows will impact the site. As designed the bottom floor of the house is not for habitation and will have "break away" walls to allow impact loading and debris originating from the bluff to flow through the lower level without damaging the habitable space above.

Qualitative Stability Assessment

The subject bluff consists of marine terrace deposits underlain by Purisima formation sandstone. Both materials are mantled by about a 3 to 5 foot layer of colluvial soil. Based on the steep inclinations that both the Purisima and Marine Terrace deposits naturally stand at, as well as our laboratory test results these materials are moderately cemented and exhibit significant cohesive strength. Geomorphology in the immediate area suggest that typical failures on the bluff face are relatively thin in depth, are limited to relatively small areal extents, and are on the order of 25 feet in length and 10 feet in width. Generally debris flow failures are comprised of the surficial colluvial soil. In some cases the surficial "rind" of bedrock loses some of it's cementation/cohesion due to exposure and weathering and is included as part of the failure mass. Additionally, near vertical jointing of isolated, relatively thin layers of sandstone and marine terrace materials can be prone to failure. We did not observe any slope features indicative of deep seated, rotational failures in the area and infer that this mode of failure is not applicable to the site.

In our opinion, due of the mode of failure (debris flows incorporating some unknown depth of surficial soil with degrading cohesional strength) and the difficulty in accurately estimating pore pressures in the surface of the slope during failure conditions (e.g. heavy rainfall), a qualitative assessment of the slope based on it's past activity and geomorphology is likely the most accurate means of estimating debris flow volumes and sources. Since the bluff is actively retreating the nature and size of failures are evident. Thus failure modes and size are more accurately assessed by a qualitative observation rather than 2D finite element computer modeling that requires numerous assumptions.

Quantitative Stability Assessment

As requested by the County of Santa Cruz we are including a quantitative slope stability analysis. To model the quantitative stability of the slope we utilized the geologic cross section provided by Zinn Geology (Reference 3) with the currently proposed improvements superimposed on it. RI Enginering has provided a draft cross section showing terraced retaining walls between the house the base of the bluff, allowing more room for debris to pass through the proposed residence. The updated vcivil engineering configuration has been incorporated into both the geologic cross sections as well as our model.

The subsurface profile was divided into 4 units: colluvium, Marine Terrace deposits, Purisima sandstone (Tp), and Beach Sand. The proposed pier supported retaining wall was modeled by inserting a near vertical 1 foot wide layer with a strength roughly equivalent to concrete.

Soil strengths of the different layers in our model were determined based on laboratory testing from soils retrieved from on site borings and supplemented by borings and lab data for the site upslope of the property and two additional sites at the top of the bluff in the immediate area.

Shear strength parameters for the various subsurface layers that were used in our analysis are listed in the table below.

Shear Strength Parameters for Stability Analysis

Subsurface Layer	Friction Angle (degrees)	Cohesion (psf)
Colluvium	26	200
Marine Terrace	32	390
Purisima Sandstone	45	500

For our seismic slope stability analysis we made the conservative assumption that short term, seismic strengths were equal to the static strengths.

For the majority of the year the ground water elevation at the site is roughly equal to the sea level. Ground water was not encountered in any of our borings. Because the colluvial soil is much more permeable than both the underlying marine terrace deposits and Purisima formation we infer that during heavy winter rainfall water becomes perched in the colluvium and rapidly moves downslope. In our opinion the likelihood that ground water will saturate the entire depth of colluvium along the entire height of the slope is very low. Due to the steepness of the slope and the relatively high permeability of the surficial soil, ground water is likely to move rapidly through the colluvium in the upper, steeper bluff face, and then possibly saturating portions of the slope below where the slope angle decreases. However, at the reviewers request our quantitative analysis assumes that ground water saturates the entire depth of colluvium and is at the ground surface for the entire length of the slope. We have labelled this the “extreme” ground water condition.

In our analyses the existing Tecco stabilization system was modelled by using a support system consisting of 18 foot long grouted tiedbacks, spaced 8 foot on center on the upper slope. Please refer to Comment #1, Tecco system, above for a discussion on the performance of the existing Tecco system.

Stability Analysis Results - Slope stability analyses were performed using SLIDE v. 7.031, a computer program for two-dimensional limit equilibrium slope stability analysis developed by Rocscience Inc. Random circular and non-circular searches were performed along the entire slope.

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The following discussion summarizes our quantitative analyses of the slope:

a. *Estimate of Debris Flow Volume with Extreme Weather High Water Conditions:* For our initial analysis we assumed that the upper portion of the slope is reinforced by the Tecco stabilization system with the caveat that the perimeter 8 feet of the Tecco could be undermined in any one event and incorporated into the unstable material. To model this the downslope perimeter "soil nail" of the Tecco system was omitted. The stabilizing effect of the plates that fix the anchors to the slope, the steel wire mesh and Tecco system were not included in our model. We initially modelled the slope assuming a worst case high water condition with a water table saturating the lower half of the colluvium. With this assumption and our strength parameters we could not produce failure conditions (e.g. failure is F.S. < 1.0).

   In order to produce failure conditions it was necessary to model the ground water at the ground surface for the entire bluff and to reduce the cohesion of the colluvial soils. For this extreme condition we obtained a hypothetical failure surface that is about 52 feet long and 7 feet in maximum depth (see Figure 1, attached). This model was used by the project geologist to calculate the worst case, single event volume of material that could potentially impact the house below. In our opinion it is highly unlikely that the entire slope could be saturated to this degree at one time. It is more likely that portions of the slope, likely the ones that lie at lower slope angles, are saturated. This finding substantiates the opinion that typical slope failures are much smaller in areal extent than what is produced by this model. Nonetheless, as requested by the reviewer, we provided this size of this failure generated by the stability analysis to the project geologist for design.

b. *Seismic Analysis:* For this case our static analysis assumed typical wet weather conditions (rainfall would saturate about one-half the depth of colluvium). Additionally we made the conservative assumption that seismic shear strengths were equal to static shear strengths. Selection of the seismic coefficient was based on Bray and Travasarou (2009)[1], which considers the allowable seismic displacement and the fundamental period of the sliding mass. Parameters used in this method are outlined below:

   Fundamental Period of Landslide Mass (Ts = 4H/Vs) = 0.02 secs
   Degraded Period of Landslide Mass (1.5Ts) = 0.03 secs
   Magnitude Earthquake (Mw) = 8.0
   Distance Seismic Source to Site = 12.5 km
   Threshold Displacement = 5 cm
   Spectral Acceleration at Degraded Period (Sa) = 0.27 secs
   Seismic Coefficient (k) = 0.16g

   The spectral acceleration at the degraded period (Sa) is based on Next Generation Attenuation relationships (NGA 2008). Magnitude and distance of the predominant fault for the site was based on the USGS Unified Hazard Tool Deaggregation website https://earthquake.usgs.gov/hazards/interactive/.

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[1] Bray, Jonathan D., Travasarou, T., "Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation" Journal of Geotechnical and Geoenvironmental Engineering, ASCE September 2009, p.1336

EXHIBIT I

Using strength values mentioned above and a seismic coefficient of 0.16g we obtained a factor of safety of 1.0 for a failure surface that is slightly smaller than the static extreme weather conditions (see Figure 2, attached).

Conclusion and Preliminary Recommendations

The above analysis yields a location and volume for potential debris flows and was used in designing the required runout area behind and under the habitable portion of the proposed development. To develop a cross sectional area of potential failure geometries we assumed a width of 30 feet to derive a debris flow volume. Additionally, the volume of material derived from the undermining of the perimeter 8 feet of the area covered by the Tecco system on the upslope property was included in the total debris flow volume.

Zinn Geology used this resulting volume and on Plate 1 of their report (Reference 5) depicts three scenarios of the debris flowing through and coming to rest within the lower level of the house. Scenarios 2 and 3 incorporate measures for reducing the flow volume that reaches the house. Scenario 2 incorporates impact walls at the top of each of the two tiered retaining walls and Scenario 3 incorporates a debris flow fence upslope of the tiered retaining walls.

In our opinion both Scenario 2 and 3 provide adequate means for reducing the risk that debris will impact the habitable portion of the house. Preliminary design for impact walls should be based on an impact loading of 1900 psf. Preliminary design of debris flow fences should be based on the parameters presented in Plate 2, Reference 5. Design of all impact structures should include "wing walls" that confine the debris to the site and prevent it from being deflected onto the adjacent properties. We request the opportunity to review proposed designs for debris fences or impact walls and to provide additional geotechnical design recommendations as needed.

Comment 2

This comment has been addressed by the project geologist (see Reference 5).

Comment 3

Based on our slope stability analysis and recommended debris structures in Scenarios 2 and 3 as addressed above, the hazard associated with proposed building location and adjacent slope has been addressed.

Comment 4

Assuming that either the impact walls or the debris fence will be constructed as shown on Plate 2, Reference 5, impact loads of 1200 psf should be applied to the columns supporting the house.. The depth of the impact load may be assumed based on the column location and the height of the green and red prisms shown on Plate 2, whichever results in the greater depth.

Comment 5

This comment is being addressed by the project geologist, Zinn Geology.

We appreciate the opportunity to be of service. Please contact us if you have any questions.

Sincerely,

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c.c. Matson Britton Architects (2)
Zinn Geology
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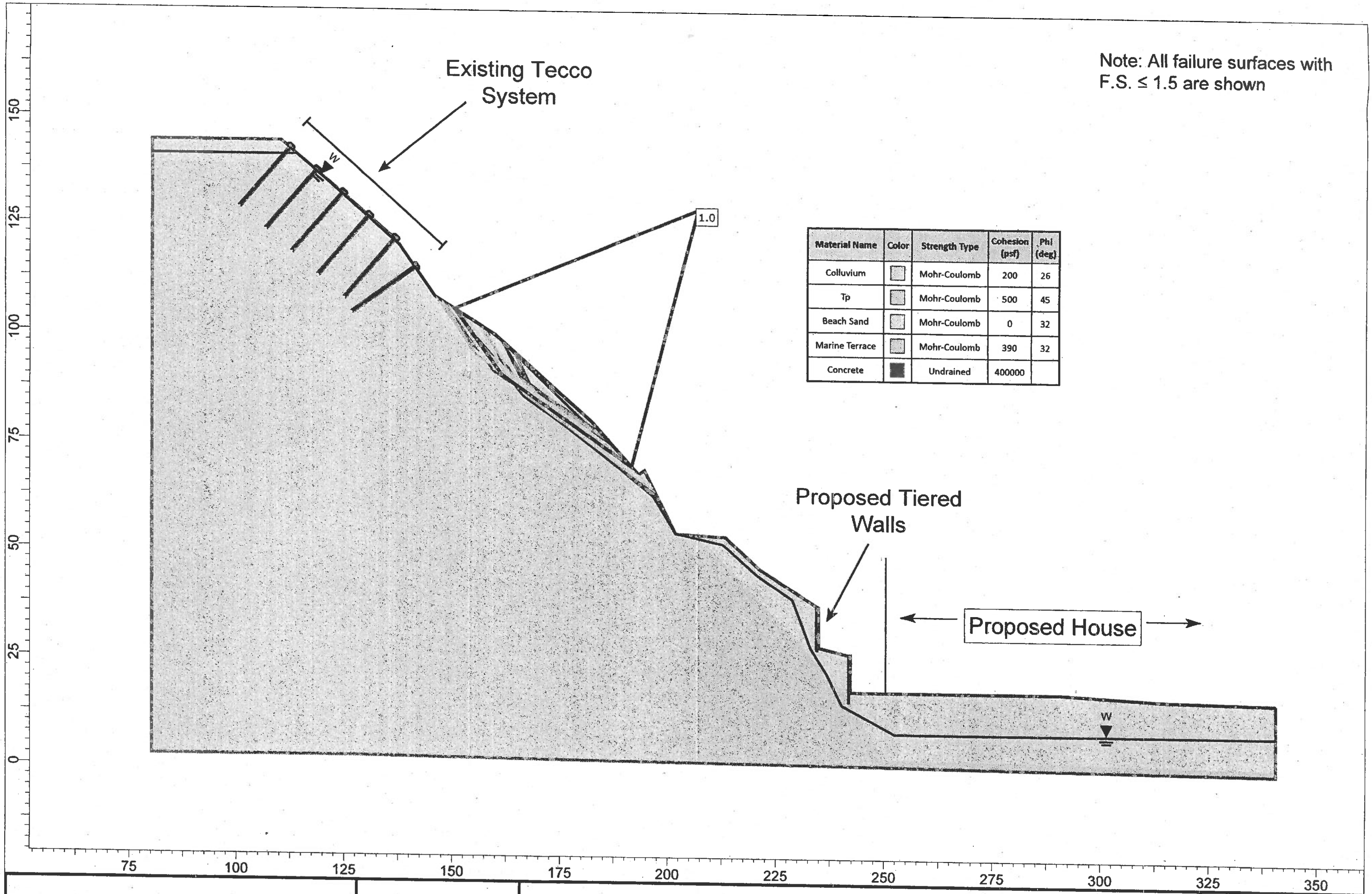
Attachments:
- Figure 1 – Static Analysis
- Figure 2 – Seismic Analysis

EXHIBIT I



Note: All failure surfaces with F.S. ≤ 1.5 are shown

Material Name	Color	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium	▨	Mohr-Coulomb	200	26
Tp	▨	Mohr-Coulomb	500	45
Beach Sand	▨	Mohr-Coulomb	0	32
Marine Terrace	▨	Mohr-Coulomb	390	32
Concrete	■	Undrained	400000	

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Date: 8/16/2018

File Name:
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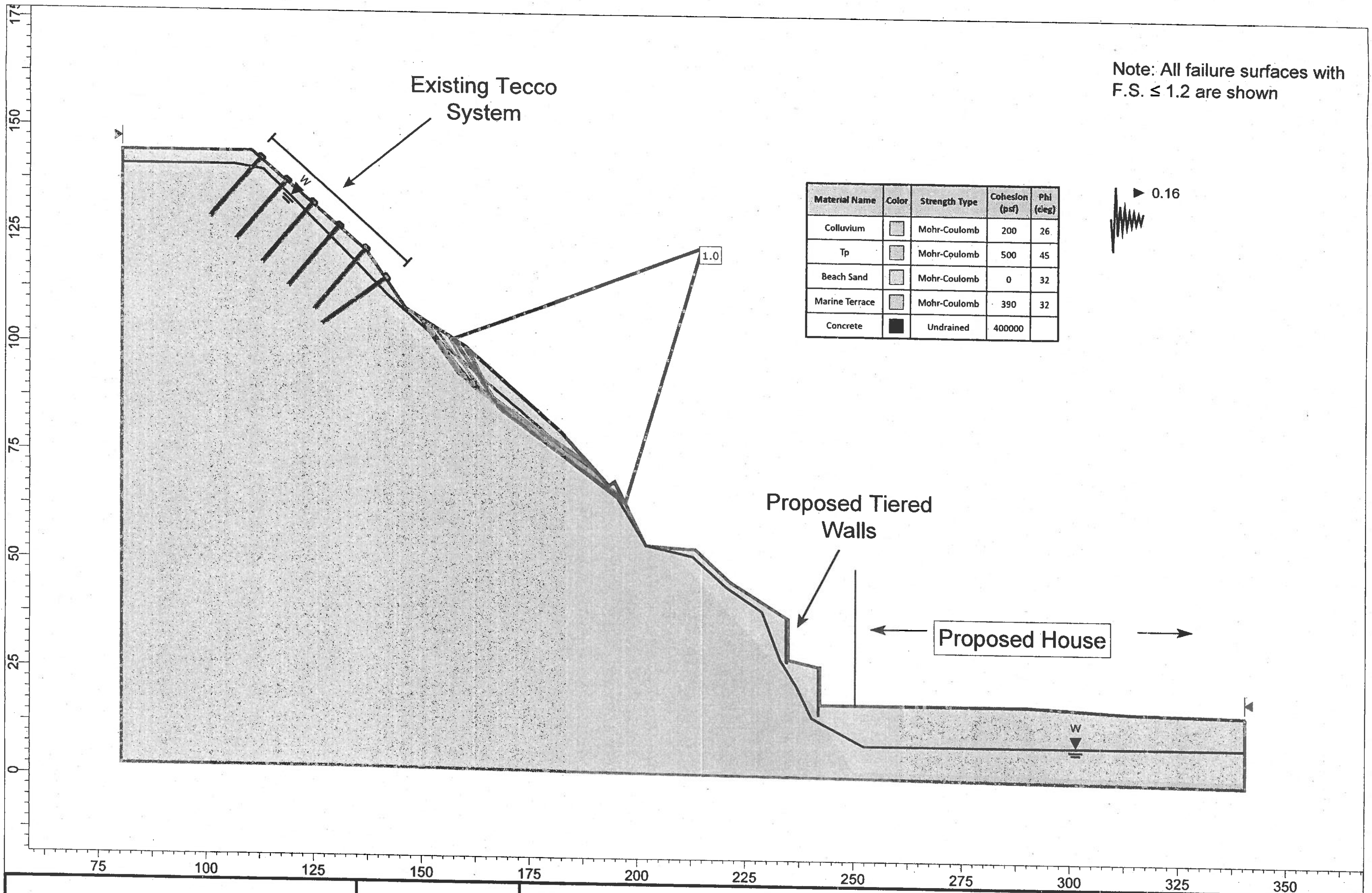
Static Analysis - Extreme High Water Conditions
379 Beach Drive
Aptos, California

Figure No: 1

Project No. 1738

EXHIBIT I



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Pacific Crest
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Date: 8/16/2018

File Name:
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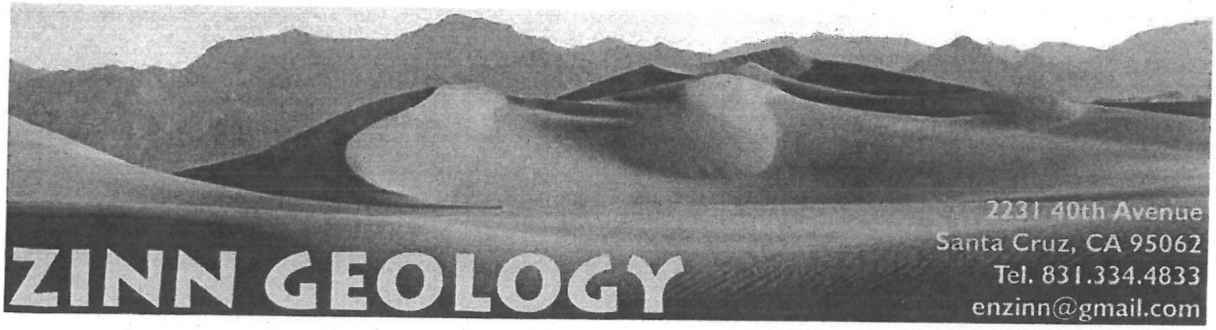
Seismic Analysis
379 Beach Drive
Aptos, California

Figure No: 2

Project No. 1738

EXHIBIT I



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16 August 2018 Job #2017011-G-SC

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Re: Response to County of Santa Cruz comments
Coastal geologic investigation
379 Beach Drive
Aptos, California
County of Santa Cruz APN 043-095-14

Dear Mr. And Ms. Vaudagna:

This letter summarizes our response to comments and request for supplemental data and analysis given in a report review letter given by the County of Santa Cruz Planning Department dated 26 March 2018. Our responses are given in the same sequence as the comments in the County letter and follow the County letter enumeration.

> *County Comment 1 . The 379 Beach Drive project site is situated at the base of the coastal bluff. A new residence was recently constructed upon the blufftop above the project site at 340 Kingsbury Drive. To minimize the effects of bluff recession upon the blufftop development, a Tecco Mesh slope stabilization system was installed below the blufftop and a buried soil pin type retaining wall system was installed inboard of sections of the blufftop. A portion of the Tecco Mesh slope stabilization system failed in March 2016 and has been repaired. The recent re-development of the property at the top of the bluff has caused erosion and debris flow hazards on the subject parcels below. The upper bluff is no longer a natural slope, therefore the stability of the entire slope must be quantitatively evaluated by the geotechnical engineer given the current conditions.*
>
> *At a minimum, the stability of the bluff face shall be evaluated by: pseudostatic analysis of a wet winter/design seismic event with a minimum factor of safety of 1.1 or greater; and surficial stability analysis of the bluff face using an infinite slope model with saturation of the colluvium mantling the slope as well as seepage parallel to the slope surface with a minimum factor of safety of 1.5 or greater. The Teeco Mesh slope stabilization system should not be utilized in the slope stability models for the long term assessment of the stability of bluff face surficial soils.*

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Zinn Geology Response To County Comment #1 - We were the project geologist for referenced project at 340 Kingsbury Drive. The Tecco Mesh did not actually fail. Instead, the edges of the mesh were compromised where concentrated construction stormwater water was focused and small debris flows followed by erosion were triggered along the edges. Our observations at the time of damage to the edges of the mesh were as follows:.

1. Construction storm water has been ponding in the driveway and flowing across the site toward the top of the coastal bluff.
2. The construction storm water is flowing over the top of the bluff in an unnatural concentrated fashion.
3. The construction storm water was the likely trigger event for the Saturday night (5 March) debris flow that undermined the erosion control fabric on the ridge.
4. Concentrated construction storm water continued to flow along the edge of the debris flow and the edge of the Tecco system after the debris flow was triggered, etching a very steep and deep erosional gully (six plus feet deep, three to four feet wide) into the slope below the Tecco fabric system.
5. The concentrated construction storm water that formed the gully under the edge of the Tecco system also etched another gully into the debris flow deposits below the property down at the bottom of the slope.
6. Stepping back and looking at the bigger picture, every time we have had gullying or debris flows occur on the steep slopes that ring the property, the trigger event has been an unnaturally high volume and discharge of construction storm water and storm water from the blocked County storm drain. The soil on the slopes are unstable under existing natural conditions, which is why pin piles and the Tecco fabric system have been installed. But neither of those systems are designed to handle large concentrated volumes of storm water discharge.

The problem that created the shallow landsliding and erosion along the edges of the mesh stemmed from poor periodic control of construction storm water discharge, resulting in unnaturally high volumes and discharge of water on the steep slopes that flank the Meyerhoff - Kingsbury site. The condition of poorly controlled construction storm water actually occurred both winters while the Meyerhoff project was under construction, with shallow landslides shedding from the upper coastal bluff and the upper arroyo flank occurring where the concentrated storm water was improperly disposed. This condition is no longer present on the Meyerhoff site, since the engineered drainage improvements have been fully implemented. In our opinion, the risk of erosion and debris flows hazards emanating from the Meyerhoff property and impacting the properties below has been significantly reduced due to the emplacement of the Tecco fabric system and the engineered drainage improvements.

We have also taken the liberty of attaching a letter prepared by John Kasunich of Haro, Kasunich and Associates on 28 October 2016 explaining the geotechnical engineering parameters for the damage to the edges of the mesh (see Appendix A).

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While the mesh did **not** fail, we do recognize, however, that the edges of the mesh have failed in the past and may fail in the future, Therefore, we have assumed that soil under the first eight feet of the mat around its periphery, excepting the top edge, will fail in the future. This assumption has been incorporated into our assumption for debris flow source areas that can generate debris flows that could strike the proposed house (see attached Plate 1).

> *County Comment 2. The project geologist has qualitatively determined anticipated modes of slope failure above 379 Beach Drive as well as estimated debris impact velocities. After the quantitative slope analyses are complete, the project geologist and geotechnical engineer should work together to determine a project design slide debris mass volume. Once the design slide debris mass volume has been determined based on quantitative analyses, the project geologist should revise the project Geologic Cross Section using the current house design (Sheet P3 -MBA 10/11 /17) and illustrating the post event configuration of the slide mass debris.*

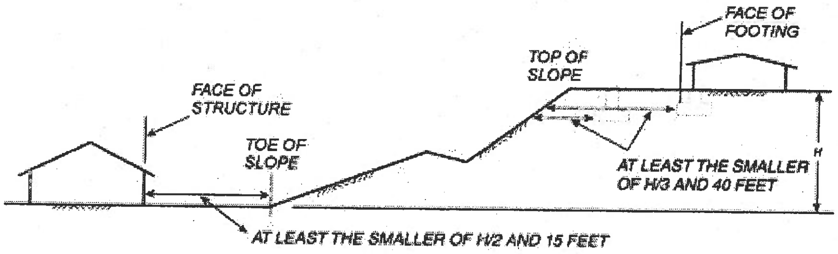
Zinn Geology Response To County Comment #2 - We have since worked with Project Geotechnical Engineer of Record, Pacific Crest Engineering, and given them geological input for their quantitative analysis. This was an iterative process that also involved some fundamental changes to the civil engineering of slope and retaining wall behind the house. The debris flow parameters issued by Pacific Crest Engineering can be seen on the attached Plate 2.

We have plotted the configuration of the quantitatively derived design debris flow on a cross section that represents the modified proposed civil engineering configuration of the back slope and retaining walls, as well as the most recent version of the house from the Matson-Britton Architect plans (see Plate 2). We have plotted three different mitigation scenarios on that base section and depicted our interpretation of the design debris flow cross section at the moment of impact and the configuration of the debris flow after it comes to rest (Plate 2).

> *County Comment 3. The 2016 California Building Code (CBC) Section 1808.7.2 requires a setback from descending slopes to the face of the structure that is equal to at least the smaller of half the height of the slope and 15 feet. The current proposal includes a second floor deck which is setback approximately 4 feet from the face of the slope (Sheet P6). As proposed, the project cannot be approved, as a reduction to this setback cannot be granted given the known and ongoing slope instability above the proposed home. In order to consider a reduced setback, the geotechnical engineer must demonstrate that the entire volume of the projected debris flow will solely pass under the home, without being scattered by the flow impacting portions _of the home (i.e. second floor deck).*

Zinn Geology Response To County Comment #3 - We have taken the liberty of citing code Section 1808.7.1 below, and not Section 1808.7.2, since we believe that the County has cited the incorrect code section. Code section 1808.7.2 actually applies to foundation setbacks from descending slope surfaces which is not applicable to this project in our opinion. In any event, Section 1808.7.1 of the 2016 California Building Code is as follows (highlighting added by us):

> *1808.7.1 Building clearance from ascending slopes*
>
> *In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided in Section 1808.7.5 and Figure 1808.7.1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.*



For SI: 1 foot = 304.8 mm.

FIGURE 1808.7.1

FOUNDATION CLEARANCES FROM SLOPES

For the sake of completeness in this response, code sections 1808.7.5 and 1803.5.10 are as follows:

> *1808.7.5 Alternate setback and clearance*
>
> *Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official shall be permitted to require a geotechnical investigation as set forth in Section 1803.5.10.*

> *1803.5.10 Alternate setback and clearance*
>
> *Where setbacks or clearances other than those required in Section 1808.7 are desired, the building official shall be permitted to require a geotechnical investigation by a registered design professional to demonstrate that the intent of Section 1808.7 would be satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.*

Our read of the appropriately plenary sequence of code citations is that section 1808.7.1 is intended for projects that do not include site specific geotechnical engineering or are in compliance with the setback parameters provided by that section.

Section 1808.7.1 does not apply to projects where site specific geotechnical engineering has been conducted, nor does it specify particulars on debris flow impact "scatter" by decks. We presume that the intent of this comment really goes to the heart of the sections 1808.7.5 and 1808.5.10 which requires the approval by the building official for the alternate setbacks. Although opinions by staff belonging to the Environmental Planning arm of the County of Santa Cruz Planning Department staff (i.e. Rick Parks and Joseph Hanna in this instance) are not actually an opinion by the County of Santa Cruz Chief Building Official (Martin Heaney in this instance), we recognize that Mr. Heaney would likely rely upon the professional opinion of Mr. Parks and Mr. Hanna in this instance with regard to whether the engineering and geology analysis for the application complies with the applicable minimum prescriptive building code elements.

Given all that, we turn to our three different debris scenario mitigation schemes depicted on Plate 2. In all three scenarios, the Pacific Crest Engineering design debris flow punches through the proposed break-away walls on the ground floor and flows through a portion of the ground floor of the proposed residence, coming to a rest within the residence. None of the interpreted design debris flows strike the proposed deck or the habitable portion of the residence for all three scenarios.

> *County Comment 4. The project geotechnical engineer shall determine the design slide debris impact force striking the columns supporting the residence as well· as the elevations of the anticipated impact zone(s).*

Zinn Geology Response To County Comment #4 - This comment does not require a response by the Project Geologist of Record.

County Comment 5. An Alternatives Analysis must be completed that demonstrates the proposed design is as safe as other potential designs such as a bunker style residence set into the blufftoe (SCCC 16.10.070(H)(3)c).

Zinn Geology Response To County Comment #5 - The section of Santa Cruz County Code cited in the review is as follows:
(c) Application for shoreline protective structures shall include thorough analysis of all reasonable alternatives to such structures, including but not limited to relocation or partial removal of the threatened structure, protection of only the upper bluff area or the area immediately adjacent to the threatened structure, beach nourishment, and vertical walls. Structural protection measures on the bluff and beach shall only be permitted where nonstructural measures, such as relocating the structure or changing the design, are infeasible from an engineering standpoint or are not economically viable.

We are not proposing to perform an "Alternatives Analysis" and have been asked by the Project Architect of Record to not work on that aspect of the project at this time.

We do have some comments regarding the recitation of that section of Santa Cruz County Code that will feed into further commentary on other comments made outside of the technical review for this application. The current proposed development scheme does not have any components that qualify as a "shoreline protective structure". The project will consist of a new house with a non-habitable ground floor with break-away walls, founded on deep piers, backed by one or more retaining walls and possibly some type of debris flow fencing on the slope above the house.

None of the aforementioned structures qualifies as a "shoreline protective structure", since none of the structures will actually protect the shoreline. We have discussed this issue further down in the letter in response to a related comment by Planning staff.

> *County Note: Note: The project geology report does not address the issue of sea level rise either historic or related to anthropomorphic global warming. Given that our the County determines Base Flood Elevation based upon maps published through the National Flood Insurance Program, the County will not at this time ask for elaboration concerning these factors and the design of the new home. The County reserves the right to request additional information concern sea level rise should questions rise during the environmental review process.*

EXHIBIT J

Zinn Geology Response To County Note - Santa Cruz County Code section 16.10.025 seems to be applicable to this note. That section is as follows:

> *16.10.025 Basis for establishing the areas of special flood hazard.*
>
> *The areas of special flood hazard identified by the Federal Insurance Administration (FIA) of the Federal Emergency Management Agency (FEMA) in the flood insurance study (FIS) dated April 15, 1986, and accompanying flood insurance rate maps (FIRMs) and flood boundary and floodway maps (FBFMs), dated April 15, 1986, and all subsequent amendments and/or revisions, are hereby adopted by reference and declared to be a part of this chapter. This FIS and attendant mapping is the minimum area of applicability of the flood regulations contained in this chapter, and may be supplemented by studies for other areas. The FIS, FIRMs, and FBFMs are on file at the County Government Center, Planning Department. [Ord. 4518-C § 2, 1999].*

We have no control over the ordinance or any requirement by the County of Santa Cruz to perform supplemental flood hazard analysis for the project. It is our understanding that the applicant has elected to accept the risk posed to the project that comes with compliance for the minimum prescriptive flood hazard parameters laid out in the FIS, FIRM and FBFM documents that apply to the project.

It seems irresponsible to specifically deflect a potential requirement for supplemental flood hazard analysis at this stage in the review and the project. Delaying that type of input will add unnecessary delay and cost to the project, particularly with respect all of the civil engineering and architecture that will need to be revised if the flood hazard parameters are changed later in the project. There is nothing unique about this site when compared to all the other homes and permits issued for work on Beach Drive and Las Olas Drive. If the County expects to require the project to go further than compliance with the applicable FIS, FIRM and FBFM documents, the correct time to make that comment and request would have been with the first submittal, not later in the project.

RESPONSE TO SELECT COMMENTS ISSUED IN 13 MARCH 2018 COUNTY OF SANTA CRUZ LETTER

We have been asked by the Project Architect Of Record, Matson-Britton Architects, to respond to the geological aspect of select comments issued in a letter dated 13 March 2018, titled "Subject: Incomplete Application - Additional Information Required - Application #: 181024; Assessor's Parcel #: 043-095-14 - Owner: Vaudagna". The comments and our responses can be seen below.

> *SCCC Section 16.10.040(59) defines a "shoreline protection structure" as any structure or material placed in an area where coastal processes (i.e. landsliding, surface runoff, wave action) operate. As such, the proposed structure (proposed retaining walls and deep piers under the home) must comply with SCCC Section 16.10.070(H)(3) as follows:*
>
> *Please provide a preliminary monitoring and maintenance plan for the proposed structure (SCCC 16.10.070(H)(3)(g).*
>
> *An alternatives analysis must be completed that demonstrates the proposed design is as safe as other potential designs (SCCC 16.10.070(H)(3)(c)).*
>
> *Please show that all protection structures shall meet approved engineering standards as determined through environmental review (SCCC 16.10.070(H)(3)(f)).*

Zinn Geology Response To Above Comments: Santa Cruz County Code Section 16.10.040(59) is as follows:

> *(59) "Shoreline protection structure" means any structure or material, including but not limited to riprap or a seawall, placed in an area where coastal processes operate.*

The mention of "riprap" and "seawall" are important, even when evaluating the qualifying passage "including but not limited to". The obvious intent of this passage is for structures that actually protect the shoreline from erosion, such as revetments and seawalls. The proposed development for the project does not include any structures that will prevent the shoreline from eroding (personal communication with Richard Irish of R.I. Engineering). The walls on the ground floor will be break-away walls that allow the passage of breaking waves. The piers upon which the house will be founded are designed to allow the passage of breaking waves. The retaining walls behind the house are NOT designed to withstand wave impact forces or being undermined by the design scour event for the house. Therefore none of the structures qualify as "shoreline protection structures" in our opinion.

We understand that there might some other bureaucratic reason for attempting to qualify the structural elements of the proposed development as "shoreline protection structures", but we felt it important to establish that none of the structures proposed will ever actually protect the bluff from being eroded should the design scour or wave run up occur during the design life of the project. We would also like to add that attempting to qualify components of the proposed residence as a "shoreline protection structure" is a slippery slope and somewhat obtuse. Why stop with any of the aforementioned elements? How about the drain pipes, the drain inlets, the gas pipes, the water supply pipes or the driveway surfacing? Those all would qualify using the same reasoning implied by the County comment (i.e. material placed in an area where coastal processes operate). The obvious response to our last point would be "that is ridiculous", but so is classifying break away walls, pier foundations and retaining walls as shoreline protection structures.

Santa Cruz County Code Section 16.10.070(H)(3) is as follows:

> *(g) All shoreline protection structures shall include a permanent, County approved, monitoring and maintenance program.*

We have established in our prior response above that there are no shoreline protection structures being proposed for the project, so this comment is invalid. Even if any structures for this project end up being classified as shoreline protection structures, we find the notion of establishing a monitoring and maintenance program for this project to be absurd. Adding this code citation to the County comments actually appears to undermine their notion that the proposed development includes shoreline protection structures, because maintenance agreements are typically reserved for true shoreline protection structures such revetments or seawalls that are subject to ongoing wave forces and erosion that actually degrade those structures. None of the structures proposed for this project will be subjected to ongoing wave forces or erosion and therefore cannot be monitored or maintained in the traditional sense with respect to those processes. Furthermore we question how a maintenance agreement and monitoring plan would be implemented. How will the piers be "inspected? What aspect of break-away walls will be inspected to determine that they will work as designed and break away when impacted by wave forces? How will the retaining wall be inspected to determine that it is NOT designed for wave impact or the design scour event?

Santa Cruz County Code section 16.10.070(H)(3)(c) is as follows:

> *(c) Application for shoreline protective structures shall include thorough analysis of all reasonable alternatives to such structures, including but not limited to relocation or partial removal of the threatened structure, protection of only the upper bluff area or the area immediately adjacent to the threatened structure, beach nourishment, and vertical walls. Structural protection measures on the bluff and beach shall only be permitted where nonstructural measures, such as relocating the structure or changing the design, are infeasible from an engineering standpoint or are not economically viable.*

We have thus far questioned the validity of classifying any components of the proposed development as shoreline protective structures. It is puzzling that the County would cite this passage at all, given the condition of the property and the current and proposed uses of the property. What alternative outcome to the current proposed development scheme is the County visualizing by requiring this code section being fulfilled? Any new house located on the property will be founded upon the underlying beach sand, be backed by the steep coastal bluff and will be

EXHIBIT J

subject to flooding due to wave run up, wave erosion of the underlying beach sand, liquefaction and differential settlement under the residence and debris flow impact from the slope above the residence. There is no room to relocate the residence on the property with respect to aforementioned geological hazards and all structures must be designed with break away walls in order to not occlude flooding due to wave run up. The site is not subject to active wave erosion and none of the elements that touch the soil or bedrock are designed to resist wave impact forces, so the project will not preclude future beach nourishment due to those processes. The project is not located on the "upper bluff" so that aspect of the citation does not apply to this project. One could attempt to construct the house ON the coastal bluff that is encompassed by the property, but that option would obviously still subject the residence to a landsliding hazard and would involve even more expensive foundation and structural options than are currently being proposed. The bottom line here is that there are no really no alternatives to be pursued for the project outside of what is currently being proposed, at least when viewed through the lens of the geologist, which makes the requested compliance with this code section unnecessary.

Santa Cruz County Code section 16.10.070(H)(3)(f) is as follows:

> *(f) All protection structures shall meet approved engineering standards as determined through environmental review.*

This code section applies to shoreline protection structures. As noted above, the project is not currently proposing to construct any shoreline protection structures, which makes compliance with this code section unnecessary.

Per ASCE 24 the entire development, and the proposed retaining walls located along the side property lines must be designed to avoid deflecting or increasing the flood risk to adjacent property. It appears that the proposed 7-foot retaining walls located on either side of the home may deflect and/or increase flooding on adjacent properties. In addition, the means of ingress and egress must be maintained after a landslide. Please demonstrate in the design calculations that the occupants can safely evacuate the building after a landslide event given the proposed concrete retaining walls encapsulate the entire building site.

Zinn Geology Response To Above Comments: The comment regarding deflecting and increasing the flood risk is puzzling. The original and new civil engineering design reflects excavation of the toe of the slope at the site, which will actually increase the volumetric capacity of flood waters at the site. Hopefully the County of Santa Cruz can substantively and quantitatively demonstrate to the design team that increasing flood storage capacity at a given site will somehow also increase the deflection, volume and elevation of flooding on adjacent properties. At this point that aspect of the comment makes no sense to us from the geological perspective.

EXHIBIT J

The only type of landslide that we are currently envisioning impacting the proposed development is a debris flow. Given that the ground floor walls are break-away and will allow for the debris flow deposit to pass through and possibly come to rest under the house, we do not understand this comment. The proposed residence currently has at least three ways of existing the upper floors - an external set of stairs, an internal set of stairs and an elevator. Even if in the unlikely event all of the stairs were partially occluded or torn away from the house, the occupants could step down on the landslide deposit. The absolute worst case scenario of no stairs left to use could be mitigated through the permanent installation of a rope ladder somewhere in the habitable portion of the residence, which would allow for occupants to exit the house through any suitable window after the landslide.

This concludes our response to the County of Santa Cruz comments.

Sincerely,
ZINN GEOLOGY

Erik N. Zinn
Principal Geologist
P.G. #6854, C.E.G. #2139

ERIK N. ZINN
No. 2139

ERIK N. ZINN
No. 6854

Attachments: Appendix A -28 OCTOBER 2016 LETTER BY JOHN KASUNICH
Appendix B - Plates 1 & 2

APPENDIX A

28 OCTOBER 2016 LETTER BY JOHN KASUNICH

HARO, KASUNICH AND ASSOCIATES, INC.

CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

Project No. SC11033
28 October 2016

BRET GRIPENSTRAW
158 Towne Terrace
Santa Cruz, CA 95060

Subject: Winter 2015/2016 Storm Damage
Coastal Bluff Repair Recommendations

Reference: 340 Kingsbury Drive
Aptos, California

Dear Mr. Gripenstraw:

At your request, we inspected the referenced coastal bluff after damage that occurred during intense rainfall events of early March 2016. The purpose of our site inspection was to evaluate the storm damage to the bluff face, to inspect the Geobrugg TECCO slope stabilization system implemented on the bluff face in 2013 and to determine appropriate repair and improvement recommendations.

The mesh for the Geobrugg TECCO slope stabilization system was anchored to the slope face using helical screw anchors in March 2013. The purpose of the Geobrugg TECCO system was to stabilize the surficial soils that mantle the steep coastal bluff. The coastal bluff at the referenced property has historically experienced shallow debris flows and translational landslides. Displaced slide masses cover much of the slope and an exposed scarp exists along its top and midsection. Tension cracking along the downcoast top edge of the bluff was noted during our site inspection. Prior geologic and geotechnical investigations by other firms for the residential development indicated that the bluff face is inherently unstable because it is too steep for the soil that mantles it and is exposed and raw from prior historical landslide events. The slope stabilization system was placed in tandem with seeding to serve dual duty for the purpose of shallow soil stabilization and erosion control. The properties on Beach Drive below the reference site have not addressed the erosion on their slope and the Geobrugg TECCO system has the risk of eventually being undermined along its lower (southern) boundary.

During the intense rainfall events of early March 2016, the coastal bluff became saturated due to direct rainfall and in two locations, concentrated storm water runoff spilling over the top of the bluff. The saturated condition of the soils and the increased pore water pressure associated with the saturation reduced shear strength of the near surface soil mantle, causing slumping and sloughing of soil under the mesh. Additionally, a pre-existing shallow landslide mass remobilized during the rainfall events, leaving a near vertical, six- to eight-foot high escarpment right at the lower property line, directly below the bottom boundary of the slope stabilization system, causing the removal of sub-adjacent lateral support and creating a deficient support condition.

EXHIBIT J

The Geobrugg slope stabilization system could only be installed within the boundaries of the Meyerhoff property, and therefore can only contain the soil on the bluff within the boundaries of the referenced property. The properties along Beach Drive, directly below the reference site, were also impacted by slope saturation due to the intense rainfall and the remnant near vertical landslide escarpments. The most downcoast southeast corner of the bluff top also experienced a moderately large debris flow due to its' oversteepened condition, soil saturation, poor erosion control cover and storm water runoff cascading over the top of the bluff. The ground cover implemented during the installation of the slope stabilization system had only partially germinated prior to the March storms, which exacerbated the loss of soil under the wire mesh.

A number of site meetings have taken place since the damage occurred in March with the original design team, the project contractors and with John David of Prime Landscape. This team has reviewed the damage to the existing Geobrugg TECCO slope stabilization system due to the 2016 March storm and formulated a repair plan to complement the existing slope stabilization system minimizing the occurrence of future potential landslide deposits from mobilizing on the Meyerhoff property and impacting the neighboring properties below.

A number of alternatives were reviewed including: placing buried pin piles on the bluff face below the toe of the existing slope stabilization system (very difficult to stage the appropriate drilling equipment at this location); constructing a tied-back, reinforced shotcrete, compression plate below the toe of the existing slope stabilization system (easier to construct with portable equipment but would be visible from the beach), or extending the existing soil nails into the slope and adding additional nails and reinforcing as necessary to shotcrete the existing mesh (very visible from the beach). It is important to note the neighbors below the property have not to date wanted to participate in stabilizing the bluff. As noted previously, the existing slope stabilization system extends only to the lower boundary of the subject property and does not protect the portion of the bluff owned by the neighboring properties below. Stabilizing the bluff below and beyond the portion of the bluff already protected will require improvements on the neighboring properties. Hence, any notion of creating an integrated or even hybrid top-to-bottom bluff face landslide mitigation system has not been considered by the team.

The consensus group conclusions evolved toward the following recommendations made:

1) Repair the erosion damage under and at the edges of the Geobrugg TECCO slope stabilization system. Prime Landscape will infill minor voids that formed under the upcoast (west side) mesh with fertilized mulch and replant with native grass sod. The downcoast (east side) mesh will have to be disconnected from the soil nails and rolled up to access the deeper voids on the bluff face. The deep void areas will be hand dressed to smooth the bluff face surface sufficiently to allow tight replacement of the mesh. The nail heads will be reconnected and mulch and native grass sod applied. A controlled, temporary irrigation system will be placed and used until the native grass root structure is well established. The irrigation system will then be removed.

2. Minor grading at the top of the downcoast point where no restraint system exists to lessen vertical scarp instabilities. Re-cover the graded area with a durable erosion control fabric.

3. Surface drainage improvements to the front yard, including a buried pond liner directing near surface seepage away from the blufftop when the landscape improvements are implemented, as well as minor hand grading to prevent future yard runoff from spilling over the bluff top on to the bluff face, and instead directing it to the existing storm drain inlet box at the southeast corner of the house.

The recommendations presented above will repair damage to the coastal bluff and Geobrugg TECCO slope stabilization system that occurred last winter and restore the project site to the same or better conditions that existed prior to the storm damage.

The proposed repairs to the Geobrugg TECCO Mesh system and seaward bluff are shown on the plans by RI Engineering, Inc. entitled "Slope Protection Plan" delta H, dated 8/22/16. The plan shows the limit of the Geobrugg TECCO system revisions and the proposed regrading of the southern corner of the bluff.

If you have any questions, please contact our office.

Reviewed By:

Pacific Crest Engineering, Inc.
Elizabeth M. Mitchell, G.E.

Zinn Geology
Eric Zinn, G.E.G.

RI Engineering, Inc.
Richard Irish

Respectfully Submitted,

HARO, KASUNICH AND ASSOCIATES, INC.
John E. Kasunich, G.E. 455

JEK/sr
Copies: 1 to Addressee + pdf
1 to Jens Meyerhoff + pdf
2 to Joe Hanna, Santa Cruz County Planning Dept. + pdf

APPENDIX B

PLATES 1 AND 2

EXHIBIT J

Basis of Elevation

County Benchmark 477 being a 2"brass disk stamped "Santa Cruz County Surveyor BM 477".
Elevation = 17.19 feet NAVD88

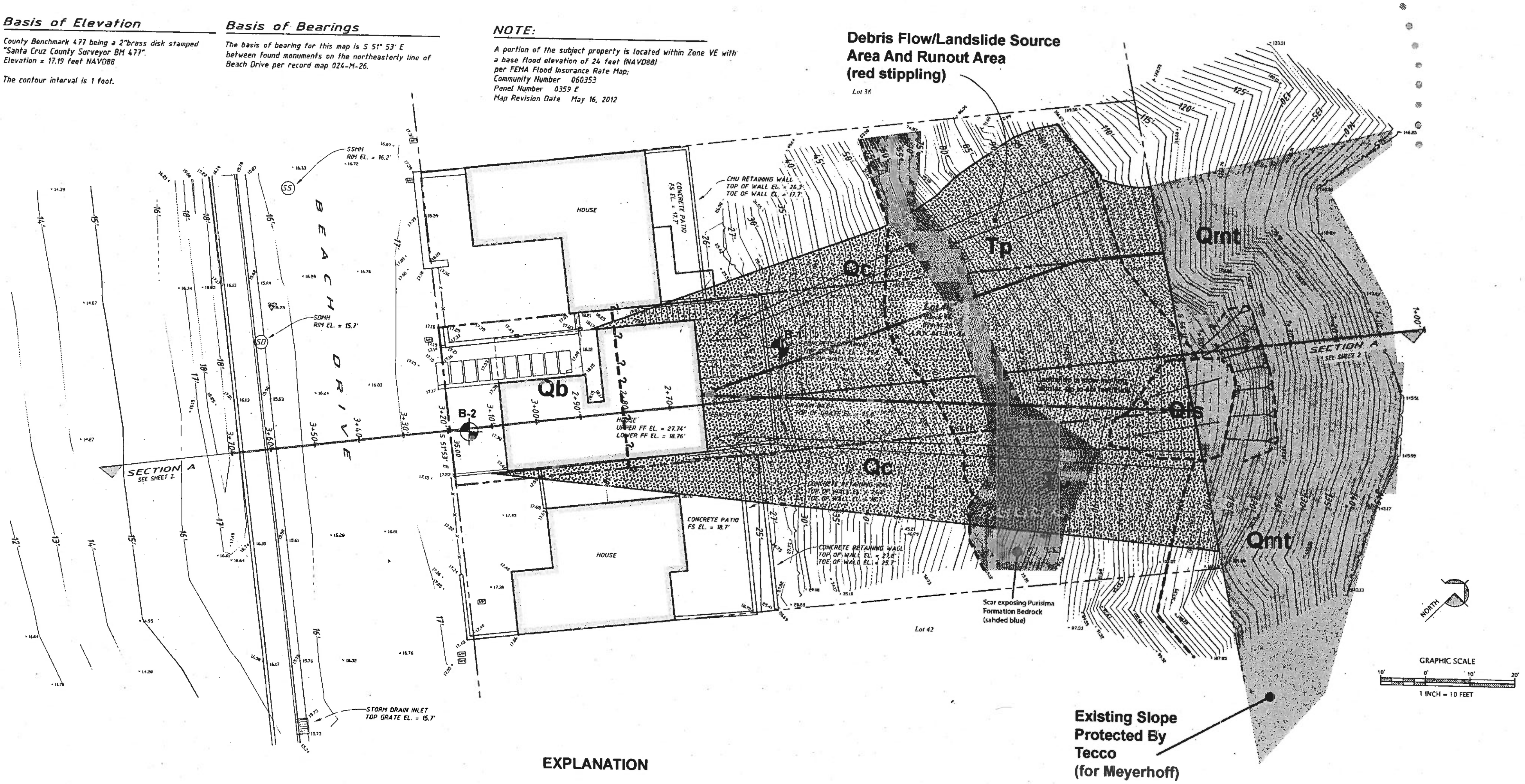
The contour interval is 1 foot.

Basis of Bearings

The basis of bearing for this map is S 51° 53' E between found monuments on the northeasterly line of Beach Drive per record map 024-M-26.

NOTE:

A portion of the subject property is located within Zone VE with a base flood elevation of 24 feet (NAVD88) per FEMA Flood Insurance Rate Map;
Community Number 060353
Panel Number 0359 E
Map Revision Date May 16, 2012



Debris Flow/Landslide Source Area And Runout Area (red stippling)

Existing Slope Protected By Tecco (for Meyerhoff)

BASE MAP: Excerpted from *"Topographic Map, The Lands Of Jim And Sue Vaudagne - 379 Beach Drive - Aptos, CA 95003"* by Hanagan Land Surveying, dated 10 May 2017, intended publication scale 1"=10'.

ZINN GEOLOGY

Geological Site Map
Lands of Vaudagne
379 Beach Drive
Aptos, California

Date: 12 November 2017	Revised: 8 August 2018
Job #2017011-G-SC	
Scale: 1"=10'	**Plate 1**
Drawn by: ENZ	

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STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
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EXPLANATION

EARTH MATERIALS

Qbs	Beach sand
Qls	Landslide deposit
Qc	Colluvium
Qmt	Marine terrace deposits
Tp	Purisima Formation

SYMBOLS

A├──┤A'	Location of geologic cross sections
── — —?—?	Earth materials contact - dashed where approximate, queried where uncertain
⊕ B-2	Location of small-diameter exploratory test borings advanced by Pacific Crest Engineering

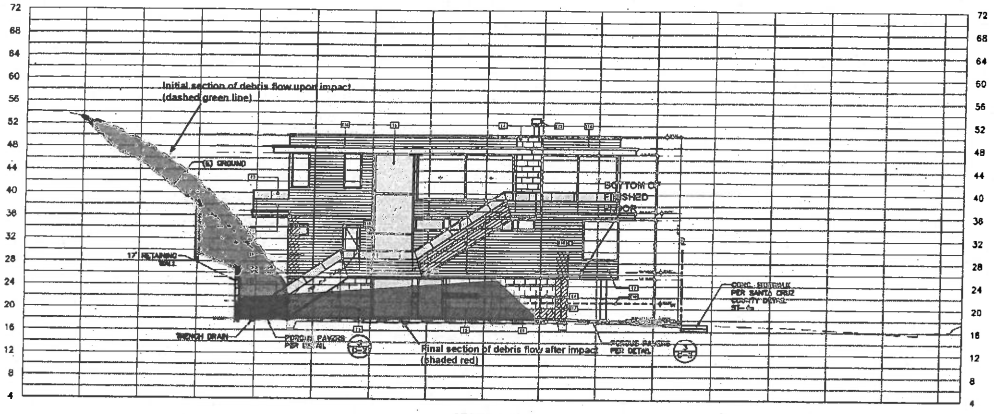
PARAMETERS TAKEN FROM GEOBRUGG SHALLSLIDE ONLINE TOOL AND PACIFIC CREST ENGINEERING SLOPE STABILITY ANALYSIS

Width of starting volume = 9 meters (29.5 feet)
Total starting volume of shallow landslide = 176 cubic meters (230 cubic yards)
Density of shallow landslide materail = 2100 kilograms per cubic meter (131 pounds per cubic foot)
Distance from breakout zone to barrier location = 10 meters (33 feet)
Spreading angle of starting volume = 5 degrees
Additional width at impact section on each side of starting width = 1 meter (3.3 feet)
Width of shallow landslide at impact point = 11 meters (36 feet)
Maximum speed of shallow landslide at impact section = 7.6 meters per second (25 feet per second)
Travel time of landslide front between starting zone and impact section = 1.32 seconds
Peak discharge = 134 cubic meters per second (175 cubic yards per second)
Flow height = 1.63 meters (5.3 feet)
System height of the filled barrier = 2.63 meters (8.6 feet)
Slope angle = 70%
Inclination of retained materail behind the barrier = 15%
Retention volume per linear meter = 6.92 cubic meters per meter (2.8 cubic yards per foot)
Total retention volume = 76 cubic meters (99 cubic yards)
Overflow = 100 cubic meters (131 cubic yards)

NOTE: The map above also presumes that the side and low-side boundaries of the existing Geobrugg Tecco Mat on the property above the subject property are capable of providing sediment from erosion and debris flows up to eight feet inboard from the aforementioned edges of the Tecco Mat.

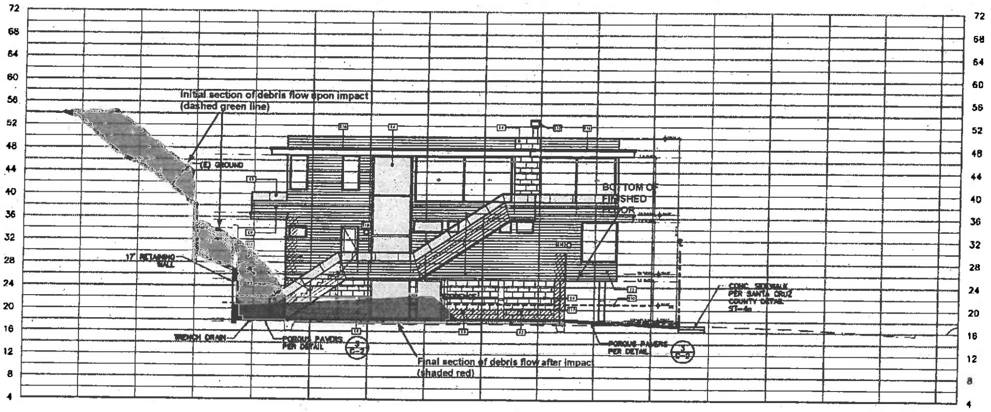
EXHIBIT J

SCENARIO ONE
DEBRIS FLOW WITH NO IMPACT OR CATCHMENT STRUCTURES



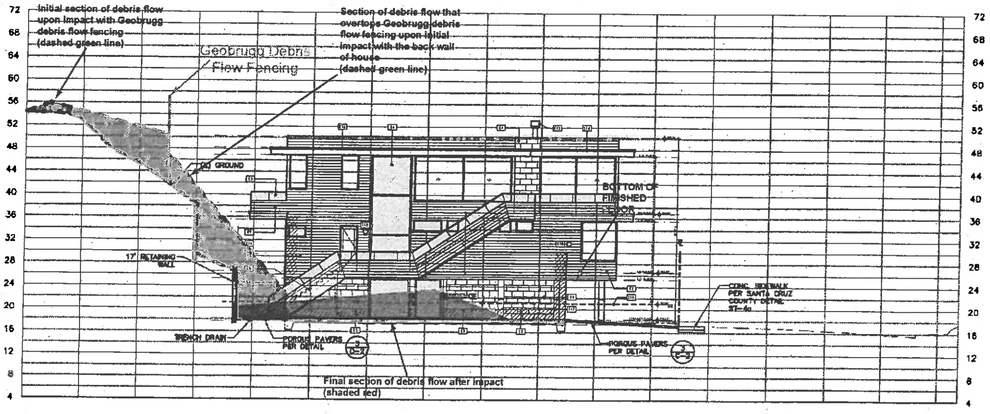
SECTION A–A
SCALE: 1"=10' HORIZONTAL, VERTICAL

SCENARIO TWO
DEBRIS FLOW WITH IMPACT WALLS PLACED AT TOP OF RETAINING WALLS



SECTION A–A
SCALE: 1"=10' HORIZONTAL, VERTICAL

SCENARIO THREE
DEBRIS FLOW WITH ONLY GEOBRUGG DEBRIS FLOW FENCING



SECTION A–A
SCALE: 1"=10' HORIZONTAL, VERTICAL

PARAMETERS TAKEN FROM GEOBRUGG SHALLSLIDE ONLINE TOOL AND PACIFIC CREST ENGINEERING SLOPE STABILITY ANALYSIS

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Retention volume per linear meter = 6.92 cubic meters per meter (2.8 cubic yards per foot)
Total retention volume = 76 cubic meters (99 cubic yards)
Overflow = 100 cubic meters (131 cubic yards)

BASE SECTION: Base section used in all three scenarios shown above was constructed using digital excerpts from Section A-A' from R.I. Engineering Sheet C-2, dated March 2018 and the house section was digitally excerpted from the West Elevation from Matson-Britton Architects Sheet A3.1 dated 11 October 2017.

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ZINN GEOLOGY

DEBRIS FLOW OUTCOMES FOR THREE DIFFERENT MITIGATIVE SCENARIOS
Lands of Vaudegne
379 Beach Drive
Aptos, California

Date: 8 August 2018	Revised:
Job #2017011-G-SC	
Scale: 1"=10', h=v	**Plate 2**
Drawn by: ENZ	

EXHIBIT J

COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT

701 OCEAN STREET - 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123
KATHLEEN MOLLOY PREVISICH, PLANNING DIRECTOR

November 7, 2018

Matson Britton Architects
728 N Branciforte
Santa Cruz, CA 95062

Subject: **Incomplete Application - Additional Information Required**
Application #: **181024**; Assessor's Parcel #: **043-095-14**
Owner: **Vaudagna**

Dear Matson Britton Architects:

This letter is an update on the status of your application. On 10/9/18, you submitted additional materials for the above listed development permit. The most recent submittal has been reviewed and it has been determined that your application remains **incomplete**. Additional information continues to be necessary to allow further processing of your application. For your proposal to proceed, please submit the following items:

1. Please submit 3 full and complete sets of revised plans (and one 8.5" x 11" reduced set) which includes all revisions required by Environmental Planning (see attached comments).

2. Please also refer to the **Compliance Section** of this letter as some comments may require modifications to the proposed design in order for staff to support your proposal.

3. This application includes a combined geological and geotechnical report review, which is currently in process. Please note that the result of this review may include a request for additional information if there are technical issues that were not fully addressed in the report. This application will remain incomplete until the technical review is finished. We will notify you of the outcome of the technical review when it is completed.

4. Please review the attached comments from all agencies. Comments which require additional information to be submitted must be addressed and resolved prior to your application being considered complete and able to move forward with the review. The agencies listed below have comments which will require additional information to be submitted. Questions related to these comments and the specific information that is required should be addressed to each separate agency:

    - <u>Environmental Planning</u> Jessica deGrassi (831) 454-3162: See attached comments

5. Please submit an annotated list detailing where the required information has been provided in your next submittal. Please affix a copy of the annotated list, and required submittal materials (technical reports, drainage calculations, arborist report, etc.) to each agency plan set prior to submittal of all the plans to ensure that requested materials are routed to the appropriate agencies.

EXHIBIT K

6. Please note that you will be required to install signage on the subject property that notifies the public of your development permit application. Please refer to the Neighborhood Notification Guidelines for the standards for preparing your sign. Please do not prepare or install the sign until all other completeness issues have been resolved as the description may change during the review process. Guidelines for Neighborhood Notification online: www.sccoplanning.com (under Zoning & Development, Brochures link). If you do not have internet access and require a paper copy, please let us know and one can be provided to you.

You must submit the required materials to the Planning Department at one time. Revisions to plans must be included in complete, updated sets of plans. All plan sets must be individually stapled and folded into an ~ 9" x 12" format (per Folding Plans handout). To reduce waste and to aid in recycling efforts, plan sets should be printed on bond (white) paper and should not include colored binding material of any kind. You have until **January 7, 2019**, to submit the all of the information required in this letter. Pursuant to Section 18.10.430 of the Santa Cruz County Code, failure to submit the required information may lead to abandonment of your application and forfeiture of fees. Alternatively, you may withdraw the application and any unused fees will be refunded to you. If you wish to withdraw the application, please notify me in writing.

You have the right to appeal the determination that the application is incomplete pursuant to Section 18.10.320 of the County Code and Section 65943 of the Government Code. To appeal, submit the required fee for administrative appeals and a letter addressed to the Planning Director stating the determination appealed from, and the reasons you feel the determination is unjustified or inappropriate. The appeal letter and fee must be received by the Planning Department no later than 5:00 p.m., November 21, 2018

Compliance Issues

In addition to evaluating the completeness of your application, the initial review has identified areas in which your proposal is in conflict with applicable codes and policies. Although it is not necessary for you to address the compliance issues for your application to be declared complete, you will need to resolve these issues in order to achieve compliance with the codes and policies that pertain to your development proposal. Planning Department staff cannot support an application that is not in compliance with County ordinances, General Plan policies, or other areas of applicable law. Please review the attached comments from all reviewing agencies. The areas of conflict with applicable codes and policies identified in this preliminary review are listed below:

- County Code Section 13.10.323 (E)(1) allows for unenclosed stairways and landings to extend three feet into a side yard. As proposed, the exterior stairway located on the north side of the home encroaches 3.5 feet into the side yard. Special circumstances do not appear to exist that would warrant granting of a variance for the proposed encroachment which exceeds the allowance under SCCC 13.10.323(E)(1). Therefore, the project shall be conditioned to ensure the final plans (building permit stage) comply with the above code section and the stairway (and railing) do not encroach more than three feet into the required five foot side yard setback.

- County Code Section 13.10.323(C):
  As previously indicated, County of Santa Cruz Planning Department Administrative Practice Guideline requires that Gross Floor Area (all "buildings") be included in the Floor Area Ratio Calculation). Gross Floor Area is defined as the primary dwelling(s), including any attached or detached garage (minus 225 square foot credit), and any other habitable or non-habitable structure on the site, whether attached or detached.

EXHIBIT K

Approximately 690 square feet of entry/storage at the first floor was omitted from the FAR calculations. This area shall be counted as it is non-habitable area contained within the main dwelling. Clarification on building components that count toward FAR are provided at the following link: http://www.sccoplanning.com/Portals/2/County/Planning/policy/interpretations/FAR-%20Admin%20%20Guidelines%2011-3-15.pdf

The addition of the lower floor to the FAR calculation exceeds the allowed 50% FAR and requires a variance. Special circumstances such as the location of the proposed development exist and the granting of a variance to height and number of stories is supported by the character of the surrounding area. Consequently, a variance to FAR is necessary and a result of the other variances being requested. A variance to FAR shall be processed as part of the application and it is advised that future plan revisions include the square footage of the lower floor in the Floor Area Ratio calculations required.

- SCCC Section 16.10.040(59) defines a "shoreline protection structure" as any structure or material placed in an area where coastal processes (i.e. landsliding, surface runoff, wave action) operate. As such, the proposed structure (proposed retaining walls and deep piers under the home) must comply with SCCC Section 16.10.070(H)(3) as follows:
    - Please provide a preliminary monitoring and maintenance plan for the proposed structure (SCCC 16.10.070(H)(3)(g).
    - An alternatives analysis must be completed that demonstrates the proposed design is as safe as other potential designs (SCCC 16.10.070(H)(3)(c)).
    - Please show that all protection structures shall meet approved engineering standards as determined through environmental review (SCCC 16.10.070(H)(3)(f)).
- California Building Code (CBC) Section 1808.7.2 requires a setback from descending slopes to the face of the structure that is equal to at least the smaller of half the height of the slope and 15 feet. The current proposal includes a second floor deck which is setback approximately 4 feet from the face of the slope (Sheet P6). As proposed, the project cannot be approved, as a reduction to this setback cannot be granted given the known and ongoing slope instability above the proposed home. In order to consider a reduced setback the geotechnical engineer must demonstrate that the entire volume of the projected debris flow will solely pass under the home, without being scattered by the flow impacting portions of the home (i.e. second floor deck).

- Per ASCE 24 the entire development, and the proposed retaining walls located along the side property lines must be designed to avoid deflecting or increasing the flood risk to adjacent property. It appears that the proposed 7-foot retaining walls located on either side of the home may deflect and/or increase flooding on adjacent properties. In addition, the means of ingress and egress must maintained after a landslide. Please demonstrate in the design calculations that the occupants can safely evacuate the building after a landslide event given the proposed concrete retaining walls encapsulate the entire building site.

- All walls below the base flood elevation are required to be break away during base flood conditions, including those that separate the storage, entry, stairway and garage. The ground flood must be non-habitable in order to comply with this requirement. Therefore, separation between the ground floor and the first floor is required. Please show the required break away walls on all relevant plan sheets.

Preliminary Conditions of Approval:

1. The applicant has indicated that they no longer wish to continue the vacation rental. A written statement surrendering the existing vacation rental application (111478) will be required as a condition of approval and prior to issuance of a building permit.

2. Prior to issuance of a building permit, the stairway (including railing) located on the north side of the home shall be revised so than the stairway (and railing) do not encroach more than three feet into the required five foot side yard setback. See the Compliance section of this letter for more information regarding this condition.

Additional Information

The following items are included as general information and do not need to be addressed in order for your application to be declared complete.

A. Please review the attached comments from all agencies. Comments may specify Conditions of Approval for this permit, if approved, or other requirements which must be met prior to approval of any Building or Grading Permit(s) for this project. Questions related to these comments can be addressed to each separate agency.

B. Please note that additional sets of revised full size plans and two sets of revised reduced (8.5" x 11") plan sets will be required prior to the public hearing for this project.

Should you have further questions concerning this application, please contact me at:
(831) 454-3118, or e-mail: nathan.macbeth@santacruzcounty.us

Sincerely,

Nathan MacBeth
Project Planner
Development Review

County of Santa Cruz, PLANNING DEPARTMENT

Discretionary Application Comments 181024

APN 043-095-14

Your plans have been sent to several agencies for review. The comments that were received are printed below. Please read each comment, noting who the reviewer is and which of the three categories (Completeness, Policy Considerations/Compliance, and Permit Conditions/Additional Information) the comment is in.

Completeness: A comment in this section indicates that your application is lacking certain information that is necessary for your plans to be reviewed and your project to proceed.

Policy Considerations/Compliance: Comments in this section indicate that there are conflicts or possible conflicts between your project and the County General Plan, County Code, and/or Design Criteria. We recommend that you address these issues with the project planner and the reviewer before investing in revising your plans in any particular direction.

Permit Conditions/Additional Information: These comments are for your information. No action is required at this time. You may contact the project planner or the reviewer for clarification if needed.

Environmental Planning

Routing No: 3 | Review Date: 11/07/2018

Jessica DeGrassi (JDEGRASSI) : Incomplete

3rd Review Comments

Completeness Comments

1. The geologic and geotechnical reports are currently under review (REV181023). Results of this review will be addressed under a separate letter.

2. Please provide grading and drainage plans, to include a construction plan for the project that shows the necessary grading and shoring for the construction of the home.

3. Please revise sheets C1 and C2 to show the second floor deck as drawn on sheet P3 (refer to compliance comment 2 below).

4. Please submit a section through the southeasternmost portion of the upper retaining wall demonstrating slope setback to the second floor deck. Sheet P6 shows the second floor deck at 9-feet from the uppermost retaining wall (refer to compliance comment 2).

5. Revised sheet C1 notes the proposed landscape wall along the sides of the home will be "breakaway", but

EXHIBIT K

Environmental Planning

Routing No: 3 | Review Date: 11/07/2018
Jessica DeGrassi (JDEGRASSI) : Incomplete

sheet P4 indicates that these walls will be concrete. Please clarify and refer to compliance comment 3.

6. Sheet P3 shows fill in front of the upper retaining wall, whereas sheet C2 shows an open-faced retaining wall. Please clarify.

<u>Compliance Comments</u>

The following comments may be revised once the report reviews have been completed.

1. SCCC Section 16.10.040(59) defines a "shoreline protection structure" as any structure or material placed in an area where coastal processes (i.e. landsliding, surface runoff, wave action) operate. As such, the proposed structure (proposed retaining walls and deep piers under the home) must comply with SCCC Section 16.10.070(H)(3) as follows:
a. Please provide a preliminary monitoring and maintenance plan for the proposed structure (SCCC 16.10.070(H)(3)(g).
b. An alternatives analysis must be completed that demonstrates the proposed design is as safe as other potential designs (SCCC 16.10.070(H)(3)(c)).
c. Please show that all protection structures shall meet approved engineering standards as determined through environmental review (SCCC 16.10.070(H)(3)(f)).

2. California Building Code (CBC) Section 1808.7.2 requires a setback from descending slopes to the face of the structure that is equal to at least the smaller of half the height of the slope and 15 feet. The current proposal includes a second floor deck which is setback approximately 4 feet from the face of the slope (Sheet P6). As proposed, the project cannot be approved, as a reduction to this setback cannot be granted given the known and ongoing slope instability above the proposed home. In order to consider a reduced setback the geotechnical engineer must demonstrate that the entire volume of the projected debris flow will solely pass under the home, without being scattered by the flow impacting portions of the home (i.e. second floor deck).

3. Per ASCE 24 the entire development, and the proposed retaining walls located along the side property lines must be designed to avoid deflecting or increasing the flood risk to adjacent property. It appears that the proposed 7-foot retaining walls located on either side of the home
may deflect and/or increase flooding on adjacent properties. In addition, the means of ingress and egress must maintained after a landslide. Please demonstrate in the design calculations that the occupants can safely evacuate the building after a landslide event given the proposed concrete retaining walls encapsulate the entire building site.

4. All walls below the base flood elevation are required to be break away during base flood conditions, including

EXHIBIT K

Environmental Planning

Routing No: 3 | Review Date: 11/07/2018
Jessica DeGrassi (JDEGRASSI) : Incomplete

those that separate the storage, entry, stairway and garage. The ground flood must be non-habitable in order to comply with this requirement. Therefore, separation between the ground floor and the first floor is required. Please show the required break away walls on all relevant plan sheets.

COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT
701 OCEAN STREET, 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123
KATHLEEN MOLLOY, PLANNING DIRECTOR

July 18, 2019

DiBenedetto & Lapcevic, LLP
Attn: Anna DiBenedetto
1101 Pacific Avenue, Ste. 320
Santa Cruz, California 95060

Subject: Clarification of Technical Issues for 379 Beach Drive

Project Site: 379 Beach Drive
APN 043-095-1
Application No. REV181023

Dear Ms. DiBenedetto:

This letter outlines County staff's responses to your request for clarification of technical comments presented in your email to Carolyn Burke on 18 July 2019. This letter also presents our concerns regarding the Moderate to Large Scale Landslide Hazards at the project site. Our concerns on this matter were previously transmitted informally in an email to Erik Zinn on May 8, 2019. We have raised this issue because we consider such landsliding to present a real life-safety hazard to the proposed development.

We were requested "to be specific about which technical issues they [we] are disputing regarding the slope stability model and analysis as presented in Pacific Crest's 8/16/18 response letter to County Comment #1 (please see letters prepared by both Pacific Crest and Zinn Geology, and specifically refer to the last paragraph of Comment #1 in the PCE letter." Our comments in this letter also address the 25 April 2019 email from Elizabeth Mitchell to Rick Parks to clarify the PCE 16 August 2018 response letter.

County Staffs Response to Requested Comments

1. From the 25 April 2019 email from Elizabeth Mitchell to Rick Parks:
"Please note – in our opinion applying a factor of safety of 1.5 for static and 1.1 for seismic to determine debris flow volume is not applicable to these analyses. For calculating a debris flow volume for design one should use the volume that is predicted to fail. Additional safety factors are then considered in the design of mitigation measures/structures to contain or redirect that flow."

County staff agrees a FS = 1.0 for a limit equilibrium type slope stability analysis represents the mathematical point below which the slope could fail. The concern is the potential difference between the idealized/simplified soil slope model and the real slope. As well, once the slope fails, the remaining slope will have reduced stability, leading to additional debris pulses.

The approximate 120-foot high slope above the proposed residence has been modeled as three soil types with an associated soil shear strength and unit weight for each soil type. A FS = 1.0 for

EXHIBIT L

a simplified soil slope model does not account for: uncertainties and variabilities of the soil profile both vertically and laterally; reliability of input parameters; and the limitations of analyses methods. Determination of an appropriate magnitude for a factor of safety should also include the consequences of slope failure, and when applicable, the unacceptable performance of structural elements.

The horizontal seismic coefficient is a primary input parameter for pseudo-static analyses. The selection of the seismic coefficient is not currently codified. We have noted a wide range of seismic coefficients utilized along Beach Drive in reports submitted to the County for review by a variety of consultants. The project horizontal seismic coefficient of 0.16 is at the lower end of seismic coefficients utilized by consultants along Beach Drive and minimizes the magnitude of slope failure.

The landslide debris mass volume is a primary component for the design of the blufftoe residence. Typically, the landslide debris mass from a slope failure with a FS = 1.1 is a larger volume than a slope failure with a FS = 1.0. The 25 April 2019 email from Elizabeth Mitchell to Rick Parks also states, "Additional safety factors are then considered in the design of mitigation measures/structures to contain or redirect that flow." We maintain our requirement for a minimum pseudo-static Factor of Safety of at least 1.1 for slope stability analyses.

2. We were also requested to specifically refer to the last paragraph of Comment #1 in the PCE letter dated 16 August 2018. The specified last paragraph:
   "In our opinion both Scenario 2 and 3 provide adequate means for reducing the risk that debris will impact the habitable portion of the house. Preliminary design for impact walls should be based on an impact loading of 1900 psf. Preliminary design of debris flow fences should be based on the parameters presented in Plate 2, Reference 5. Design of all impact structures should include "wing walls" that confine the debris to the site and prevent it from being deflected onto the adjacent properties. We request the opportunity to review proposed designs for debris fences or impact walls and to provide additional geotechnical design recommendations as needed."

County staff's understanding of the proposed project slope stabilization/soil confinement system to accommodate the design of the elevated blufftoe residence is as follows:

- The existing blufftop Tecco steel mesh anchored with either helix screw anchors or grouted soils nails (conflicting anchor types/lengths are referenced in the project documents);
- A mid-bluff face Geobrugg steel debris net system;
- Blufftoe retaining walls with above grade debris impact walls to contain slope debris; and
- Siting the blufftoe residence above and seaward of the design landslide debris mass.

We agree with the consultants that the above outlined system or some similar variation can protect the occupants of the proposed blufftoe structure from surficial debris flows. A caveat would be the slope stabilization/soil confinement system will need to be monitored and maintained for the design life of the proposed residence to function as designed. We anticipate the bluff top Tecco steel mesh and the mid-slope Geo-brugg steel net will need to be replaced at some time in the future due to corrosion. Both the mid-slope Geo-brugg steel net and the blufftoe impact walls will need to be cleared of accumulated debris to re-establish design debris mass capacity.

We also agree there may be enough redundancy in the proposed project slope stabilization/soil confinement system to accommodate a larger volume of surficial slide mass. Please quantify the capacity of the proposed project slope stabilization/soil confinement system and develop an

EXHIBIT I

estimate of the Factor of Safety for the project debris mass storage capacity. Provided that sufficient excess capacity exists in the proposed system, we will accept the proposed mitigation.

Moderate to Large Scale Landslide Hazards

For the purposes of this discussion, we consider moderate to large scale landsliding to include landslides in excess of about 15 feet deep originating at or near the crest of the coastal bluff, of either translational or rotational mechanism. The potential for such landslides to occur at the project site is suggested by several lines of evidence:

1. There was a large landslide that destroyed two homes at 337 and 339 beach drive in 1982, a short way up coast from the project site. A photograph of the landslides and damaged houses was provided with the May 8, 2019 email. From the photo, it appears that the landslide was not highly fluid—there appears to be a slump type landslide mass or "sand flow" on the slope behind the houses and there is no evidence for a liquefied mass having flowed out into the street. Comparison of pre-landslide topographic mapping prepared by Santa Cruz County (Towill, Inc.,1965; 1"=100' scale) and topographic contours produced from Lidar coverage of the County (AMBAG, 2009) indicates that the landslide mass at origination was about 20' thick, when adjusted for the change in vertical datum. We do not consider this evidence definitive, but strongly indicative of moderate to large scale landsliding, as distinct from debris flows.

2. We noted evidence for three large scale landslides that had formed in the coastal bluff up coast from the project in 1928 aerial photos (frames 20-25, flight 1928H, UCSC aerial photo collection.

3. A number of geotechnical studies have identified the potential for moderate to large scale landsliding through slope stability analyses (quantitative or qualitative). We list three relevant studies here:

a. Haro, Kasunich, and Associates, 2001, Geotechnical Investigation for APN 043-095-11, 385 Beach Drive;
b. Haro, Kasunich, and Associates, 2002, Geotechnical Investigation for APNB 043-095-12, 383 Beach Drive;
c. Zinn Geology, 2016, Focused Geologic Investigation of Debris Flow Hazards for Existing Residence and Proposed Deck,615 Beach Drive, APN 043-152-28.

All three of these reports identify a potential for a 20' thick wedge or slab of sand to fail from the bluff above the homes in a non-fluid or only partially fluid state. The Haro, Kasunich, and Associates reports are for new houses located two and three doors downcoast from 379 Beach Drive.

There have been a large number of slope stability analyses performed for projects along Beach Drive. Not all of the analyses identify a potential for moderate to large scale landslides. In some cases, the critical depths are on the order of 6 to 8 feet. We are not taking the position that the observational evidence and analytical studies cited above prove that there is a moderate to large scale landslide hazard at the subject property, but that sufficient concern exists that a thorough analysis of the hazards associated with such an event must be performed.

Although the previous landsliding of the bluff identified in photos is located hundreds or thousands of feet from the subject property, the geologic and geomorphic conditions at the different landslide locations and the present site are similar. The geologic susceptibility to landsliding at the subject site must also be considered similar, until proven otherwise.

The reason for evaluating the potential for medium to large scale landsliding at this site is that such landsliding will undercut the 14' to 16' deep tecco mesh slope stabilization scheme located above the proposed residence. Consequently, the teccomesh system it will provide little or no protection for the project in the event of a moderate to large scale landslide event. The geologic report by Zinn Geology and the geotechnical report by Pacific Crest Engineering do not address the potential for moderate to large scale landsliding, nor do any of the response to comment letters provided by the project consultants.

We have reviewed geotechnical reports for all new homes built on landward side of Beach Drive in about the last 20 years. Every project we have looked at has included a global analytical slope stability analysis of the bluff behind the proposed home. Our review indicates that there is a well established local standard of practice for geotechnical analysis of new homes. We require a properly constituted slope stability analysis of the entire bluff face for this project. The analysis should correctly model the teccomesh system as installed. It is our understanding that the system consists of screw anchors embedded about 14' deep. Should a landslide risk be identified that is not adequately mitigated by the existing teccomesh system and the currently proposed debris flow mitigation scheme, new mitigation measures must be implemented.

We are willing communicate or meet with you, formally or informally, to help expedite this analysis. Please contact Rick Parks at (831) 454-3168/email: Rick.Parks@santacruzcounty.us or Jeff Nolan at (831) 454-3175/Jeffrey.Nolan@santacruzcounty.us if we can be of any further assistance.

Sincerely,

Rick Parks, GE 2603
Civil Engineer
Planning Department
County of Santa Cruz

RICK L. PARKS
No. 2603
Exp. 7-15-19

Jeffrey Nolan, CEG 2247
County Geologist
Planning Department
County of Santa Cruz

JEFFREY M. NOLAN
2247

Cc: Pacific Crest Engineering, Inc. Attn: Soma Goresky, GE
Zinn Geology, Attn: Eric Zinn, CEG

Pacific Crest
ENGINEERING INC

GEOTECHNICAL | ENVIRONMENTAL | CHEMICAL | MATERIAL TESTING | SPECIAL INSPECTIONS

August 31, 2019

Project No. 1738-SZ70-B44

Jim and Sue Vaudagna
19501 Scotland Drive
Saratoga, CA 95070

Subject: **Response to "Clarification of Technical Issues for 379 Beach Drive"**
379 Beach Drive
APN 043-095-14
\Aptos, California

References:

1) **Geotechnical Investigation, 379 Beach Drive, Aptos, California**, Project No. 1738-SZ70-B44, dated November 30, 2017, prepared by Pacific Crest Engineering, Inc.

2) **Review of the Geotechnical Investigation,** dated March 26, 2018, prepared by Planning Department, County of Santa Cruz, dated March 2018.

3) **Coastal Geologic Investigation, Lands of Vaudagna**, Job #2017011-G-SC, dated February 11, 2018 and prepared by Zinn Geology.

4) **Draft Site Retaining Wall Section,** prepared by RI Engineering

5) **Response to County of Santa Cruz Comments** prepared by Zinn Geology and dated August 16, 2018

6) **Response to Review of Geotechnical Investigation and Supplemental Analysis,** dated August 16, 2018, prepared by Pacific Crest Engineering

7) **Geotechnical/Geologic Peer Review**, dated February 13, 2019, prepared by Alan Kropp and Associates

8) **Geotechnical Investigation for APN 043-095-11, 385 Beach Drive**, Haro Kasunich Associates 2001, Project No. SC6864

9) **Response to 18 July 2019 County of Santa Cruz Comments,** prepared by Zinn Geology and dated August 23, 2019

Dear Mr. and Mrs. Vaudagna,

As requested by you, we have reviewed the County of Santa Cruz's "Clarification of Technical Issues for 379 Beach Drive" letter dated July 18, 2019 regarding the geotechnical and geologic hazards at the subject site. Our response to the technical issues discussed in the County's recent review letter is presented below. Following the County's review of our response and that of the project geologist Mr. Erik Zinn, we request a meeting with County Planning Department be set up if any further discussion is warranted.

1. Slope Stability Results

We are not clear if the reviewer is aware that the seismic slope stability analysis that we submitted in our letter dated August 16, 2018 took into account all potential failure surfaces that could initiate anywhere on the slope, from the top of the bluff and extending all the way to the bottom of the slope. Analysis for potential failure surfaces have included assessing the factor of safety of potential failures that go through or behind the existing County-permitted Geobrugg Tecco stabilization system that has been installed on the property above the Vaudagna's property. **Our analysis shows that all potential failure surfaces have a Factor of Safety greater than the required 1.1 for seismic analysis.** Please see the attached figure which illustrates the factor of safety for a moderate to large scale slide that the County has required us to address. According to our analysis this potentially large-scale slide has a seismic factor of safety against sliding of 1.2.

Based on the geologic report and findings (Reference #3, Zinn Geology), this larger sized, hypothetical slide surface is not consistent with the historical behavior of the bluff at the subject site and it exceeds the minimum required seismic factor of safety of 1.1. Therefore, it was not considered in estimating the volume of debris flow material for mitigation design.

2. Slope Stability Model and Depth of Potential Landsliding

As the reviewer correctly points out, slope stability analysis, and geotechnical engineering in general, is not an exact science. Slope stability results are highly dependent on the geologic subsurface model, strength parameters and design water elevations. If these parameters are carefully selected, then the results of the analysis should be similar to the observed behavior of the slope – e.g. depths and sizes of failures that are known to occur should show a factor of safety roughly equal to 1.0.

Based on the project geologists findings and as stated in our previous 8/16/18 letter, "Geomorphology in the immediate area suggest that typical failures on the bluff face are relatively thin in depth, are limited to relatively small areal extents, and are on the order of 25 feet in length and 10 feet in width. Generally, debris flow failures are comprised of the surficial colluvial soil. In some cases the surficial "rind" of bedrock loses some of its cementation/cohesion due to exposure and weathering and is included as part of the failure mass".

The reports by HKA (Reference 8) cited in the County's latest review letter (7/19/19) also state similar findings: "it appears the primary mechanism of slope failure is shallow, 4 to 10 foot thick, translational (top to bottom) sliding. (Pg 11, Reference 8) and, "We do not anticipate a massive failure of the entire bluff during one seismic event, but rather a series of slides." (Pg 15 Reference 8).

In our opinion the geologic model, strength parameters, water elevation and seismic coefficient used in our analysis are conservative and appropriate for the site-specific conditions. The seismic coefficient used in our analysis is based on more recent studies and technical literature than that used by HKA (Reference 8). Current methods in the geotechnical literature for estimating seismic coefficients incorporate the site specific seismic and soil characteristics including the magnitude and distance of relevant faults and the depth, period and soil characteristics of the hypothetical landslide body. It is an empirical method derived from displacement analysis – an analysis widely agreed in the geotechnical literature as a more accurate means of assessing slope stability.

Alan Kropp, a local geotechnical engineer with over 47 years of experience who specializes in landslides and is recognized for his contributions in earthquake hazard research, performed a peer review of both the geologic and geotechnical reports for this site and concluded that, "the geologic conditions at the site have been satisfactorily characterized" (Reference 7). Furthermore, he states, "we have reviewed a number of these materials as part of our review (referring to current technical literature regarding debris flow methodology). Based on this review, it is our judgment that the methodology utilized by ZG (Zinn Geology) to estimate the debris flow scenarios and volumes are reasonable."

Based on the above we continue to stand behind our analysis as previously submitted for determining the volume of material that could be generated in a debris flow slide and the impact forces it could impose on structures at the base of the slope.

3. Mitigation Measures and Capacity of the System to Accommodate and/or Contain Debris Flows

It is our intent to demonstrate that the mitigation systems proposed have "sufficient excess capacity" to either reduce, retain or accommodate potential debris flows at the site.

Three alternative mitigation systems were previously proposed by Zinn Geology (Reference 5) and are depicted in Plate 2 of that letter. All of these assume construction of a nonhabitable first story with breakaway walls:

a. Scenario One – Design the house so that debris can flow through the bottom story of the house without impacting the habitable 2nd story.

b. Scenario Two – Design impact walls placed at top of retaining walls to contain some portion of the debris and show the subsequent smaller flow volume through the bottom story.

c. Scenario Three – Design Geobrugg debris flow fencing, to slow down and retain a portion of the debris flow and show the reduced debris volume flowing through the bottom story.

As discussed in Reference #9, two additional possible mitigation schemes can be added to the above list for this project:

d. Scenario Four - Reinforce the upper portion of the slope with Geobrugg Tecco Mat effectively reducing the potential volume of material that could be mobilized.

e. Scenario Five - Raise the finished floor elevation of the residence to provide excess capacity for debris flow volumes.

Based on the County's latest response we understand that they would like us to demonstrate that the mitigation scenarios provide sufficient "excess capacity" for the design debris flow volume to flow under the second story of the house and that the debris flow mass is either reduced, contained and/or shown to have sufficient free board to flow through the first nonhabitable story without impacting the habitable portion of the house. We suggest a rationale to demonstrate this as outlined below:

a. Estimate a design debris flow volume based on a seismic Factor of Safety of 1.1 (previously completed, see Reference #6).

b. Reduce, retain or accommodate that flow by one or a combination of the means suggested above (as shown in Plate 2, Reference #5).

c. Demonstrate that mitigation scenarios can decrease the design debris volume, thereby showing increased system capacity.

Points a and b above have previously been addressed in References 5 and 6. These documents demonstrate a design debris flow volume using a slope stability analysis supported by site specific geologic information. This volume of material is shown to flow behind and into the nonhabitable, first story of the proposed residence using break away walls.

Point c is partially addressed by Scenarios 2 and 3, Plate 2 of Reference 5, where Geobrugg fencing or impact walls are shown to retain a portion of the design debris flow and the volume that flows under the house is significantly reduced. These systems are further described in Reference 9 where the design volume flowing under the house is described as being reduced by roughly 25 to 40%, thus increasing the system capacity significantly.

In our opinion the above information demonstrates that implementation of a Geobrugg shallow landslide barrier or an impact wall can reduce the design debris volume that flows through the nonhabitable first story of the house by between 25 and 40 percent. In our opinion, a system or combination of systems that can reduce the design volume that flows under the house by at least 20% can be considered sufficient to demonstrate adequate excess capacity and thereby lower the risk to the habitable portion of the house.

4. Comparison of Project Site to Other Sites on Beach Drive and the Potential for Moderate to Large Scale Landslide Hazards

Response to this item is addressed in Reference #9. As described therein the site-specific geologic investigation provides data that the slope conditions for at least one of the projects cited are very different than those of the current site; most notably the depth of residual colluvial soil that potentially could be mobilized in a debris flow.

In our review of the geotechnical aspects of Reference 8 the Geotechnical Engineers appear to agree with our findings in principle and state that the "primary mechanism of slope failure is shallow, 4 to 10-foot-thick translational sliding". Additionally, they state their seismic coefficient selection is **very** conservative, implying that the results of their seismic analysis is also very conservative. Additionally, in characterizing the potential 20 to 25-foot-deep failure surfaces depicted in their seismic analysis they state "these (massive failures) are not expected to occur in one massive event but rather a series of smaller events that occur over several winter seasons." Based on our review of Reference #8 it does not appear that these reports provide either qualitative or quantitative evidence indicating that moderate to deep seated sliding is a typical slope process.

We appreciate the opportunity to be of service. Please contact us if you have any questions.

Sincerely,

PACIFIC CREST ENGINEERING INC.

Soma B Goresky

SOMA B. GORESKY
GE 2252
EXP. 6-30-2021

Elizabeth Mae Mitchell

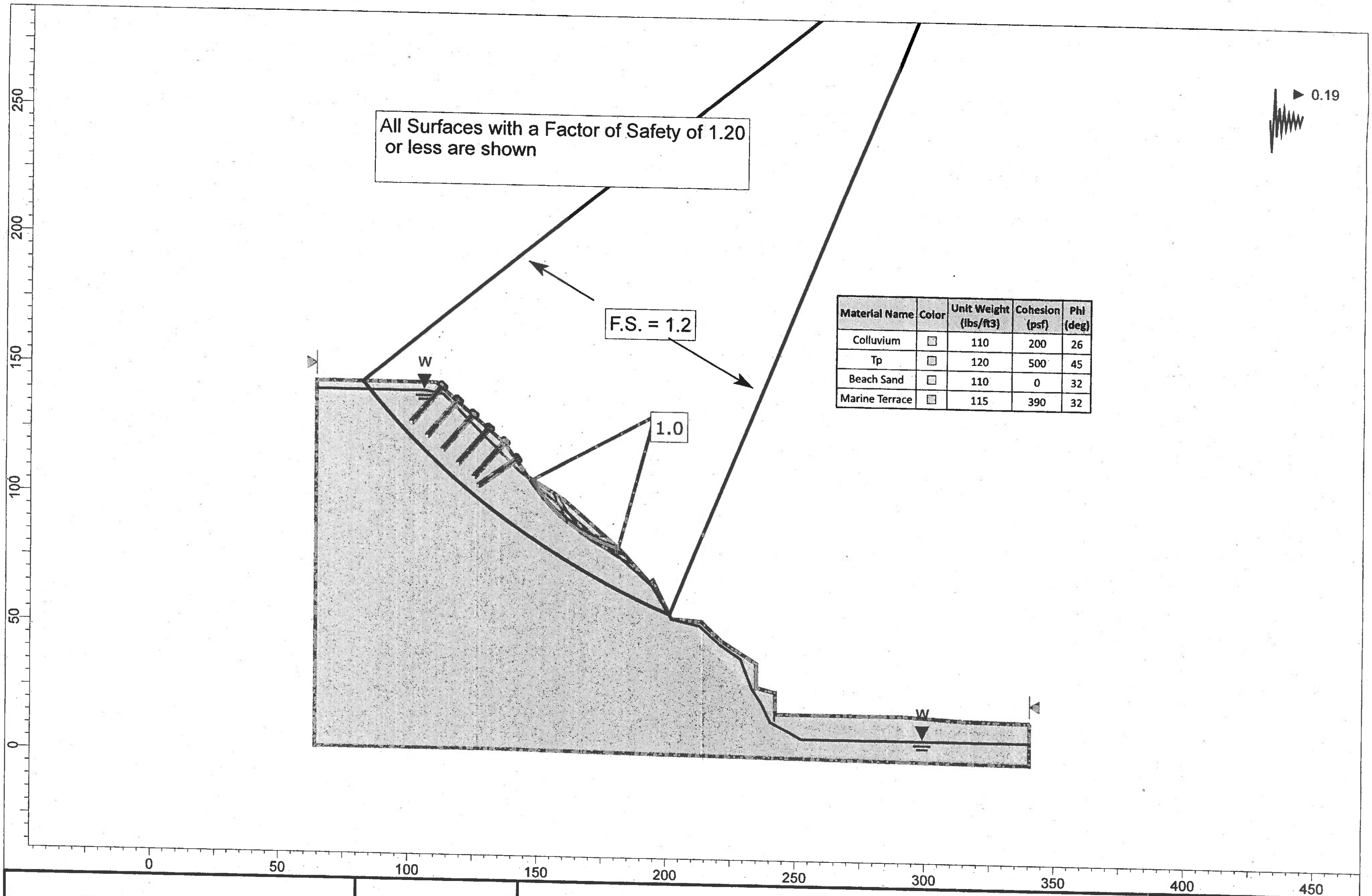
ELIZABETH MAE MITCHELL
GE2718
Exp. 12/31/20

Soma B. Goresky, GE
Associate Engineer
GE 2252

Elizabeth M. Mitchell, GE
President/Principal Geotechnical Engineer
GE 2718

c.c. Matson Britton Architects
Zinn Geology
RI Engineering

Attachments:
Figure 1 – Seismic Analysis



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Colluvium	▨	110	200	26
Tp	▨	120	500	45
Beach Sand	▨	110	0	32
Marine Terrace	▨	115	390	32

Pacific Crest
ENGINEERING INC

Date: 8/26/2019

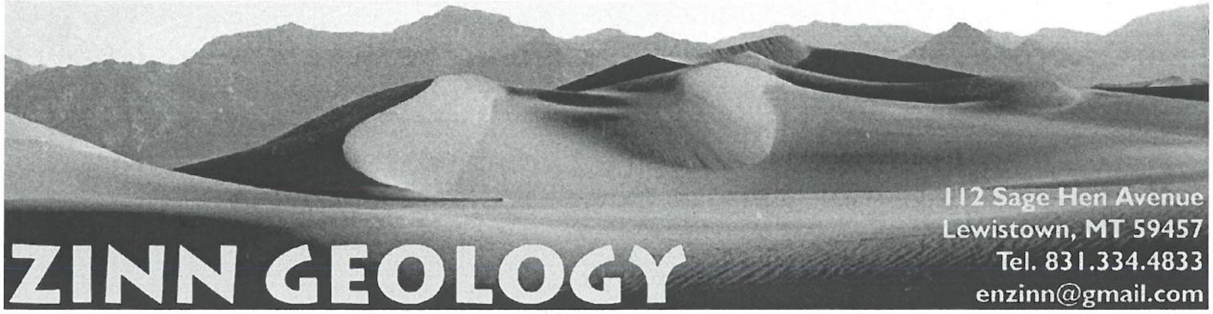
File Name: Vau 6d seism circ .19b.slim

Seismic Analysis
379 Beach Drive
Aptos, California

Figure No: 1

Project No. 1738

EXHIBIT M



ZINN GEOLOGY
112 Sage Hen Avenue
Lewistown, MT 59457
Tel. 831.334.4833
enzinn@gmail.com

23 August 2019 Job #2017011-G-SC

Jim and Sue Vaudagna
19501 Scotland Drive
Saratoga, CA 95070

Re: Response to 18 July 2019 County of Santa Cruz comments
Coastal geologic investigation
379 Beach Drive
Aptos, California
County of Santa Cruz APN 043-095-14

Dear Mr. And Ms. Vaudagna:

This letter summarizes our response to comments and request for supplemental data and analysis given in a letter written by the County of Santa Cruz Planning Department [COSCPD] dated 18 July 2019 (to view a copy of the letter, see the attachment to this letter). Our responses are given in the same sequence as the comments in the County letter and follow the County letter enumeration.

Under the section titled "County Staffs {sic} Response To Requested Comments":

COSCPD Comment #1: No response needed by Zinn Geology. See forthcoming response letter by the Project Geotechnical Engineer of Record, Pacific Crest Engineering [PCE] for their response to this comment.

COSCPD Comment #2: Most of the response to this comment is contained within the forthcoming response letter by PCE. There was some geological input from Zinn Geology to PCE that assisted them with their response, which can be seen below.

We note that the specifics of risk reduction mitigation schemes including selection of the schemes, design and construction are prepared and submitted after the geological and geotechnical engineering parameters have been provided by the client or their agent in a report or supplemental letters to the COSCPD. It is our understanding that our original report and supplemental letters still have not been accepted by the COSCPD. Additionally, it is our understanding, after reading the 18 July 2019 letter by the COSCPD that our original debris flow design parameters are being contested.

EXHIBIT N

We continue to stand by our debris flow design parameters presented in our response letter dated 16 August 2018, for the reasons which are stipulated below. This volume is shown on the attached Plate1, which is an excerpt from our first scenario from our 16 August 2018 letter that depicted the entire volume of debris flow mass breaking through the lower story walls and flowing under the proposed residence.

The COSCPD has requested that the team *"quantify the capacity of the proposed project slope stabilization/soil confinement system and develop an estimate of the Factor of Safety for the project debris mass storage capacity. Provided that sufficient excess capacity exists in the proposed system, we will accept the proposed mitigation."*

We have discussed the geological parameters and debris flow parameters with the design team, including numerous discussions with PCE, the Project Civil Engineer of Record, R.I. Engineering and the Project Architect, Matson Britton Architects. There a number of mitigation schemes that could be implemented, some of which have been rejected by the client or the team for reasons unrelated to the geology of the site. In our opinion, the potential mitigation schemes that can be implemented singularly or in combination include, but are not limited to:

a. Geobrugg Tecco system,
b. Geobrugg Shallow Landslide Barrier,
c. Impact walls mounted above the house and building pad,
d. Elevation of the habitable quarters of the house away from the slope and above the building pad,
e. Design of breakaway walls for the house to allow debris flow to move through the ground floor,
e. Removal of the proposed rear deck to increase the debris flow "flow through" rear height

A complete redesign of the house to that of a "bunker house" has been repeatedly discussed throughout our involvement of the project, but that type of structure has been deemed unacceptable by the owners and the architect.

Discussion Of Mitigation Schemes

Geobrugg Tecco System

A portion of the upper slope that lies above the property (the Meyerhoff property) has already been covered in a designed, permitted and constructed Geobrugg Tecco system, which retains the upper eight feet of the loose soil (see light green shading on attached Plate 1). It is possible that the remaining slope on the subject property could be covered with the Geobrugg Tecco system too, which would eliminate that portion of the slope as contributing to the risk related to debris flow impact to the structure.

There are other permutations to the extent and geometry of the potential coverage of Geobrugg Tecco system, such as "toeing it out" where Purisima Formation bedrock outcrops on the slope (i.e. reducing coverage), or placing it on the adjacent properties to protect the residence from debris flows originating from the neighboring properties and striking the proposed residence obliquely. We understand that some of the possibilities of protecting the residence using this scheme will involve complicated easements and permitting conditions, or in reduced coverage.

If this scheme is pursued, the overall goal would be to reduce the area on the slope capable of generating a debris flow, which would reduce the debris flow volume.

Geobrugg Shallow Landslide Barrier

We have utilized the SHALLSLIDE ONLINE TOOL provided by Geobrugg to calculate the debris flow volume that could be stopped by this special fencing barrier (see attached worksheet for the input and output parameters). Results show that an 11½ foot high fence constructed only on the subject property could potentially stop about 130 cubic yards, reducing the original design volume by about 60%. The area behind the house and under the 2nd floor is more than enough to capture the remaining volume of debris flow deposit that might over top the fence, as shown in our 16 August 2018 letter and Plate 1 attached to this letter.

Placing a fence on the subject property only provides about 2/3 of the coverage needed to stop potential debris flows that will come into the proposed residence obliquely (see attached Plate 1). The coverage could potentially be extended onto the neighboring properties and give complete protection to the proposed residence. We understand that the same easement and permitting issues might arise from attempting to place the fence on the neighboring properties, so attempting to do that may not be feasible.

Impact Walls Above The House And Building Pad

Impact wall(s) can be constructed on the slope above the proposed residence. The design premise will be to use roughly similar parameters to those appended to this letter to design a wall that would stop a significant portion of the debris flow volume that would otherwise flow under the house.

The issue of a wall spanning the subject property providing partial coverage is identical to that discussed for the Geobrugg Shallow Landslide Barrier, as well as the issue of placing the wall on the neighboring properties.

Elevation Of Habitable Quarters Away From Slope And Above The Building Pad

It is our understanding the ground floor story of the proposed residence cannot be habitable due to wave run up flooding issues. Raising the upper story would increase the volume of material that can pass through and under the house, provided that exterior walls are adequately designed and constructed to break away and whatever foundation elements are used can resist the forces of the debris flow striking them (albeit at a reduced speed from the original up slope impact velocity). Removing structural elements that hang off of the back of the residence, such as decks, will help increase the height any debris flows approaching the rear of the residence. The volume of the initial rear catchment area on grade directly behind the house can be increased by increasing distance between the back of the residence and the rear retaining wall. This could be achieved by sliding the house forward or excavating deeper into the hillside, which in turn would result in a taller retaining wall. Catchment areas could also be increased by creating smaller excavated basins with retaining walls stepping up the slope.

In our 16 August 2018 letter, we attempted to construct idealized debris flow sections using different mitigation concepts. The first concept involved only lifting the 2nd floor of the residence (as required by FEMA for wave run up risks). We plotted the configuration of the quantitatively derived design debris flow (by PCE) on a cross section that represented the modified proposed civil engineering configuration of the back slope and retaining walls, as well as the most recent version (at that time) of the house from the Matson-Britton Architect plans. We plotted three different mitigation scenarios on that base section and depicted our interpretation of the design debris flow cross section at the moment of impact and the configuration of the debris flow after it comes to rest. The first concept involved only lifting the 2nd floor of the residence (as required by FEMA for wave run up risks) and only provided an estimated 6 inches of head space between the top of the debris flow and the bottom of the finished floor.

We estimate that if the bottom of the finished floor is raised approximately 10 more inches, a debris flow 20% greater than the one shown on Plate 1 could be accommodated with no further mitigation.

Breakaway Walls For The House

As discussed above, designing and constructing walls to breakaway upon impact by debris flows or waves will allow debris flows to pass through and under the house. Most of the debris flows we have observed over the years along the Beach Drive area haven't run out much further than 50 to 60 feet once they hit the flat grade that lies at the toe of the bluff.

Removal Of The Proposed Rear Deck

We have considered the removal of the deck in multiple discussions with the design team, but doing so will not buy much more height and volume of flow passing through that gap between the deck and the slope or retaining wall (depending upon the final design). If extra volume accommodation is required beyond the design debris flow, then the limiting factor is the height of the bottom of the 2nd story finished floor. This could be raised to accommodate a greater volume of debris, which in turn would also increase the gap between the rear deck and back slope.

Further Discussion Of Mitigation Schemes

We have discussed several mitigation schemes above. The most effect mitigation scheme appears to be the Geobrugg Shallow Landslide Barrier fence system. If it is erected across only the subject property, it will leave an unprotected area 6 feet wide to the northwest and 12 feet southeast of the property, assuming the fence is constructed right above the retaining wall as currently depicted on the civil engineering plans. This corresponds to a reduction of the total width of the design debris flow at impact of 36 feet, because a portion of the debris flows will either strike the fence or the neighboring residences. The volume of debris that could flow beyond the ends of the barrier erected only on the subject property are 38 cubic yards (from the northwest) and 77 cubic yards (from the southeast). Furthermore, if a debris flow moves past the fence to the southeast, it will have to run out across a flat patio on neighboring property over a distance between 16 and 56 feet before striking the proposed residence on the subject property. Looking at that pathway to the southeast even closer, we have noted that the northeast corner of the proposed residence is actually protected by a retaining wall on the neighboring property. Any debris flows slipping past the fence to the southeast would have to slide across the neighboring patio, with a portion striking and presumably coming to rest on the neighboring property against the neighbor's retaining wall. This would result in a reduction of the estimated 77 cubic yards that might slip by the southeastern end of the fence, if the Geobrugg Shallow Landslide Barrier is only erected on the subject property.

Even if one were to assume that entire volumes of 38 or 77 cubic yards slides past the ends of the barrier, it would still result in reduced volumes of about 50% and 27% respectively. Actual volume reduction percentages will likely be higher because the debris flows will strike the neighbor's properties first, stripping some percentage of material from the debris flow before it strikes the Vaudagna residence.

COSCPD Comment Regarding "Moderate To Large Landslides"

This comment requires input by both the Project Geotechnical Engineer and the Project Geologist.

It is our understanding that PCE has extended their quantitative slope stability analysis for deeper landsliding and the net outcome supported the original premise that the largest landslide with a Factor of Safety of 1.1 or less is about 52 feet long and seven feet deep.

This fits the geological history of the subject property and the slope on the neighboring property above. The geological setting for the subject property is as follows:

1. The slope above the subject property is planar to broadly concave. We have not observed hollows or basins filled with loose soil, nor have we observed any evidence of discarded fill or repeated cast off of landscaping debris off of the bluff.

2. The slope above subject property has not generated large, deep seated landslides since at least 1928. Furthermore, if the slope had failed that deeply, such as in the 1906 Earthquake, we would have presumably observed a scar or escarpment that created a very deep bowl into the slope. We have not observed that evidence of scarring.

3. The slope above the property is both naturally and artificially well drained. A large portion of the bluff on the Meyerhoff property is actually several isolated peninsulas, flanked by small arroyos that drain to the east. This has the effect of draining the portion of the bluff above the Vaudagna property. The Meyerhoff property also has an established engineered drainage system that captures the storm water drainage coming from the hardened surfaces on the property. A very large portion of the drainage above the bluff is captured and directed by the engineered system to the east where it subsequently enters the south-flowing arroyo. The net effect is that the contributing area of storm water runoff and groundwater behind the top of the bluff is drained and very little water falling on the terrace above the Vaudagna percolates into the ground - it is carried to the east to the south flowing arroyo instead.

4. A portion of the bluff above the subject property is protected with a Geobrugg Tecco system that retains the loose surficial soil. This reduces the area that contributes to the debris flow hazard and the subsequent impact risk to the residence. It is important to note that valuable subsurface data was also gleaned from observation by PCE of the drilling of the soil nails on the bluff face.

We have reviewed the documents cited in the COSCPD 18 July 2019 letter and noted that none of those sites had conditions similar to those cited above, at least not that we can tell. One of the reports cited was a former Zinn Geology report and we can definitively say that the geological conditions for that site are not similar to the conditions observed on the Vaudagna property. With all due respect to the authors of the 18 July 2019 letter, it is impossible to bring our level of understanding of the slope above the Vaudagna property to the other projects they have cited without doing a site specific investigation of those properties. As noted above, the other properties cited by the COSCPD as exemplars for "global slope stability" do not appear to have the same physical conditions as the Vaudagna property. Alluding to vague geological and geographical similarities is not the same as performing an in-depth case study for each cited parcel, which would involve detailed mapping and detailed subsurface work that equals or parallels the work performed on this project by PCE and Zinn Geology for both the up slope property (the Meyerhoff property) and the subject property (the Vaudagna property). We have worked in tandem with PCE to develop qualitative and quantitative models specific to the slope conditions above the subject property. This level of investigation supercedes generalizations of slope characteristics and findings from past studies at any other locations along Beach Drive.

Sincerely,
ZINN GEOLOGY

Erik N. Zinn
Principal Geologist
P.G. #6854, C.E.G. #2139

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PROFESSIONAL GEOLOGIST
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Attachments: County of Santa Cruz letter dated 18 July 2019
Excerpt from Plate 1 from Zinn Geology letter dated 16 August 2018

References Cited

Alan Kropp and Associates, 13 February 2019, Geotechnical/Geologic Peer Review, unpublished consultant letter.

County of Santa Cruz Planning Department, 26 March 2018, Review of the Geotechnical Investigation, unpublished agency letter.

Haro, Kasunich & Associates, 2001, Geotechnical Investigation for APN 043-095-11 - 385 Beach Drive, Project No. SC6864, unpublished consultant report.

Pacific Crest Engineering, Inc., 16 August 2018, Response to Review of Getochnical Investigation and Supplemental Analysis, unpublished consultant letter.

Pacific Crest Engineering, Inc., 30 November 2017, Geotechnical Investigation, 379 Beach Drive, Aptos, California, Project No. 1738-SZ70-B44, unpublished consultant report.

R.I. Engineering, March 2018, New residence for Jim & Sue Vaudagna - 379 Beach Drive - Aptos, California - APN 043-095-14, Sheet C-2, unpublished consultant plans.

Zinn Geology, 11 February 2018, Coastal Geologic Investigation, Lands of Vaudagna, Job #2017011-G-SC, unpublished consultant report.

Zinn Geology, 16 August 2018, Response to County of Santa Cruz Comments, unpublished consultant letter.

COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT
701 OCEAN STREET, 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123
KATHLEEN MOLLOY, PLANNING DIRECTOR

July 18, 2019

DiBenedetto & Lapcevic, LLP
Attn: Anna DiBenedetto
1101 Pacific Avenue, Ste. 320
Santa Cruz, California 95060

Subject: Clarification of Technical Issues for 379 Beach Drive

Project Site: 379 Beach Drive
APN 043-095-1
Application No. REV181023

Dear Ms. DiBenedetto:

This letter outlines County staff's responses to your request for clarification of technical comments presented in your email to Carolyn Burke on 18 July 2019. This letter also presents our concerns regarding the Moderate to Large Scale Landslide Hazards at the project site. Our concerns on this matter were previously transmitted informally in an email to Erik Zinn on May 8, 2019. We have raised this issue because we consider such landsliding to present a real life-safety hazard to the proposed development.

We were requested "to be specific about which technical issues they [we] are disputing regarding the slope stability model and analysis as presented in Pacific Crest's 8/16/18 response letter to County Comment #1 (please see letters prepared by both Pacific Crest and Zinn Geology, and specifically refer to the last paragraph of Comment #1 in the PCE letter." Our comments in this letter also address the 25 April 2019 email from Elizabeth Mitchell to Rick Parks to clarify the PCE 16 August 2018 response letter.

County Staffs Response to Requested Comments

1. From the 25 April 2019 email from Elizabeth Mitchell to Rick Parks:
"Please note – in our opinion applying a factor of safety of 1.5 for static and 1.1 for seismic to determine debris flow volume is not applicable to these analyses. For calculating a debris flow volume for design one should use the volume that is predicted to fail. Additional safety factors are then considered in the design of mitigation measures/structures to contain or redirect that flow."

County staff agrees a FS = 1.0 for a limit equilibrium type slope stability analysis represents the mathematical point below which the slope could fail. The concern is the potential difference between the idealized/simplified soil slope model and the real slope. As well, once the slope fails, the remaining slope will have reduced stability, leading to additional debris pulses.

The approximate 120-foot high slope above the proposed residence has been modeled as three soil types with an associated soil shear strength and unit weight for each soil type. A FS = 1.0 for

EXHIBIT N

a simplified soil slope model does not account for: uncertainties and variabilities of the soil profile both vertically and laterally; reliability of input parameters; and the limitations of analyses methods. Determination of an appropriate magnitude for a factor of safety should also include the consequences of slope failure, and when applicable, the unacceptable performance of structural elements.

The horizontal seismic coefficient is a primary input parameter for pseudo-static analyses. The selection of the seismic coefficient is not currently codified. We have noted a wide range of seismic coefficients utilized along Beach Drive in reports submitted to the County for review by a variety of consultants. The project horizontal seismic coefficient of 0.16 is at the lower end of seismic coefficients utilized by consultants along Beach Drive and minimizes the magnitude of slope failure.

The landslide debris mass volume is a primary component for the design of the blufftoe residence. Typically, the landslide debris mass from a slope failure with a FS = 1.1 is a larger volume than a slope failure with a FS = 1.0. The 25 April 2019 email from Elizabeth Mitchell to Rick Parks also states, "Additional safety factors are then considered in the design of mitigation measures/structures to contain or redirect that flow." We maintain our requirement for a minimum pseudo-static Factor of Safety of at least 1.1 for slope stability analyses.

2. We were also requested to specifically refer to the last paragraph of Comment #1 in the PCE letter dated 16 August 2018. The specified last paragraph:
   "In our opinion both Scenario 2 and 3 provide adequate means for reducing the risk that debris will impact the habitable portion of the house. Preliminary design for impact walls should be based on an impact loading of 1900 psf. Preliminary design of debris flow fences should be based on the parameters presented in Plate 2, Reference 5. Design of all impact structures should include "wing walls" that confine the debris to the site and prevent it from being deflected onto the adjacent properties. We request the opportunity to review proposed designs for debris fences or impact walls and to provide additional geotechnical design recommendations as needed."

County staff's understanding of the proposed project slope stabilization/soil confinement system to accommodate the design of the elevated blufftoe residence is as follows:

- The existing blufftop Tecco steel mesh anchored with either helix screw anchors or grouted soils nails (conflicting anchor types/lengths are referenced in the project documents);
- A mid-bluff face Geobrugg steel debris net system;
- Blufftoe retaining walls with above grade debris impact walls to contain slope debris; and
- Siting the blufftoe residence above and seaward of the design landslide debris mass.

We agree with the consultants that the above outlined system or some similar variation can protect the occupants of the proposed blufftoe structure from surficial debris flows. A caveat would be the slope stabilization/soil confinement system will need to be monitored and maintained for the design life of the proposed residence to function as designed. We anticipate the bluff top Tecco steel mesh and the mid-slope Geo-brugg steel net will need to be replaced at some time in the future due to corrosion. Both the mid-slope Geo-brugg steel net and the blufftoe impact walls will need to be cleared of accumulated debris to re-establish design debris mass capacity.

We also agree there may be enough redundancy in the proposed project slope stabilization/soil confinement system to accommodate a larger volume of surficial slide mass. Please quantify the capacity of the proposed project slope stabilization/soil confinement system and develop an

EXHIBIT N

estimate of the Factor of Safety for the project debris mass storage capacity. Provided that sufficient excess capacity exists in the proposed system, we will accept the proposed mitigation.

Moderate to Large Scale Landslide Hazards

For the purposes of this discussion, we consider moderate to large scale landsliding to include landslides in excess of about 15 feet deep originating at or near the crest of the coastal bluff, of either translational or rotational mechanism. The potential for such landslides to occur at the project site is suggested by several lines of evidence:

1. There was a large landslide that destroyed two homes at 337 and 339 beach drive in 1982, a short way up coast from the project site. A photograph of the landslides and damaged houses was provided with the May 8, 2019 email. From the photo, it appears that the landslide was not highly fluid—there appears to be a slump type landslide mass or "sand flow" on the slope behind the houses and there is no evidence for a liquefied mass having flowed out into the street. Comparison of pre-landslide topographic mapping prepared by Santa Cruz County (Towill, Inc.,1965; 1"=100' scale) and topographic contours produced from Lidar coverage of the County (AMBAG, 2009) indicates that the landslide mass at origination was about 20' thick, when adjusted for the change in vertical datum. We do not consider this evidence definitive, but strongly indicative of moderate to large scale landsliding, as distinct from debris flows.

2. We noted evidence for three large scale landslides that had formed in the coastal bluff up coast from the project in 1928 aerial photos (frames 20-25, flight 1928H, UCSC aerial photo collection.

3. A number of geotechnical studies have identified the potential for moderate to large scale landsliding through slope stability analyses (quantitative or qualitative). We list three relevant studies here:

a. Haro, Kasunich, and Associates, 2001, Geotechnical Investigation for APN 043-095-11, 385 Beach Drive;
b. Haro, Kasunich, and Associates, 2002, Geotechnical Investigation for APNB 043-095-12, 383 Beach Drive;
c. Zinn Geology, 2016, Focused Geologic Investigation of Debris Flow Hazards for Existing Residence and Proposed Deck,615 Beach Drive, APN 043-152-28.

All three of these reports identify a potential for a 20' thick wedge or slab of sand to fail from the bluff above the homes in a non-fluid or only partially fluid state. The Haro, Kasunich, and Associates reports are for new houses located two and three doors downcoast from 379 Beach Drive.

There have been a large number of slope stability analyses performed for projects along Beach Drive. Not all of the analyses identify a potential for moderate to large scale landslides. In some cases, the critical depths are on the order of 6 to 8 feet. We are not taking the position that the observational evidence and analytical studies cited above prove that there is a moderate to large scale landslide hazard at the subject property, but that sufficient concern exists that a thorough analysis of the hazards associated with such an event must be performed.

Although the previous landsliding of the bluff identified in photos is located hundreds or thousands of feet from the subject property, the geologic and geomorphic conditions at the different landslide locations and the present site are similar. The geologic susceptibility to landsliding at the subject site must also be considered similar, until proven otherwise.

The reason for evaluating the potential for medium to large scale landsliding at this site is that such landsliding will undercut the 14' to 16' deep tecco mesh slope stabilization scheme located above the proposed residence. Consequently, the teccomesh system it will provide little or no protection for the project in the event of a moderate to large scale landslide event. The geologic report by Zinn Geology and the geotechnical report by Pacific Crest Engineering do not address the potential for moderate to large scale landsliding, nor do any of the response to comment letters provided by the project consultants.

We have reviewed geotechnical reports for all new homes built on landward side of Beach Drive in about the last 20 years. Every project we have looked at has included a global analytical slope stability analysis of the bluff behind the proposed home. Our review indicates that there is a well established local standard of practice for geotechnical analysis of new homes. We require a properly constituted slope stability analysis of the entire bluff face for this project. The analysis should correctly model the teccomesh system as installed. It is our understanding that the system consists of screw anchors embedded about 14' deep. Should a landslide risk be identified that is not adequately mitigated by the existing teccomesh system and the currently proposed debris flow mitigation scheme, new mitigation measures must be implemented.

We are willing communicate or meet with you, formally or informally, to help expedite this analysis. Please contact Rick Parks at (831) 454-3168/email: Rick.Parks@santacruzcounty.us or Jeff Nolan at (831) 454-3175/Jeffrey.Nolan@santacruzcounty.us if we can be of any further assistance.

Sincerely,

RICK L. PARKS
No. 2603
Exp. 7-15-19

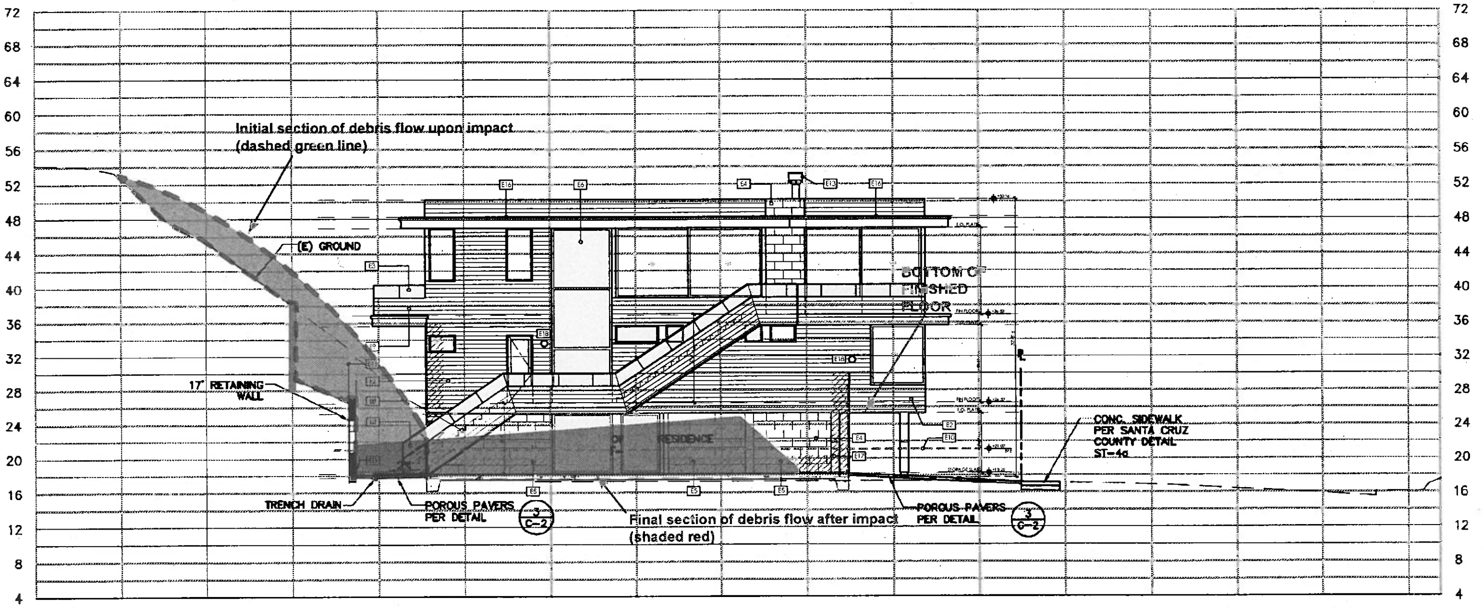
Rick Parks, GE 2603
Civil Engineer
Planning Department
County of Santa Cruz

JEFFREY M. NOLAN
2247

Jeffrey Nolan, CEG 2247
County Geologist
Planning Department
County of Santa Cruz

Cc: Pacific Crest Engineering, Inc. Attn: Soma Goresky, GE
Zinn Geology, Attn: Eric Zinn, CEG

SCENARIO ONE
DEBRIS FLOW WITH NO IMPACT OR CATCHMENT STRUCTURES



SECTION A–A

SCALE: 1"=10' HORIZONTAL VERTICAL

PARAMETERS TAKEN FROM GEOBRUGG SHALLSLIDE ONLINE TOOL AND PACIFIC CREST ENGINEERING SLOPE STABILITY ANALYSIS

Width of starting volume = 9 meters (29.5 feet)
Total starting volume of shallow landslide = 176 cubic meters (230 cubic yards)
Density of shallow landslide materail = 2100 kilograms per cubic meter (131 pounds per cubic foot)
Distance from breakout zone to barrier location = 10 meters (33 feet)
Spreading angle of starting volume = 5 degrees
Additional width at impact section on each side of starting width = 1 meter (3.3 feet)
Width of shallow landslide at impact point = 11 meters (36 feet)
Maximum speed of shallow landslide at impact section = 7.6 meters per second (25 feet per second)
Travel time of landslide front between starting zone and impact section = 1.32 seconds
Peak discharge = 134 cubic meters per second (175 cubic yards per second)
Flow height = 1.63 meters (5.3 feet)
System height of the filled barrier = 2.63 meters (8.6 feet)
Slope angle = 70%
Inclination of retained material behind the barrier = 27%
Retention volume per linear meter = 9.53 cubic meters per meter (3.8 cubic yards per foot)
Total retention volume = 102 cubic meters (133 cubic yards) (assumes complete coverage of property width of 35' at retaining wall)
Overflow = 74 cubic meters (97 cubic yards)

BASE SECTION: Base section used was constructed using digital excerpts from Section A-A' from R.I. Engineering Sheet C-2, dated March 2018 and the house section was digitally excerpted from the West Elevation from Matson-Britton Architects Sheet A3.1 dated 11 October 2017.

CERTIFIED ENGINEERING GEOLOGIST
ERIK N. ZINN
No. 2139
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
ERIK N. ZINN
No. 6854
STATE OF CALIFORNIA

ZINN GEOLOGY

DEBRIS FLOW OUTCOME ASSUMING NO IMPACT BARRIERS OR WALLS

Lands of Vaudagna
379 Beach Drive
Aptos, California

Date: 23 August 2018
Revised:
Job #2017011-G-SC
Scale: 1"=10', h=v
Drawn by: ENZ

Plate 1

EXHIBIT N

From:	Carolyn Burke
Sent:	Tuesday, October 29, 2019 9:55 AM
To:	Anna DiBenedetto
Cc:	Nathan MacBeth; Jeff Nolan; Rick Parks
Subject:	RE: 379 Beach Drive - Additional Information Request

Hi Anna,

I would like to follow up with you to confirm whether you intend to submit a response to our request for additional information sent to you via email on October 8. Please reply with your estimated submittal date at your earliest convenience.

Thank you,

Carolyn Burke
Senior Civil Engineer
County of Santa Cruz – Environmental Planning
(831) 454-5121
Carolyn.Burke@santacruzcounty.us

From: Rick Parks <Rick.Parks@santacruzcounty.us>
Sent: Tuesday, October 8, 2019 3:27 PM
To: Anna DiBenedetto <anna@dl-lawllp.com>
Cc: Jeff Nolan <Jeff.Nolan@santacruzcounty.us>; Jessica deGrassi <Jessica.deGrassi@santacruzcounty.us>; Nathan MacBeth <Nathan.MacBeth@santacruzcounty.us>; Carolyn Burke <Carolyn.Burke@santacruzcounty.us>; Soma Goresky <soma@pacengineering.net>; Erik Zinn <enzinn@gmail.com>
Subject: RE: 379 Beach Drive - Additional Information Request

Ms. DiBenedetto,

Our review of the global slope stability analysis for 379 Beach Drive is attached.
Hard copies will be mailed to you as well as to Pacific Crest Engineering and Zinn Geology.

Thank you,

Rick Parks, GE2603
Environmental Planning Section – Civil Engineer
County of Santa Cruz Planning Department
701 Ocean Street, 4th Floor
Santa Cruz, CA 95060
831-454-3168

From: Carolyn Burke <Carolyn.Burke@santacruzcounty.us>
Sent: Monday, October 7, 2019 11:19 AM
To: Anna DiBenedetto <anna@dl-lawllp.com>; Rick Parks <Rick.Parks@santacruzcounty.us>

EXHIBIT O

Cc: Jeff Nolan <Jeff.Nolan@santacruzcounty.us>; Jessica deGrassi <Jessica.deGrassi@santacruzcounty.us>; Nathan MacBeth <Nathan.MacBeth@santacruzcounty.us>
Subject: RE: 379 Beach Drive - Additional Information Request

Hi Anna,

Thank you for your email; the information is currently under review, and we will likely have a response letter issued this week.

Please feel free to contact me with any questions or concerns.

Sincerely,

Carolyn Burke
Senior Civil Engineer
County of Santa Cruz – Environmental Planning
(831) 454-5121
Carolyn.Burke@santacruzcounty.us

From: Anna DiBenedetto <anna@dl-lawllp.com>
Sent: Monday, October 7, 2019 8:58 AM
To: Carolyn Burke <Carolyn.Burke@santacruzcounty.us>; Rick Parks <Rick.Parks@santacruzcounty.us>
Cc: Jeff Nolan <Jeff.Nolan@santacruzcounty.us>; Jessica deGrassi <Jessica.deGrassi@santacruzcounty.us>; Nathan MacBeth <Nathan.MacBeth@santacruzcounty.us>
Subject: RE: 379 Beach Drive - Additional Information Request

Carolyn, do you know whether the additional information has been accepted and satisfies the County's requirements/concerns?

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From: Carolyn Burke <Carolyn.Burke@santacruzcounty.us>
Sent: Monday, September 23, 2019 12:46 PM

EXHIBIT O

To: Rick Parks <Rick.Parks@santacruzcounty.us>; Anna DiBenedetto <anna@dl-lawllp.com>
Cc: Jeff Nolan <Jeff.Nolan@santacruzcounty.us>; Jessica deGrassi <Jessica.deGrassi@santacruzcounty.us>
Subject: RE: 379 Beach Drive - Additional Information Request

Hi Anna,

Please confirm that you have received our request for the slope stability model data output sheets (see below). Do you have an estimated time frame for submittal of this documentation?

Thank you,

Carolyn Burke
Senior Civil Engineer
County of Santa Cruz – Environmental Planning
(831) 454-5121
Carolyn.Burke@santacruzcounty.us

From: Rick Parks <Rick.Parks@santacruzcounty.us>
Sent: Wednesday, September 11, 2019 11:21 AM
To: Anna DiBenedetto <anna@dl-lawllp.com>
Cc: Carolyn Burke <Carolyn.Burke@santacruzcounty.us>; Jeff Nolan <Jeff.Nolan@santacruzcounty.us>; Jessica deGrassi <Jessica.deGrassi@santacruzcounty.us>
Subject: 379 Beach Drive - Additional Information Request

Hello Anna,

Attached is our request for the slope stability model data output sheets utilized by Pacific Crest Engineering and Zinn Geology to determine the global stability of the bluff above 379 Beach Drive.

Thank you,

Rick Parks, GE2603
Environmental Planning Section – Civil Engineer
County of Santa Cruz Planning Department
701 Ocean Street, 4th Floor
Santa Cruz, CA 95060
831-454-3168

EXHIBIT O

COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT
701 OCEAN STREET, 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123
KATHLEEN MOLLOY, PLANNING DIRECTOR

7 October 2019

DiBenedetto & Lapcevic, LLP
Attn: Anna DiBenedetto
1101 Pacific Avenue, Ste. 320
Santa Cruz, California 95060

Subject: Review of Global Slope Stability Analysis Output Data

Project Site: 379 Beach Drive
APN 043-095-14
Application No. REV181023

Dear Ms. DiBenedetto:

We have not accepted the 379 Beach Drive global slope stability analysis for the following reasons:

The submitted Slide Analysis Information sheets for 379 Beach Drive dated 5/14/2018 at 5:06:30pm from Pacific Crest Engineering, Inc. references grouted tiebacks with an out of plane spacing of 8 feet as well as the grouted tieback tensile capacity, bond strength, and bond length. The associated Slope Stability Output Schematic shows a vertical row of six grouted tiebacks approximately 18 feet long at the top of the bluff face.

Our understanding of slope stability analysis modeling is the presence of the grouted tiebacks or grouted soil nails increases the global stability of the bluff face by reinforcing the blufftop soils.

Based upon our review of construction documentation in the County project files, grouted tiebacks or grouted soils nails were not used at 340 Kingsbury Drive in either the December 2012 - Emergency Bluff Repair or the September through November 2014 - Installation of the Tecco Slope Protection System.

As stated in the 340 Kingsbury Drive letter Emergency Bluff Repair dated 11 January 2013 by Pacific Crest Engineering, Inc., "a total of thirty-two CS 2⅜" Viking Helical Anchors were installed from December 27 through 31 December 2012. Each anchor had a lead section comprised of eight-inch, ten-inch and twelve-inch anchor plates. The anchors were installed to depths ranging from 14 to 16 feet, as measured from the face of the slope."

EXHIBIT P

As outlined in the 340 Kingsbury Drive letter Construction Summary and Final Site Observations dated 29 September 2016, by Pacific Crest Engineering, Inc., “A total of 129 Chance Series SS anchors were installed between September 24 and November 3, 2014. Each anchor had a lead section comprised of six and eight-inch anchor plates. The anchors were installed to depths ranging from 10 to 18 feet, as measured from the face of the slope.” The two referenced construction observation letters from Pacific Crest Engineering, Inc. are attached to this document.

The installed helix anchors are not equivalent to grouted soil nails or tiebacks for stabilization/reinforcement of slopes. Also, a portion of the as-built installed anchor lengths are 12% to 45% shorter than the soil nail length utilized in the submitted 379 Beach Drive global slope stability analysis reviewed by County staff.

Please request the project geotechnical engineers submit a global bluff face slope stability analysis for review which does not incorporate blufftop soil reinforcement from grouted tiebacks or grouted soil nails.

Please contact Rick Parks at (831) 454-3168/email: Rick.Parks@santacruzcounty.us or Jeff Nolan at (831) 454-3175/email: Jeffrey.Nolan@santacruzcounty.us if we can be of any further assistance.

Sincerely,

Dick

RICK L. PARKS
No. 2603
Exp. 10/8/2019

JEFFREY M. NOLAN
2247

10/8/19

Rick Parks, GE 2603
Civil Engineer
Planning Department
County of Santa Cruz

Jeffrey Nolan, CEG 2247
County Geologist
Planning Department
County of Santa Cruz

Cc: Pacific Crest Engineering, Inc. Attn: Soma Goresky, GE
Zinn Geology, Attn: Eric Zinn, CEG

Attachments: Emergency Bluff Repair dated 11 January 2013
Construction Summary and Final Site Observations dated 29 September 2016

ic Crest Engineering Inc.

www.4pacific-crest.com

444 Airport Blvd, Suite 106
Watsonville, CA 95076
Phone: 831-722-9446
Fax: 831-722-9158

Project No. 1256.2-SZ70-C47

September 29, 2016

Jens and Suzanne Meyerhoff
14539 East Edgewater Court
Fountain Hills, AZ 85268

Subject: **Construction Summary and Final Site Observations**
340 Kingsbury Drive
APN 04-094-06
Aptos, California

References: **Pacific Crest Engineering, Inc.**
Geotechnical Investigation For Meyerhoff Residence
340 Kingsbury Drive
Project No. 1256-SZ70-C47, dated April 16, 2013

Addendum to Geotechnical Investigation Report
Revised March 19, 2014

Dear Mr. and Mrs. Meyerhoff,

As requested, our firm has been performing geotechnical engineering observation and testing services during earthwork activities in conjunction with construction of the single family residence and associated improvements. Representatives from our firm have been present at the site on an intermittent basis since September 2014. Subject areas observed and/or tested by our firm include the following:

- Observation of the pier excavations for the system of buried pin piles constructed along the top of the arroyo adjacent to the north, east and southeast perimeter of the residence, and along the outboard edge of a portion of the entrance driveway. A total of 41 pin piles were drilled between August 28th and September 16th, 2014. A representative from our firm was present on a continuous basis during pier drilling to confirm that the pin pile excavations were of adequate depth and diameters in accordance with the project plans and our geotechnical recommendations.
- Observation during installation and testing of the soil nails for the Tecco slope protection system. A total of 129 Chance Series SS anchors were installed between September 24th and November 3rd, 2014. Each anchor had a lead section comprised of six and eight-inch anchor plates. The anchors were installed to depths ranging from 10 to 18 feet, as measured from the face of the slope. All anchors were installed to minimum capacities ranging from approximately 9 to 55 kips, as determined by correlating installation torque to capacity. *Correlation data was provided by Sunstone Construction. Anchors that did not meet the* required minimum installation depth of 18 feet, or those demonstrating an axial capacity of less than 10 kips were proof tested by Sunstone Construction to 125% of design capacity.

EXHIBIT P

A total of 16 soil nails were tested by the contractor in the presence of Pacific Crest Engineering. Based on the data collected during testing all tested anchors met acceptance criteria and were found to be in accordance with project specifications. Our site observations were limited to installation and testing of the soil nails as required by the project plans; we did not observe the placement and securing of the mesh or installation of erosion control provisions.

- On December 10, 2014 Pacific Crest Engineering performed compaction testing on backfill for the upper keyway trench. The tests indicate minimum compaction values ranging from 87.2% to 94.1% were achieved in the 5 areas tested. Based on our test results, it is our opinion that the trench backfill was adequately compacted in accordance with the project plans and our geotechnical recommendations.

- Observation and testing of the engineered fill for the residence building pad (Earthwork Observation and Testing Report #1, dated November 24, 2014).
- Compression testing of structural concrete for drilled piers, slabs and footings (Results of Concrete Compression Testing, dated October 23, 2014 and February 4, 2015).
- Observation of the footing excavations for the residence (Footing Observation letter dated December 23, 2014).
- Observation of footing excavations for the retaining wall extension (Footing Observation Letter – Retaining Wall Extension, dated October 5, 2015).
- Compaction testing of driveway soil subgrade on June 7, 2016. Compaction test results indicate relative compaction values of 96.5% and 97.5% at finish subgrade. Our test results indicate that adequate compaction was achieved in the areas tested. Our firm did not observe or test the aggregate base section of the new driveway. It is our understanding that the testing was performed by the contractor's testing firm.
- Final observation of general surface drainage facilities. A representative from our firm visited the site on September 19th and 28th, 2016 to observe the residential development and associated improvements with respect to grading and drainage and completion of the project. We noted that grades around the residence were observed to be generally sloping away from foundation elements. Sheet flow within the coastal bluff setback is directed away from the top of the bluff toward area drains along the south side of the residence. The downspouts and area drains are channeled to closed conduits which carry flow away from the development to a level spreader located along the north side of the property.

To the best of our knowledge, based on our site observations and test results, it is our professional opinion that the project has been completed in general accordance with the recommendations of the referenced geotechnical reports.

Our firm did not observe repair and/or maintenance activities recently performed in conjunction with the section of failed coastal bluff at the southeast property boundary. It is our understanding that this work was done under the observation of the contractor's Geotechnical Engineer, who will be providing a summary and acceptance of that work.

Erosion of the coastal bluff and the arroyo slopes surrounding your property is a continuous process, and especially exacerbated in areas where slopes have become over steepened and/or subjected to concentrated surface runoff. You have made a significant effort to control erosion of the slopes around your property. Proper drainage control, diligent maintenance and timely repair will be required in order to reduce the potential for continued erosion of the arroyo slopes and coastal bluff. Likewise, any portion of the buried pin pile wall system that becomes exposed in the future as a result of downslope soil movement will need to be retrofitted as a fully drained retaining structure by installing continuous support between the exposed piles with retaining wall components and backdrains.

Owners and occupants of the property should closely monitor the storm drainage provisions through the first significant rain season following completion of the project, in order to confirm the drainage systems are performing adequately and, if necessary, rectify unforeseen malfunctions.

We appreciate the opportunity to be of service. Should you have any questions, we can be reached at (831) 722-9446.

Sincerely,

PACIFIC CREST ENGINEERING INC.

Elizabeth Mitchell

Elizabeth Mae Bahns Mitchell
No. GE2718
Exp. 12/31/16

Elizabeth M. Mitchell, GE
President/Principal Geotechnical Engineer
GE 2718
Expires 12/31/16

Copies: 3 to Client

Pacific Crest Engineering Inc. www.4pacific-crest.com

444 Airport Blvd, Suite 106
Watsonville, CA 95076
Phone: 831-722-9446
Fax: 831-722-9158

January 11, 2013 Project No. 1256.1-SZ70-C47

Mark and Anny Corley
225 Whippet Run
Watsonville, CA 95076

Subject: **Emergency Bluff Repair**
340 Kingsbury Drive
A.P.N. 043-094-06
Aptos, Santa Cruz County, California

Dear Mr. and Mrs. Corley,

As requested, a representative from Pacific Crest Engineering Inc. was present at the subject site to observe the emergency bluff repair operations.

Our services included observation of stripping of vegetation, continuous observation during installation and proof testing of both test and production anchors, and observation of final mesh placement and erosion control provisions.

The landslide zone was exposed as designated by the project geologist and stripped of vegetation as recommended. A double helix test anchor was installed and load tested for design capacity, however since the anchor did not meet design capacity requirements the contractor proposed an alternative anchor configuration.

A total of thirty-two CS 2⅜“ Viking Helical anchors were installed from December 27th through December 31st, 2012. Each anchor had a lead section comprised of eight-inch, ten-inch and twelve-inch anchor plates. The anchors were installed to depths ranging from 14 to 16 feet, as measured from the face of the slope. All anchors were installed to minimum capacities ranging from about 20 to 30 kips, as determined by correlating installation torque to capacity. Correlation data was provided by Fresno House Movers, Inc.

It is our professional opinion that the anchors were installed in general conformance with the project plans, specifications and our geotechnical recommendations. Three of the production anchors were successfully proof tested 133% of design capacity.

The MacMat reinforced mesh was secured to the slope via cables anchored to face plates at each helical anchor location. Top soil was brushed into the mesh to assist with the hydroseed application. The mesh was then hydroseeded and covered with erosion control matting. Erosion control provisions were also placed at the upslope and downslope periphery to control runoff entering onto, and emerging from, the work zone.

EXHIBIT P

Based upon our observations, it is our professional opinion that the landslide repair work was performed in general conformance with the project plans, specifications, and our geotechnical recommendations.

We appreciate the opportunity to be of service on this project. If you have any questions concerning this report, please contact me at your convenience. I can be reached at 831-722-9446.

Sincerely,

PACIFIC CREST ENGINEERING INC.

Elizabeth M. Mitchell

No. GE2718
Exp. 12/31/14

Elizabeth M. Mitchell, G.E.
Vice-President, Geotechnical Group
GE 2718, Expires 12/31/14

Copies:	3 to Client
	1 to RI Engineering
	1 to Zinn Geology

EXHIBIT P

Comments & Correspondence

Application Number 181024

EXHIBIT Q

EXHIBIT Q

Nathan MacBeth

From: Richard Dye <richcdye@gmail.com>
Sent: Wednesday, November 20, 2019 3:17 PM
To: Nathan MacBeth; nathanmacbeth@santacruzcounty.org; Nathanmacbeth@santacruzcounty.us; nathanmacbeth@santacruzcounty.usa
Subject: 32.5 vs. 28.0

Hi, Nathan,

We are concerned again about a "slippery slope" issue on Beach Street. It is the next street below Bayview Drive.

Allowing 32.5 feet above street level rather than maintaining the 28 foot limit will not block the view of anyone on the cliff above them.

Nevertheless, looking at a street map rather than a topographical map would allow builders on Bayview Drive to assert that a precedent had been set to go above the 28 foot County limit.

Comments? Mediation?

Thanks for your advice on this issue.

Richard and Leslie Dye
633 Bayview Drive
Aptos, 95003

EXHIBIT Q 1

Nathan MacBeth

From: Richard Dye <richcdye@gmail.com>
Sent: Friday, November 22, 2019 9:03 AM
To: Nathan MacBeth; nathanmacbeth@santacruzcounty.org; Nathanmacbeth@santacruzcounty.us; nathanmacbeth@santacruzcounty.usa
Subject: 379 Beach Street Aptos 95003

Nathan,

This is the address of the building permit that we are concerned about.

32.5 versus 28,0 is our concern. We are the next block over and don't want to have a precedent set.

The question is: Can we do something? Maybe meet with the owner/architect?

Thanks.

Richard and Leslie Dye

EXHIBIT Q

From: william.stonhaus@ubs.com
Sent: Monday, March 9, 2020 10:43 AM
To: Nathan MacBeth
Subject: Regarding 379 Beach Dr in Aptos Ca

Nathan

My wife and I own 377 Beach Drive in Aptos Ca since 1993. We want to let who ever know that we are not happy with the purposed development at 379 Beach Dr. Why would they be exempt from building a bunker home when over the last many years that has been the requirement, no one else has had an exemption, it' s not right nor fair. We are also not happy with the fact that they want to build up 4.5 feet taller than any other structure, why again are they exempt from everyone else. Please share our concern.

Thanks Bill and Karen Stonhaus

UBS

William Stonhaus
Senior Vice President – Wealth Management
Toll Free: 888-274-5536

www.ubs.com/team/sfgroup

The Stonhaus Folland Group
UBS Financial Services, Inc.
50 W San Fernando St., 8th Floor
San Jose, CA 95113

Fig Garden Financial Center
5200 North Palm Suite 101
Fresno, CA 93704

Fax: 855-479-4550

EXHIBIT Q