Exhibit D

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REPORT
to
MR. TODD SLAWSON
C/O MR. DAVID DWIGHT
225 CANNERY ROW, SUITE A
MONTEREY, CALIFORNIA 93940

GEOTECHNICAL REPORT
for the proposed
SLAWSON RESIDENCE
30770 AURORA DEL MAR
CARMEL, CALIFORNIA
A. P. N. 243-341-005

by

GRICE ENGINEERING, INC.
561-A BRUNKEN AVENUE
SALINAS, CALIFORNIA
SEPTEMBER 2018

GRICE ENGINEERING INC

ENGINEERING GEOTECHNICS SEPTIC HYDROLOGY
FOUNDATIONS SOILS EARTH STRUCTURES

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File No. 6965-18.06
September 10, 2018

Mr. Todd Slawson
C/O Mr. David Dwight
225 Cannery Row, Suite a
Monterey, California 93940

Project: Slawson Residence
30770 Aurora Del Mar
Carmel, California
A. P. N. 243-341-005

Subject: Geotechnical Report

Dear Mr. Slawson;

Pursuant to your request, we have completed our geotechnical investigation and evaluation of the above named site. It is our opinion that this site is suitable for the proposed development, provided the recommendations made herein are followed.

In general, the native surface soils are loose, however most of these materials have been removed during construction of the existing structures. However loose or disturbed soils will most likely be encountered due to the proposed demolition. Recommendations are given relative to this and other characteristics within the report and especially under Special Recommendations.

The report contained herein is made with our best efforts to evaluate the site, determine the site's geotechnical conditions and provide recommendations for these conditions. We submit this report with the understanding that it is the responsibility of the owner, or his representative, to ensure incorporation of these recommendations into the final plans, and their subsequent implementation in the field.

In addition, we recommend that GRICE ENGINEERING, INC., be retained to review the project plans and provide the construction supervision and testing required to document compliance with these recommendations. Should any site condition not mentioned in this report be observed, this office should be notified so that additional recommendations can be made, if necessary.

This report and the recommendations herein are made expressly for the above referenced project and may not be utilized for any other site without written permission of GRICE ENGINEERING, INC.

Please feel free to call this office should you have any questions regarding this report.

Very truly yours,
GRICE ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
LAWRENCE E.
GRICE
RCE 66857
Exp. 09-30-20
091018DIGITAL
CIVIL
STATE OF CALIFORNIA

Lawrence E. Grice, P.E.
R.C.E. 66857

NOTICE TO OWNER

Any earthwork and grading performed without direct engineering supervision and materials testing by Grice Engineering Inc., will not be certified as complete and in accordance with the requirements set forth herein.

Foundations placed without observation of bearing conditions will not be certified as being in accordance with the requirements set forth herein.

Inspection of Work

It is recommended that all site work be inspected and tested during performance by this firm to establish compliance with these recommendations.

NOTIFY:	GRICE ENGINEERING INC.	SALINAS	(831) 422-9619
	561-A Brunken Avenue	MONTEREY	(831) 375-1198
	Salinas, California 93901	FAX	(831) 422-1896

A minimum of 48 hours (2 working days) notification is required prior to commencement of work so that scheduling for testing and inspections can be made.

Please be advised that costs incurred during inspection and testing of all site work is separate and not considered part of the fees as charged by Grice Engineering, Inc. for the report contained herein.

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GEOTECHNICAL REPORT
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SLAWSON RESIDENCE
30770 AURORA DEL MAR
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Introduction, Method and Scope of Investigation

The purpose of this report is to evaluate the geotechnical properties of the site relative to the construction of a single family residence. From these findings recommendations are given for the design of the development and subsequent construction.

For this purpose, the site was investigated, and prior information concerning construction and subsurface exploration in this area was examined for soils and materials data. The investigation consisted of a detailed site evaluation, which included: a site inspection; a review of literature made available to GRICE ENGINEERING, INC., including Site Plans from Holdren + Lietzke Architecture; geotechnical drilling and soil sampling; materials evaluation; and analysis of the geotechnical properties of the site soils. This report concludes the results of the investigation and provides recommendations based on that work.

The findings and recommendations contained in this report are applicable only to the above named site and its proposed development, and may not be utilized for any other site or purpose without written permission of GRICE ENGINEERING, INC.

Site Description

The project site is located to the west of 30770 Aurora Del Mar in the Carmel Highlands, south of the City of Carmel, in an un-incorporated area of westernmost Monterey County, California. Please refer to the Vicinity and Location Maps and the Site Map in Appendix A for details.

The topography of the 1.49 acre site encompasses an area containing a moderate western slope on a west facing marine terrace. The parcel is highest at Aurora Del Mar at an elevation of approximately 107 and descends to approximately 50 feet above mean sea level (msl). The majority of the site is covered with grass, landscaping, hardscaping and several trees.

Currently a single family residence is located on the western portion of the site with a driveway from the detached garage located to the east of the residence providing access to the street on the eastern property line. The residence is located on a nearly level pad approximately 12 feet below that of the parking and garage area.

As proposed the existing residence is to be demolished, and new structures will be built in the same approximate location. The garage will be improved with an addition to the eastern side which is to contain a gym. The approximately 4,000 square foot residence and 1,000 square foot gym are to be of conventional wood construction with support provided by isolated and/or continuous spread footings. The gym is to have a slab-on-grade floor. The lower walls of the gym will be constructed as retaining structures.

Field Investigation

Our field investigation consisted of a site inspection, along with drilling and sampling 5 exploratory bores to establish the subsurface soil profile, and obtain sufficient soil specimens to determine the soil characteristics. Drilling was accomplished by hand and continuous flight auger, with the spoil constantly examined, classified, and logged by field method in accordance with the Unified Soil Classification Chart[1] which is the basis of ASTM D2487-10. In the hand augured bores Penetration Resistance values were obtained through use of a dynamic cone penetrometer (ASTM Special Technical Publication #399). The blow count as measured in this method is Standard Penetration Resistance.

Relatively undisturbed soil samples were obtained by the penetration resistance method, (ASTM Method D1586-08), by which a split barrel sampler (ASTM D-3550-01) was driven a minimum of 18 inches into the sampled materials by free dropping a 140 pound weight 30 inches. The number of blows required to drive the sampler were recorded in 6 inch increments after conversion to Standard Penetration Resistance values utilizing the Burmister Formula. The number of blows required to drive the sampler the last two increments taken as the Standard Penetration Resistance. The split barrel sampler (ASTM D-3550-01), with dimensions of 2.4" I.D. x 3.0" O.D., is provided with 1 inch tall brass ring liners for the purpose of returning the samples to the laboratory in as near *in-situ** condition as possible.

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[1] Adopted 1952 by Corps of Engineers and Bureau of Reclamation. ASTM D2487 was developed as based on the Uniform Soils Classification Chart and System. The methods are equivalent.

The first four bores were advanced in 1977 for the original construction. Inspection of field conditions of the existing structure and grounds indicates no negative conditions other than some light erosion along the crown of the ocean bluff.

* *In-situ* refers to the in place state of soil. *In-situ* native soils are those which are in-place as deposited by nature and have not been disturbed by man's actions in the historic past.

Site Soil Profile

As found in the exploratory drilling, the site soils are generally consistent between each of the bores.

The shallowest soil horizon is fill materials generated from on site sources. These soils were encountered in the fifth bore located within the footprint of the gym. Fill materials should be expected along the western edge of the parking area west of the garage and along the western margin of the residence.

The natural topsoil was observed to be a dark grayish brown sand of fine to medium gradation with few amounts to coarse sand and little to some amounts of silty clay. This soil was observed moist and loose-medium dense.

At approximately 2 feet below natural grade the soils become a sandy clay of dark to medium yellowish brown color. These soils were observed to be a very weathered portion of the underlaying terrace deposit.

At approximately 3 feet below natural grade are less weathered terrace deposits primarily consisting of fine to medium sands with coarser clasts noted. These little to some silty clay which decreases some with depth. They were observed medium dense to dense and damp to moist.

Complete soil characteristics and comments are reported on the boring logs at the depths observed. The logs are located in Appendix B.

Groundwater

No groundwater was encountered at this site to the maximum depth of exploration, approximately 25 feet below grade.

Laboratory Testing

Laboratory testing consisted of establishing the *in-situ* ** moisture content and dry density (ASTM D 2487-10) and unconfined penetration, direct shear testing (ASTM D 3080-04) and expansion index (ASTM D4829-08a). Standard Penetration Resistance values gained during the exploratory drilling are also included.

The following is a tabulation of the field and laboratory test result extremes:

TABLE 1

SUMMARY OF SOIL PROPERTIES

TEST	MAXIMUM	MINIMUM
Standard Penetration Resistance	42 blows/foot	6 blows/foot
Unconfined Compression*	9+ kips/ft^2	6 kips/ft^2
In-Situ Density	116.0 lbs/ft^3	108.7 lbs/ft^3
In-Situ Moisture	27 %	5.6 %
Angle of Internal Friction	32 degrees	26 degrees
Cohesion	1810 lbs/ft^2	126 lbs/ft^2
Expansion Index	35	

All data obtained is reported in Appendix B including the boring logs, with soil classified described at depth observed.

* Pocket Penetrometer

** *In-situ* refers to the in-place state.

Seismic History

Although no fault traces are thought to directly cross the building site, Monterey County is traversed by a number of faults most of which are relatively minor hazards for the purposes of the site development. As such, this site will experience seismic activity of various magnitudes emanating from one or more of the numerous faults in the region.

Various maps presently exist, allowing observation on the site of distinctive geologic features. Some maps, such as that by Burkland and Associates (Reference No. 10) developed for Monterey County, are compilations from various sources detailing the locations of studied faults. Faults have inherit variances within their zones, and discoveries of new fault segments or entire faults is ongoing. There is also some difference in exact fault line location from source map to map, making precise location of said faults difficult. Therefore, relative to the information contained within this report, the following is considered to be as accurate as is currently possible from information made available to Grice Engineering Inc..

Regional Faults

Of most concern are active faults which have tectonic movement in the last 11,000 years and as such are called Holocene Faults and potentially active faults. The following are those nearest listed (Reference No. 12).

The most active is the San Andreas Rift System (Pajaro), located approximately 33.2 miles to the northeast. It has the greatest potential for seismic activity with estimated intensities of VI-VII Mercalli in this location.

Other fault zones are the San Gregorio-Palo Colorado (Sur) Fault Zone, the center of which is located approximately 1.7 miles to the southwest, the Monterey Bay-Tularcitos Fault Zone, approximately 7.6 miles to the northeast, the Rinconada Fault Zone, approximately 16.5 miles to the northeast, and the Zayante-Vergeles Fault Zone, approximately 29.0 miles to the northeast. These zones are not as liable to rupture as the San Andreas and a seismic event at any of the above fault zones would likely produce earth movements of a lesser intensity at the site.

Local Faults

In addition to the fault zones as discussed above, the local fault is as listed below as shown on the following maps, “Preliminary geologic map of the Monterey and Seaside 7.5 minute quadrangles, Monterey County, California, with emphasis on active faults” (Reference No. 15), “Geological Map of the Monterey and Seaside 7.5 minute Quadrangles, Monterey County, California: A Digital Database” (Reference No. 16), “Geologic Map of the Monterey Peninsula and Vicinity, Monterey, Salinas, Point Sur, and Jamesburg 15-Minute Quadrangles, Monterey County” (Reference No. 22), “Fault Activity Map of California: California Geological Survey Geologic Data Map” (Reference No. 32), and “Quaternary Fault and Fold Database for the United States” (Reference No. 46) including the USGS overlay on Google Earth.

TABLE OF LOCAL FAULTS			
FAULT, PERPENDICULAR TO SITE	**APPROXIMATE DISTANCE FROM SITE**	**DIRECTION**	**TIME OF LAST DISPLACEMENT ON FAULT (Ref. 32)**
Palo Colorado Fault, inferred, concealed beneath the Pacific Ocean	0.90 miles	southwest	Holocene

Liquefaction

The site soils are considered not susceptible to liquefaction as they are un-saturated and dense sands or stiff silty clays.

Differential-Total Settlement - Static and Dynamic

The recommendations given in the Geotechnical Report are such that concerns of settlement are negligible. The total settlement is expected to be less than 1/4 inch and the expected differential settlement less than one half that.

Hydro-Collapse and Subsidence

As observed the native surface soils to an approximate depth of two feet are loose. These soils possess some capacity to settle under hydraulic loading. However this effect is not common in the area. The recommendations given in this report were established to reduce the potential of this occurring.

The area is not within a known Subsidence Zone.

Slope Stability

Inspection of the site indicates that no landslides are located above or below the building area and the area is generally not susceptible to slope failure. The shearing strengths are moderate to high and combine both angle of internal friction ranging from 26 to 32 degrees and cohesion ranging from 126 to 1810 p.s.f..

Slope Stability and Erosion

The parcel was evaluated for landslides located above or below the building area. The site evaluation included the method as delineated in "Special Publication 117A Guidelines for Evaluating and Mitigating Seismic Hazards in California" was reviewed as applicable to this site. The following summarizes the findings.

The following methods and publications were utilized to determine the presence of land movement or excessive erosion above and below the project site.

A. On site evaluation of land features.
B. Aerial photographs spanning the time frame from May 27, 1994 to February 4, 2018.
C. Open File Report 7-718, 1977, Green
D. Geologic Map of California - Santa Cruz Sheet, 1958, Jennings etc.
E. Ground Failures in the Monterey Bay Counties Region, Professional Paper 993, Dept. of the Interior.

1. "Are existing landslides, active or inactive, present on, or adjacent (either uphill or downhill) to the project site?"

There are no existing landslides, active or inactive, present on, or adjacent to the project site.

The generally area is considered not susceptible to mass slope failure due to the medium dense to dense character of the underlain soils. These soils are a portion of the local granite bedrock which is generally exposed at the bluff and located between 7 to 25 feet below grade.

No features or conditions were visually observed during the site exploration which indicate or suggest landsliding has or will occur above or below the project site.

No recorded features were noted on any of the reviewed publications which suggest, imply or note landslides have or will occur above or below the project site.

2. “Are there geologic formations or other earth materials located on or adjacent to the site that are known to be susceptible to landslides?”

There are no geologic formations, or other earth materials located on or adjacent to the site that is known to be susceptible to landslides. The natural grade is approximately 14% however site grading has provided near level pads for the existing residence, garage and parking area. Cut banks have been provided with retaining structures.

The native topsoil is compressible due to its natural characteristics. This characteristic is addressed in the Geotechnical Report.

3. “Do slope areas show surface manifestations of the presence of subsurface water (springs and seeps), or can potential pathways or sources of concentrated water infiltration be identified on or upslope of the site?”

No springs or seeps or the indication of such were observed during the site exploration. Review of the aerial imagery did not indicate any locations of seepage as suggested by increased or more active vegetation or topography (erosion scarp, slump). Spring or seeps within the general area and lithology are not typical.

Drainage over the local terrain is unfocused with some managed drainage around developed areas and along the street to the east.

Inspection of areal photographs indicates the terrain and presence of vegetation has been consistent during that period. The photographs detail the coastal bluff with nearly no change in form or cover.

These characteristics in conjunction with the firm soils indicated a low potential for rapid solifluction or debris flow.

4. “Are susceptible land forms and vulnerable locations preset?”

No excessively steep or erodible slopes are located above or below the site. Although the ocean bluff is relatively steep, the bedrock projects relatively far to the ocean providing moderate protection from times of high surf.

5. “Given the proposed development, could anticipated changes in the surface and subsurface hydrology (due to watering of lawns, on-site sewage disposal, concentrated runoff from impervious surfaces, etc.) increase the potential for future landsliding in some areas?”

The area is generally fully developed. No developable lands are located up slope of the parcel. Future construction within the area will most likely be residential additions or replacement of existing structure. Further mass grading of land is unlikely. Future changes to land use (new septic, increase landscape, use of land) is unlikely. Any changes to drainage conditions will be minor. Only minor changes to drainage and landscaping are proposed for this project.

Seismic Strength Loss

The site soils are considered resistant to seismic strength loss and the resulting momentary liquefaction. The relatively short duration of earthquake loading will not provide a significant number of high amplitude stress cycles to alter the strain characteristics. Additionally the clay-silt fraction is not considered quick nor sensitive, as such it will not have the associated loss of strength.

Chemical Reactivity

The area is well developed with structures, generally found on Portland Cement products. Additionally these structures date back to the 1940's or earlier. Much of the concrete used in these structures has remained as cast. The area soils are not known for sulfate reaction with Portland cement products and as such chemical reactivity is not considered a problem in this area.

Expansive Soils

In general the site soils are or contain silty clays of low to medium plasticity. These soils are typical to the area. Expansivity has not been influential to the existing structure as no deformations attributable to expansive soils were observed. Additionally there are no known problems with expansive soils in the area.

Surface Rupture and Lateral Spreading

The project site is located 0.9 miles to the northeast of the Palo Colorado Fault. The site inspection did not reveal any surface features indicating a fault rupture has occurred at the site. The existing structure, driveways and roads do not reveal any strains which would be attributable to subsurface lateral or vertical displacements resulting from fault slip. Therefore surface rupture from fault activity across the site is considered improbable.

The project site is underlain by relatively strong soils and granite bedrock at a shallow depth. These materials are considered resistant to lateral spreading. As such surface rupture from lateral spreading is considered improbable.

Seismicity

It is recommended that all structures be designed and built in accordance with the requirements of the California Building Code's current edition. All buildings should be founded on undisturbed native soils and/or tested and accepted engineering fill to prevent resonance amplification between soils and the structure.

2016 California Building Code Geoseismic Classifications

The California Building Code, 2016 edition (Reference No. 13), provides for seismic design values. These values are to be utilized when evaluating structural elements. The soils profile determination is based on the penetration resistance data developed from advancement of exploratory bores. Using averaged penetration values per depth of soils type gives an overall site value of 45 blows/foot penetration resistance as per Equation 20.4-3, ASCE 7-10. The geoseismic character is as listed in the following table.

2015 I.B.C. - 2016 C.B.C. EARTHQUAKE LOADS: SECTION 1613				
LATITUDE	36.476460	SOIL PROFILE:	Stiff Soils	
LONGITUDE	-121.937674	SITE CLASS	D	
PERIOD	S	F	Sm	Sd
0.2 sec	Ss = 1.783	Fa = 1.0	Sms = 1.783	Sds = 1.188
1.0 sec	S1 = 0.711	Fv = 1.5	Sm1 = 1.066	Sd1 = 0.711
Seismic Design Category to be assigned by structural engineer or designer				

CONCLUSIONS OF INVESTIGATION

In general, the suitable, *in-situ**, native soils and certified engineered fill are acceptable for foundation purposes and display engineering properties adequate for the anticipated soil pressures, providing the recommendations in this report are followed.

Special Recommendations

After demolition of the existing residence it is recommended that the surficial soils be processed as engineered fill to an approximate depth of 2 feet or 1 foot below the bottom of new foundations. The lateral extent of the processing is to include the footprint of the structure and attached features such as porches and as given under General Grading Recommendations.

Such processing is also recommended for unattached minor structures or on grade structures (slabs on grade) beyond the foot print of the residence. However application of this is left to the discretion of the owner.

For the proposed gym structure there is no recommendation for processing of the subgrade soils at this time. As detailed the structure is to be positioned near the floor elevation of the garage which will require an excavation between 5 to 9 feet below grade. Due to this all unsuitable, loose or disturbed soils will most likely be removed. Further recommendation can be given after excavation to subgrade.

The existing driveway extending from Aurora del Mar to the parking area is distressed. This condition is most likely due to poor development of the subgrade soils. Should the driveway be repaired or improved it is recommended that the subgrade soils be processed as engineered fill to prevent early failure of new pavement.

The area has been developed and as such underground utilities may be located within the area of proposed construction. In addition, buried objects or deeply disturbed soils may also be encountered. As such all care and practice is to be exercised to observe for and locate any such objects. Where these objects are to be removed or use discontinued, they are to be removed in their entirety and all disturbed soils are to be processed as engineered fill.

The base of all excavations and over-excavations are to be inspected by the Soils Engineer prior to further processing, steel or form placement. Any further site activity, especially grading and foundation excavations, should be under the direction of a qualified Soils Engineer or their Representative. Should the spectrum of development change, this office should be notified so that additional recommendations can be made, if necessary.

* Suitable, *in-situ,* native soils are those soils which are in-place as deposited by nature and have characteristics adequate for support of the intended load or application.

Foundations and Footings

Geotechnical evaluation indicates that square, round, and continuous spread footings are satisfactory types of support. The minimum embedment for shallow, spread foundations is 12 inches for single stories and 18 inches for two stories into suitable, *in-situ*,* native soils or certified engineered fill. Embedment depths do not take into account the loose upper top soils, disturbed soils or any other unacceptable soils which exist at the site, e.g., any un-engineered fill, landscaping soils, etc.

VERTICAL SOIL PRESSURES[1]	
FOOTING TYPE	DEAD + LL, $kips/ft^2$
Spread & Isolated	3.0
LATERAL SOIL PRESSURES[1]	
TYPE	VALUE, lbs/ft^2
Active Earth Pressure	28 lbs/ft^3 (Equivalent Fluid Pressure)
Restrained Earth Pressure	49 lbs/ft^3 (Equivalent Fluid Pressure)
Seismic	2 $lbs/ft^3 x H^2$ applied at 0.6H
Friction at Base	0.35 × Dead Load
Passive Earth Pressure	325 $lbs/ft^3 \times H^2$ NOTE2
Uplift Friction	175 $lbs/ft^2 \times H$

Notes: LL = Live Load; DL = Dead Load; H = Vertical height of material retained.
One-third increase to be allowed for wind and seismic forces.
[1] For depths into acceptable native materials or engineered fill.
[2] Excludes near surface 0.5 feet of *in-situ* soils.

Pile and Pier foundation information is not provided as none are required or proposed. All foundation excavations are to be cleaned of debris and loose or otherwise unsuitable soils prior to placement of concrete.

* Suitable, *in-situ,* native soils are those soils which are in-place as deposited by nature and have characteristics adequate for support of the intended load or application.

Slabs-on-Grade

All slabs should be constructed over a prepared sub-grade placed on suitable *in-situ** native material or certified engineered fill. The site exploration observed that the existing surficial soils are loose to depths of approximately 2 feet. These soils should not be relied upon for support of slabs on grade or other surficial structures.

As such where any unsuitable soils remain after excavation to subgrade they are to be processed as engineered fill prior to further fill placement or construction of the on grade structure. At a minimum the upper 6 inches of subgrade below all surficial structures should be processed as engineered fill in areas of on grade structures.

The sub-grade materials should be observed and accepted by a qualified Soils Engineer or their representative prior to placement of forms, reinforcing or concrete.

On-grade slabs should be placed over a moisture vapor barrier consisting of a waterproof membrane (Moist Stop, 10 mil Visqueen, or equal) with a 2 inch protective sand cover. The waterproof membrane should be placed over a capillarity break consisting of 4 inches of open graded rock; round and sub-round rock is recommended to prevent puncture of the membrane. Open graded crushed aggregate may be utilized, provided the vapor barrier is protected from puncture by a cushion of filter fabric (Mirafi 140N or equal) laid over the aggregate prior to placement of the membrane. Where such concerns are not warranted, alternative underlayment may be utilized at the owners discretion.

All care and practice required to prevent puncture of the membrane during placement and pouring of covering slabs should be utilized during construction. Unless otherwise required for structural purposes, all slabs should be reinforced with a minimum of No.4, Grade 40, deformed steel reinforcing bar, 24 inches o.c., each way, to prevent separation and displacement in cases of cracking.

* Suitable, *in-situ,* native soils are those soils which are in-place as deposited by nature and have characteristics acceptable for support of the intended load or application.

Specifications for Rock Under Floor Slabs

Definition: Graded gravel of crushed rock for use under floor slabs shall consist of a minimum thickness of mineral aggregate placed in accordance with these specifications and in conformance with the dimensions shown on the project plans. The minimum thickness is specified under the section Slabs-on-Grade above.

Material: The mineral aggregate for use under floor slabs shall consist of broken stone, crushed or uncrushed gravel, quarry waste, or a combination thereof. The aggregate shall be free from adobe, vegetable matter, loam, volcanic tuff, and other deleterious substances. It shall be of such quality that the absorption of water in a saturated dry condition does not exceed 3 percent of the oven dry weight of the sample.

Grading: The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by the use of laboratory sieves, U.S. Standard, in compliance with ASTM C 136-06, Standard Method for Sieve Analysis of Fine and Coarse Aggregates, will conform to the following grading specification:

SIEVE SIZE	PERCENTAGE PASSING SIEVE
3/4 inch	100 %
No. 4	0 - 10 %
No. 200	0 - 2 %

Placing: Sub-grade upon which gravel or crushed rock is to be placed shall be prepared as outlined in the Recommended Grading Specifications. In addition, the Sub-grade shall be kept moist so that no drying cracks appear prior to pouring slabs. If cracks appear, Sub-grade shall be moistened until cracks close.

Slope Ratio and Drainage

Analysis of site soils indicate that cut and fill slope ratios of 2 horizontal to 1 vertical will be satisfactory provided they are landscaped with soil retaining ground covers and are protected against concentrated over slope drainage. Cut slopes exposing the granite bedrock or similar stable materials may be allowed to steeper gradients. These conditions should be reviewed on site.

Surface Drainage and Erosion Control

Design and construction of the project should fit the topographic and hydrologic features of the site. It is important to minimize unnecessary grading of or near steep slopes. Disturbing native vegetation and natural soil structure allows runoff velocity and transport of sediments to increase.

General surface drainage should be retained at low velocity by slope, sod or other energy reducing features sufficient to prevent erosion, with concentrated over-slope drainage carried in lined channels, flumes, pipe or other erosion-preventing installations.

Runoff flows should be directed into pipes or lined ditches and then onto an energy dissipater before discharging into streams or drainage ways. De-silting should be provided as necessary and may take form of stilling basins, gravel berms, forested/vegetated screens, etc.

All concentrated roof and area drainage should be conveyed and released to the bedrock surface at the base of the ocean bluff.

Storm runoff should never be directed to septic tank system leachfields and no collected or concentrated drainage should be allowed to discharge to adjacent steep slopes.

A sub-surface dispersal system **MAY NOT** be used on this site.

During construction, never store cut and fill material where it may wash into streams or drainage ways. Keep all culverts and drainage facilities free of silt and debris. Keep emergency erosion control materials such as straw mulch, plastic sheeting, and sandbags on-site and install these at the end of each day as necessary.

Re-vegetate and protect exposed soils by October 15. Use appropriate grass/legume seed mixes and/or straw mulch for temporary cover. Plan permanent vegetation to include native and drought tolerant plants. Seeding and re-vegetation may require special soil preparation, fertilizing, irrigation, and mulching.

Subsurface Drains

Use of spun filter fabric is not recommended for use in construction subsurface drains as this type of fabric typically becomes clogged. Should filter fabric be necessary it is recommended that a woven fabric be used such as Mirafi Filterweave 300. Otherwise we would recommend omission of the fabric and placement of Caltrans Class 1, Type ‘A” or “B” drain rock, and that any fabric only be placed near the top of the trench between the gravel and earth backfill or where the gravel extends to grade, 1 foot below finish grade.

CLASS 1		
SIEVE SIZES	**PERCENTAGE PASSING**	
	TYPE A	TYPE B
50.0-mm/2 inches	----	100
37.5-mm/1.5 inches	----	95-100
19.0-mm/0.75 inches	100	50-100
12.5-mm/0.5 inches	95-100	-----
9.5-mm/0.415 inches	70-100	15-55
4.75-mm/No. 4	0-55	0-25
2.36-mm/No. 8	0-10	0-5
75.0-μm/No.200	0-3	0-3

General Grading Recommendations

For those items not directly addressed, it is recommended that all earthwork be performed in accordance with the following.

General: This item shall consist of all clearing and grubbing; preparation of land to be filled; excavation and fill of the land; spreading, compaction and control of the fill; and all subsidiary work necessary to complete the graded area to conform with the lines, grades and slopes as shown on the approved plans.

The Contractor shall provide all equipment and labor necessary to complete the work as specified herein, as shown on the approved plans as stated in the project specifications.

Preparation: Site preparation will consist of clearing and grubbing any existing structures and deleterious materials from the site, and the earthwork required to shape the site to receive the intended improvements, in accordance with the recommended grading specifications and the recommendations as provided above.

All vegetable matter, irreducible material greater than 4 inches and other deleterious materials shall be removed from the areas in which grading is to be done. Such materials not suitable for reuse shall be disposed of as directed.

After the foundation for fill has been cleared, it shall be brought to the proper moisture content by adding water or aerating and compacting to a Relative Compaction of not less than 90% or as specified. The soils shall be tested to a depth sufficient to determine quality and shall be approved by the Soils Engineer for foundation purposes prior to placing engineered fill.

General Fill: General fill shall be placed only on approved surfaces, as engineered fill, and shall be compacted to 90% Relative Compaction. Native soils accepted for fill or existing aggregate fill may be used for fill purposes provided all aggregate larger than 6 inches are removed. The material for engineered fill shall be approved by the Soils Engineer before commencement of grading operations.

Each layer shall be compacted to a Relative Compaction of not less than 90% or as specified in the soils report and on the accepted plans. Compaction shall be continuous over the entire area of each layer.

The selected fill material shall be placed in layers which, when compacted, shall not exceed 6 inches in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to ensure uniformity of material in each layer. Fill shall be placed such that cross fall does not exceed 1 foot in 20 unless otherwise directed.

When fill material includes rock or concrete rubble, no irreducible material larger than 4 inches in greatest dimension will be allowed except under the direction of the Soils Engineer.

Imported Materials: Materials imported for fill purposes shall be classified as: SAND, group symbol SW, SP, SC or SM, as given in ASTM 2487-10, "The Classification of Soils For Engineering Purposes." In all cases the portion finer than the No. 200 sieve shall not contain any greatly expansive clays and shall be free from vegetable matter and other deleterious materials. The material for engineered fill shall be approved by the Soils Engineer before commencement of grading operations.

Structural Backfill: Trench, wall and structural backfill shall be placed only on approved surfaces, as engineered fill, and shall be compacted to 95% Relative Compaction. Materials imported for backfill purposes shall have a Sand Equivalent of no less than 30 and shall be classified as Clean Sands as designated in "The Classification of Soils For Engineering Purposes" (ASTM 2487-10).

Pavement Grades: All pavement grades shall be of uniform thickness, density and moisture prior to placement of the next grade. Flexure of each or all grades shall not exceed 0.25 inches in 5 feet under an axial load of 18.5 kip.

Aggregate Base Course: All aggregates used for specified base courses, shall be handled in a manner which prevents segregation and non-uniformity of gradation.

Compaction: All re-compacted soils and/or engineered fill should be placed at a minimum 90% Relative Compaction or at the value required for that portion of the work. All pavement sections should be compacted to a minimum of 95% Relative Compaction.

Field density testing shall be completed by the Soils Engineer on each compacted layer or as determined by the Soils Engineer. At least one test shall be made for each 500 cubic yards or fraction thereof, placed with a minimum of two tests per layer in isolated areas. Where a sheeps'-foot roller is used, the soil may be disturbed to a depth of several inches. Density tests shall be taken in compacted materials below the disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof, is below the required density, that particular layer or portion shall be reworked until the required density has been obtained.

Moisture: During compaction moisture content of native soils should be that consistent with the moisture relative to 95% Relative Compaction and in no case should these materials be placed at less than 3 percent above the specific optimum moisture content for the soil in question. The engineer may elect to accept high moisture compacted soils provided the materials are at 95% Relative Wet Density at that moisture content.

The moisture content of the fill material shall be maintained in a suitable range to permit efficient compaction. The Soils Engineer may require adding moisture, aerating, or blending of wet and dry soils.

All earth moving and work operations shall be controlled to prevent water from running into and pooling in excavated areas. All such water shall be promptly removed and the site kept drained.

Tests: All materials placed should be tested in accordance with the Compaction Control Tests: “Density of Soil In-Place by Sand Cone Method” (ASTM D-1556-07), “Moisture-Density Relationship of Soils” (ASTM D-1557-09), and “Density of Soils In-Place by Nuclear Method” (ASTM D-6938-10).

The standard test used to define maximum densities of all compaction work shall be the A.S.T.M. D-1557-09, Moisture Density of Soils, using a 10-pound ram and 18-inch drop. All densities shall be expressed as a relative density in terms of the maximum density obtained in the laboratory by the foregoing standard procedure.

Deleterious Materials: Materials containing an excess of 5% (by weight) of vegetative or other deleterious matter may be utilized in areas of landscaping or other non-structural fills. Deleterious material includes all vegetative and non-mineral material, and all non-reducible stone, rubble and/or mineral matter of greater than 6 inches.

Over-Excavations: Over-excavations, when required, should include the foundation and pavement envelopes. Such excavations should extend beyond edge of development a minimum of 5 feet and to an imaginary line extending away and downward at a slope of 45 degrees from the edge of development. The process shall include the complete removal of the required soils and subsequent placement of engineered fill. After removal of the soils to the required depth, the base of the excavation shall be inspected and approved by the Soils Engineer or his representative prior to further soils processing or placement. Based on this inspection other recommendations may be made.

Existing Conditions: In developed areas underground utilities may be located within the area of proposed construction. In addition, buried objects or deeply disturbed soils may also be encountered. As such all care and practice is to be exercised to observe for and locate any such objects. Where these objects are to be removed or use discontinued, they are to be removed in their entirety and all disturbed soils are to be processed as engineered fill.

Key: All fills on slopes greater than 1 vertical to 6 horizontal shall be keyed into the adjacent soil. The toe of all slopes should be supported by a key cut a minimum of 3 feet into undisturbed soils to the inside of the fills toe. This key should be a minimum of 6 feet in width and slope at no less than 10% into the slope. In addition, as the fill advances up slope benches, 3 feet across, should be scarified into the fill/undisturbed soil interface.

Seasonal Limits: When the work is interrupted by rain, fill operations shall not be resumed until field tests by the Soils Engineer indicate that the moisture content and density of the fill is as previously specified and soils to be placed are in suitable condition

Unusual Conditions: In the event that any unusual conditions are encountered during grading operations which are not covered by the soil investigation or the specifications, the Soils Engineer shall be immediately notified such that additional recommendations may be made.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based on our understanding of the project as represented by the plans, and the assumption that the soil conditions do not deviate from those represented in this site soils investigation. Therefore, should any variations or undesirable conditions be encountered during construction, or if the actual project will differ from that planned at this time, GRICE ENGINEERING INC. should be notified and provided the opportunity to make addendum recommendations if required.

NOTIFY:	GRICE ENGINEERING INC.	SALINAS	(831) 422-9619
	561-A Brunken Avenue	MONTEREY	(831) 375-1198
	Salinas, California 93901	FAX	(831) 422-1896

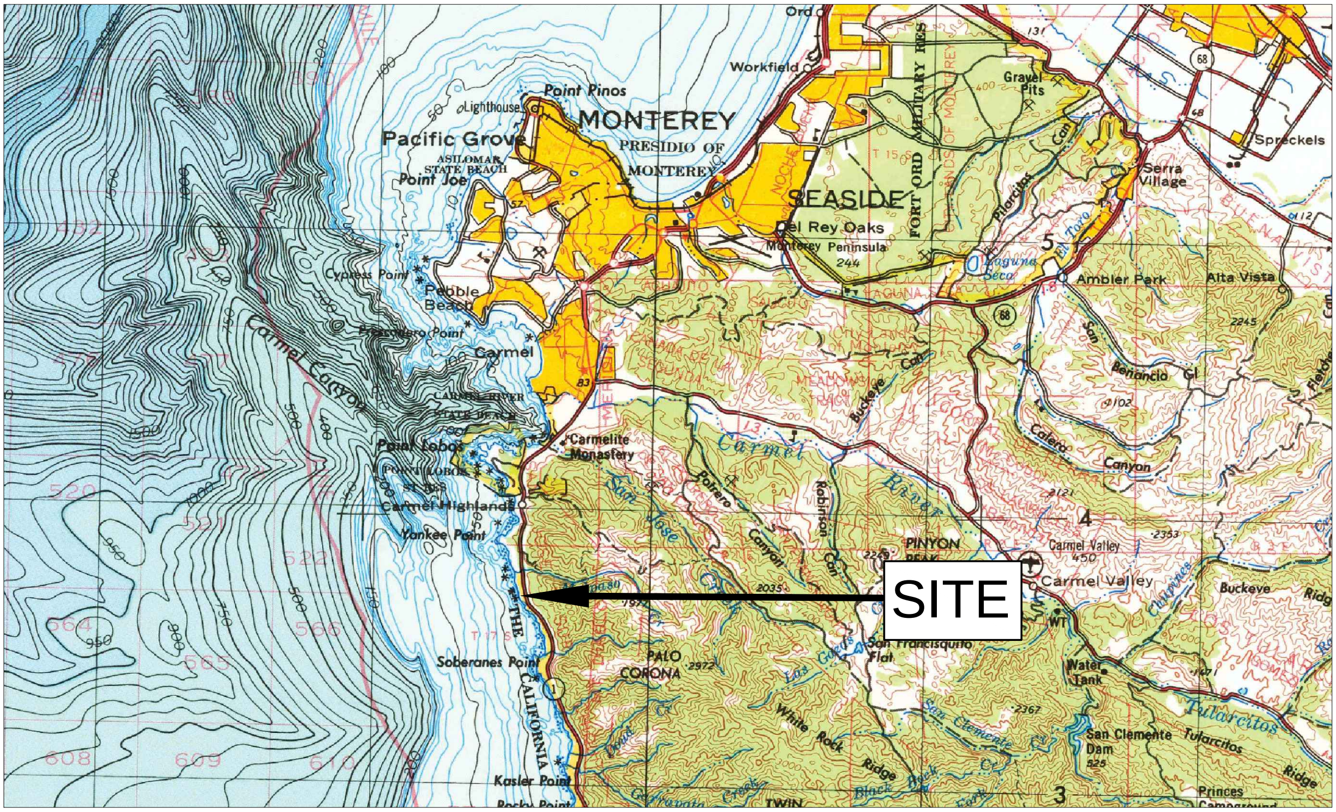
This report is issued with admonishment to the Owner and to his representative(s), that the information contained herein should be made available to the responsible project personnel including the architects, engineers, and contractors for the project. The recommendations contained herein should be incorporated into the plans, the specifications, and the final work.

It is requested that GRICE ENGINEERING INC. be retained to review the project grading and foundation plans to ensure compliance with these recommendations. Further, it is the position of GRICE ENGINEERING INC. that work performed without our knowledge and supervision, or the direction and supervision of a project responsible professional soils engineer renders this report invalid.

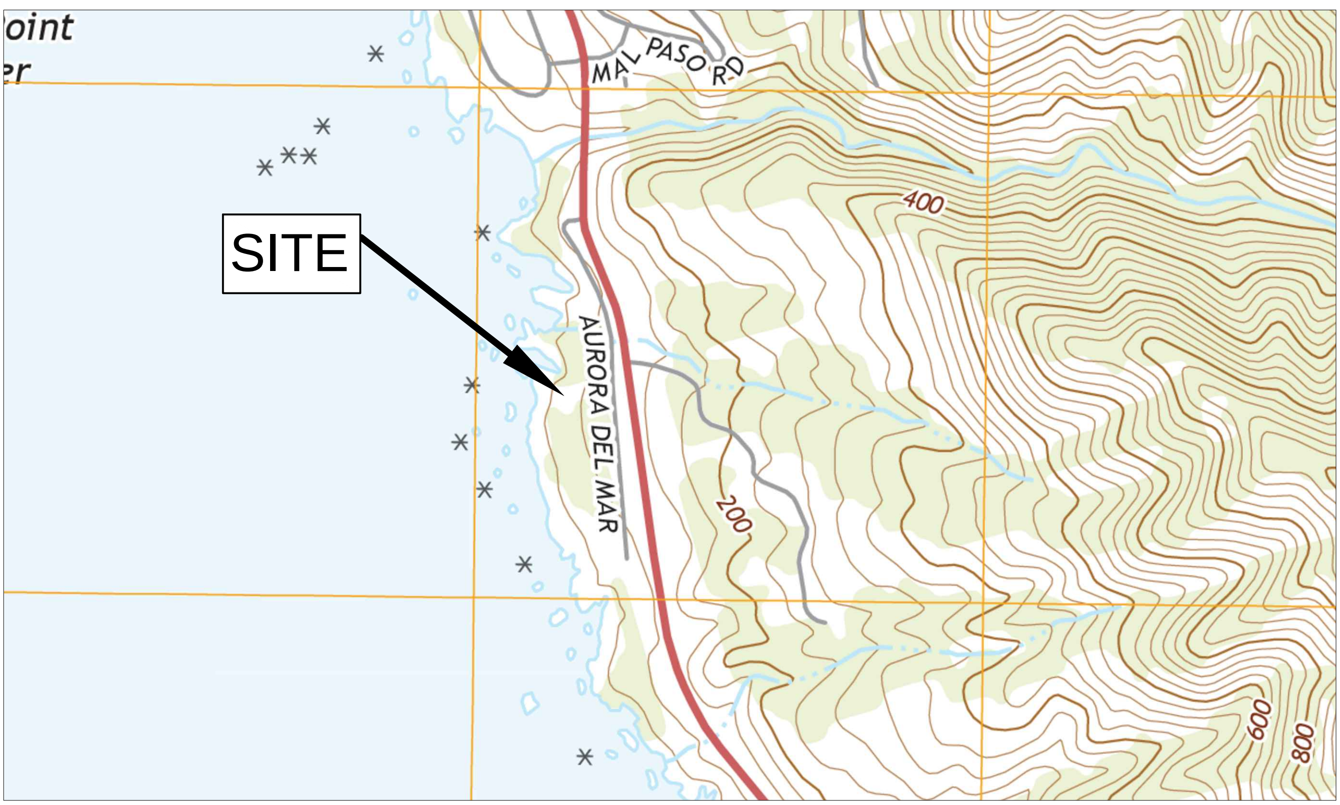
It is our opinion the findings of this report are **valid** as of the **present date**, **however**, changes in the **Codes and Requirements** can occur and change the recommendations given within this report concerning the property. In addition changes in the conditions of a property can occur with the passage of time, due either to natural processes or to the works of man and may effect this property. In addition, changes in **standards** may occur as a result of legislation, or the broadening of knowledge, and these changes may require re-evaluation of the conditions stated herein. Accordingly, the findings of this report may be invalidated wholly, or partially, by changes beyond our control. Therefore, this report is subject to review and should not be relied upon after a period of **three years**.

REVISED 01-07-2011

APPENDIX A



Vicinity Map



Location Map

GRICE ENGINEERING INC
561A Brunken Avenue
Salinas, California 93901
Salinas: (831) 422-9619
Monterey: (831) 375-1198 FAX: (831) 422-1896

Vicinity and Location Map
Slawson Residence; 30770 Aurora Del Mar
Carmel, CA File No. 6965-18.07

GRICE ENGINEERING INC
ENGINEERING • GEOTECHNICS • HYDROLOGY • SOILS • FOUNDATIONS • EARTH STRUCTURES
561A Brunken Avenue Salinas, California Salinas: (831) 422-9619 Monterey: (831) 375-1198 FAX: (831) 422-1896

SITE PLAN with BORE LOCATIONS
Slawson Residence, 30770 Aurora Del Mar, Carmel, CA A.P.N. 243-341-005

APPENDIX B

File No. 1239-77
June 8, 1977

BORING LOGS

Depth	Boring Number 1	Boring Number 2	Boring Number 3
0	Dark Sandy Silt	Dark Sandy Silt	Dark Sandy Silt
1	Tan Silty Clay	Tan Silty Clay	
2			
3	Medium-Firm Sandy Clay		Dark Silty Clay
4		Tan Clayey Sand	Tan Silty Clay
5	Firm Sandy Clay		
6			
7		Tan Clayey Silt	Refusal - Impervious
8			
9	Refusal - Impervious		
10		Very firm Sandy Clay	
11			
12		Refusal - Impervious	

Boring No. 4

Depth	S	G	D	W%	C	ϕ	Qn	N	Remarks
0									
		SC	109	8	1810	32	9+	42	Topsoil - dark humis silty sand
									Very firm tan sandy clay
5									
			116	22	860	26	9+	35	
10									
									Very firm gravelly sandy clay
15			114	27	126	29	9+	39	
20									Cobbles
25									Very hard - apparent bedrock
30									

Slawson Residence 30770 Aurora Del Mar, Carmel Highlands

Boring No. 5

September 04, 2018

Depth	Symbol	Sample	Field BlowCount per 6 inch	Standard Pen. Burmister	Description	Auger Pen.	Density	Moisture	Unconfined	Cohesion	Shear
0.00	SM-SC				(CUTTINGS) Dark yellowish brown; lightly mottled | SAND; fine to medium ; few to coarse; granitic base | few: silt to clots of clay with sands | slightly moist; loose-medium dense | Appears as a fill of native source.						
0.50											
1.00					Tube Sample		108.7	5.6	6.00		
						Atterburg: Non plastic					
						Percent fine No. 200 = 12.8					
1.50	SC				(CUTTINGS) Dark grayish brown | SAND; fine to medium; few to coarse; granitic base | little to some (varies): silty clay | moist; loose-medium dense | Presumably native topsoil.						
2.00			DCPT								
		2.00	6.00	6.00							
			7.00	7.00							
2.50											
3.00					Blending between the topsoil and subsoil which is weathered terrace deposit						
3.50					Tube Sample		113.5	6.8	7.00		
						Atterburg: Non plastic					
						Percent fine No. 200 = 10.3					
4.00	CL				(CUTTINGS) Dark to medium yellowish brown | silty CLAY; low-medium plasticity; moderately pliable- friable | few to little (varies): sand; fine to medium; few to coarse; granitic base | damp; medium dense | Presumably very weathered native terrace deposit.						
4.50					Bag	Atterburg: LL=33: PL= 15: PI=18					
						Expansion Index = 35					
5.00	SC				(CUTTINGS) Dark to medium yellowish brown | SAND; fine to medium; few to coarse; granitic base | little to some (varies): silty clay | damp; medium dense increasing to dense | Presumably native terrace deposit.						
5.50			DCPT								
		2.00	38.00	38.00	Very firm drilling to end of bore						
6.00											
6.50											
7.00											
7.50					Appears to be either weathered granite or weathered granite cobbles.						
8.00											
8.50					Top of bore at is near to eve line of garage						
9.00					End of bore at 9.0 feet. No free water encountered. Bore backfilled with cuttings						
9.50											
10.00											

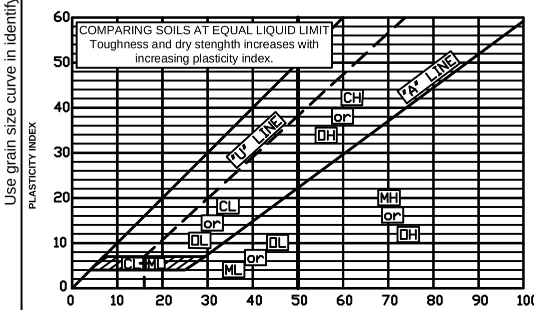
UNIFIED SOIL CLASSIFICATION & ASTM D2487: INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES: Excluding particles larger than 3 inches and basing fractions on estimated weights

Major division	Division	Fines	Field identification procedures	Group symbols N.	Typical names	Information required for describing soils	Laboratory classification criteria	
COARSE GRAINED SOILS More than half of material is **larger** than No. 200 sieve size is (The No. 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size (For visual classifications, the 1/4" size may be used as equivalent to the No. 4 sieve size.)	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.	GW	Well graded gravels, gravel-sand mixtures, little or no fines.	Give typical name, indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information, and symbol in parentheses.	Use grain size curve in identifying the fractions as given under field identification. Determine percetages of gravel and sand from grain size curve. Depending on percentage of fines (fractin smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% GW,GP, SW, SP; More than 12 % GM, GC, SW, SC; 5% to 12% **Borderline** cases requiring use of dual symbols.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$ Between one and 3
			Predominatly one size or a range of sizes with some intermediate sizes missing.	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.			Not meeting all gradation requirements for GW
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.		Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are **borderline** cases requiring use of dual symbols
			Plastic fines (for identification procedures see CL below).	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.			Atterberg limits above "A" line or PI greater than 7
	SANDS More than half of coarse fraction is smaller than No. 4 sieve size	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	SW	Well graded sands, gravelly sands, little or no fines.	EXAMPLE:		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$ Between one and 3
			Predominatly one size or a range of sizes with some intermediate sizes missing.	SP	Poorly graded sands, gravelly sands, little or no fines.	**Silty Sand,** gravelly; about 20% hard, angular gravel particles ½ inch maximum size; rounded and subangular sand grains coarse to fine, about 15 % non-plastic fines with low dry strength, well compacted and moist in place, alluvial sand; (SM).		Not meeting all gradation requirements for SW
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for idendification procedures see ML below).	SM	Silty sands, poorly graded sand-silt mixtures.			Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are **borderline** cases requiring use of dual symbols
			Plastic fines (for identification procedures see CL below).	SC	Clayey sands, poorly graded sand-clay mixtures.			Atterberg limits above "A" line or PI greater than 7

IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE

Major division	Division	Dry strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group symbols N.	Typical names	Information required for describing soils
FINE GRAINED SOILS More than half of material is **smaller** than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to the naked eye)	SILTS AND CLAYS Liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very vine sands, rock flour, silty or clayey fine sands withg slight plasticity.	Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, color in wet conditions, odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses.
		Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	
		Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity.	
	SILTS AND CLAYS Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	For undisturbed soils add information or structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
		High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays.	EXAMPLE:
		Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity.	**Clayey silt**, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes, firm and dry in place, loess; (ML).
HIGHLY ORGANIC SOILS		Readily identified by color, odor, spongy feel and frequently by fibrous texture.			Pt	Peat and other highly organic soils.	



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

N. **Boundary** classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For examle GW-GC, well graded gravel-sand mixture with clay binder.
N. All sieve sizes on this chart are U.S. Standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately $\frac{1}{64}$ inches. For field classification purposes, screening is not intended; simply remove by hands the coarse particles that interfere with the test.

DILATANCY (Reaction to shaking)

After removing portions larger than No. 40 sieve size, prepare a pot of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pot in the open palm of one hand and shake horizontally, stricking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pot which changes to a livery consistancy and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pot stiffens and finnally it cracks or crumbles. The rapidity of appearance of water during shaking and of its diappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickes and most distinct reactoin whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

DRY STRENGTH (Crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quality of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sand and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

TOUGHNESS (Consistency near plastic limit)

After removing particles larger that the No. 40 sieve size, a specimen of soil about one-half inch cube in size is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface of between the palms into a thread about one-eight inch in diameter . The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticitiy, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occure below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

ADOPTED BY: CORPS OF ENGINEERS AND BUREAU OF RECLAMATION-JANUARY 1962

IO3-D-347

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GEOLOGICAL REPORT

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Ref No.: 5034-01

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EXECUTIVE SUMMARY

The subject property is an oceanfront site proposed for remodeling of an existing single family residence. The roughly rectangular-shaped property consists of one parcel totaling approximately 1.48 acres. The building site elevation is approximately 50 feet above sea level, resting on a graded building pad comprised of artificial fill.

The subject property lies in a highly seismically active region. No active faults are known to cross the property. The San Andreas Fault has the potential of producing a Richter Magnitude 7.3 earthquake, and the nearby Palo Colorado-San Gregorio fault has the potential to produce a maximum credible earthquake (MCE) of Richter magnitude 7.3, in the next fifty years. Should any of these events occur, it will probably generate moderate to severe ground shaking at the property.

The site is comprised of a thin mantle (< 10-15 feet thick) of older debris fan deposited unconformably on a marine terrace surface overlaying Mesozoic age diorite (Dibblee, 1999). Remnants of marine terraces are also common along this part of the coast, these indicate a relatively slow rate of wave erosion and landslide movement (California Geological Survey, 2001). The Mesozoic age granitic bedrock appears to be relatively resistant to land sliding, compared to the Franciscan Complex in the area to the south (California Geological Survey, 2001).

Slopes on the subject property range from gentle to extreme. The subject property has open faces on the western portion, facing the Pacific Ocean. The general area proposed for buildings is approximately 100 feet from the Pacific Ocean and is considered safe from lateral spreading although some ground cracking could occur in the building envelope.

No mapped landslides occur on or around the subject property. The debris fan deposits at the subject property are only minimally susceptible to erosion (Dupre and Tinsley, 1980). Maintaining good vegetative ground cover and controlling drainage will reduce the risk of erosion at the property. The sea cliff on the western edge of the subject property and associated retreat is the subject of the Drainage and Erosion section of this report. Surface drainage should be evaluated by the civil engineer and discharge locations of runoff should be directed away from areas prone to coastal erosion.

We have evaluated geologic hazards that may impact the proposed single family residence within its design life (50 to 100 years), The geologic risks associated with the proposed project are considered ordinary (Joint Committee on Seismic Safety, 1974). Any structures must have a well-designed, site specific, engineered foundation. Such a foundation is also crucial to surviving the strong shaking that could be generated at the subject property during a large-magnitude earthquake and related ground movement.

INTRODUCTION

This report presents the findings, conclusions, and recommendations of a geological investigation for the above named site in Monterey County, California. The geologic report is designed to conform to current guidelines of the California Geological Survey. This report is applicable only to the intended project site.

OBJECTIVE

The objective of the geological investigation is to:

1. Evaluate the general geologic conditions at the proposed site by reviewing existing available published and unpublished geologic maps and studies performed by the United States Geological Survey, California Geological Survey, Monterey County Resource Management Agency, Monterey County Water Resources Agency and other reports and aerial photographs made available.

2. Identify geologic factors which could affect proposed land use.

METHOD AND SCOPE OF INVESTIGATION

The geologic investigation consisted of:

1. Review and compilation of available geologic data. The primary sources of geologic data for this report are Dibblee (1999), California Geological Survey (2001), and Rosenberg (2001).

2. Review of available aerial photographs of the site.

3. A field investigation of the site.

4. Preparation of report. This report was prepared to document the findings, conclusions and recommendations based upon the existing data.

PROJECT SITE AND TOPOGRAPHY

The subject property is located in Big Sur in Monterey County on the western side of Aurora del Mar One (Figure 1). The site is currently developed with a 2584 square foot house and driveway. Remodeling of the house is proposed with minimal changes to the footprint and proximity to the coastal bluff. The property slopes gradually down from Aurora Del Mar with a steep coastal bluff located a minimum of thirty feet west of the existing house.

A land survey was completed May, 2018 by Whitson Engineers, allowing for temporally accurate topographic data to be assessed regarding the position of proposed construction. The survey shows the elevation of the subject property to be approximately 60 feet above mean sea level. Drainage on the property consists of surface runoff and subsurface flow and is controlled by topography and earth materials, with regional drainage generally being west towards the Pacific Ocean. The drainage of the subject property reflects the regional westward trend and is uninhibited geologically.

GEOLOGIC SETTING

Regional Geology

The subject property is located in the Santa Lucia Mountains. The Santa Lucia Range lies between the Pacific Ocean to the west and Carmel Valley to the east, in the central section of the larger Coast Range geomorphic and geologic province. Tectonically, the Santa Lucia Range lies in a portion of the Coast Range known as the Salinian Block. The Salinian Block consists of Cenozoic age sedimentary rocks overlying older metamorphic and igneous rocks. The overall structural grain of the Salinian Block is oriented northwest-southeast. The Santa Lucia Range is fault-controlled, with the orientation of the Range roughly paralleling the orientation of the larger faults, such as the San Gregorio-Palo Colorado, King City-Reliz, and Tularcitos faults. Large and small scale faults and folds are characteristic of the Salinian Block.

Local Geology

The subject property is underlain by granitic rock which is hornblende-biotite quartz diorite in composition and of the Mesozoic age (Figure 6). The diorite is fractured in a northwest trend leading to propagation of coastal erosion along the fracture lineament. The dioritic bedrock under the subject property is overlain by Older Debris Fan deposited unconformably near the base of the bluff on the hornblende-biotite rich quartz diorite marine terrace surface. The unconsolidated to loosely cemented debris fan material is generally medium yellowish brown, slightly clayey sand with gravel. No mapped landslides occur on or around the subject property (California Geologic Survey 2001, Nolan, 2011).

Structural Geology

The subject area lies within the geologic and tectonic unit called the Salinian Block. The Salinian Block is an elongate, northwest trending segment of the Coast Ranges, bounded to the northeast and southwest by the San Andreas and San Gregorio-Sur Nacimiento fault zones, respectively (Greene, 1977). The Salinian Block is characterized by a basement of Paleozoic high grade metamorphic rocks and Cretaceous granitic rocks. Overlying these rocks is a sequence of dominantly marine sediments of Paleocene to Miocene age and nonmarine sediments of Pliocene to Pleistocene age (Page, 1970; Greene, 1977). The faults that partition the Salinian Block (Figure 2&3), have generally been active throughout the latter third of the Cenozoic Era (approximately 15 million years ago to the present). Although these faults are, in general, part of a right lateral strike-slip fault system, they have also controlled the relative vertical movements between smaller structural blocks within the larger Salinian Block. The relative differences in vertical displacement between the smaller blocks have, in turn, controlled patterns of sediment accumulation for the late Tertiary and Quaternary sediments. The down-dropped basement blocks produced structural basins in which a relatively thick, and in some cases complete, Tertiary sequence accumulated. The upthrown basement blocks produced structural highs in which the Tertiary and Quaternary sedimentary deposits are thin or nonexistent.

Tectonic History

The faults that partition the Salinian Block, along with the San Andreas Fault and its eastern branches, form a broad system of inter-related right lateral strike-slip faults that have dominated the tectonic history of western California since the middle of the Miocene Epoch (approximately 15 million years ago). Western California's system of right lateral strike-slip faults represents a segment of the boundary between the Pacific and North American crustal plates. For roughly the past 15 million years, the Pacific Plate has been slipping towards the northwest with respect to the North American Plate (Atwater, 1970; Graham, 1978). This movement is accommodated by right lateral strike-slip faulting. In California, most of the movement has been taken up by the San Andreas Fault system, which has been more or less continuously active since the Mid-Miocene. However, the other faults in this broad system have also experienced right lateral slip, although the movement on any individual fault has been limited in duration and magnitude compared to the San Andreas Fault. Several strike-slip faults cut the Salinian Block, some of which were active in the past and are now inactive, while others probably began slipping later and remain active today. In summary, the composite faulting history of this seismically active region has been extremely complicated.

REGIONAL SEISMICITY

California and the Monterey Area Coast Ranges have been subjected to considerable earthquake activity. The most severe historical earthquakes in the vicinity of the project site were the 8.3 Magnitude 1906 San Francisco event (U.S.G.S. Prof. Paper 993, 1978), the 6.1 Magnitude 1926 Monterey Bay Earthquake (McCrory, 1977), and the 7.1 Magnitude 1989 Loma Prieta event (Plafker and Galloway, 1989). Although California's broad system of strike-slip faults has a complex history, only some of the fault traces present a seismic hazard to the proposed project. Consequently, the project area could experience seismic activity of various magnitudes emanating from one or more of the numerous faults or fault systems within the region. Active faults are those faults having experienced movement within the last 11,000 years (the Holocene period). Active faults may have the greatest potential for disturbance. Potentially active faults have had movement between 11,000 and 3,000,000 years ago (the Pleistocene period) and have had no movement within the last 11,000 years. Inactive faults have had no movement within the last three million years. The major faults are the San Andreas Fault, the Monterey Bay fault zone and it's possible on land extensions that include the Tularcitos-Navy fault, the King City-Reliz fault, the Palo Colorado-San Gregorio fault zone and the Zayante-Vergeles fault (Figures 2&3). These faults are either active or considered potentially active (Buchanan-Bank and others, 1978; Bullis, 1980; Jennings, 1975; Greene, 1977; Hall and others, 1974; Burkland and Associates, 1975). Each of the faults is discussed below.

Faults at the Proposed Project Site

Review of published maps (Dibblee, 1999 (Figure 7); Rosenberg, 2001; Greene et al., 1973; Buchanan-Banks et al., 1978) indicates that no faults have been mapped on the subject property. Several large fault systems are present in the area, as well as numerous smaller faults, both named and unnamed. All of these faults are discussed in the following sections (Figure 3).

Aerial Photograph Examination for Faults at the Proposed Project Site

Aerial photographs from 1956 through 2018 were examined for evidence of past faulting on the subject property. No evidence for past faulting was observed in any of the photographs.

San Andreas Fault

The San Andreas Fault typically represents the major seismic hazard in California (Jennings, 1975; Buchanan-Banks and others, 1978). This fault system has experienced right lateral slip movement throughout the later part of the Cenozoic Era (the last 15 million years), and is currently considered very active. The San Andreas Fault is divided into a series of individual segments, each having a characteristic earthquake magnitude, recurrence interval, and slip rate (Sykes & Nishenko, 1984; Lindh, 1983; Hall, 1984; Wesnousky, 1987; U.S. Geological Survey, 1988). There appear to be "characteristic" earthquakes associated with each segment of the fault, and each segment can be expected to experience an earthquake similar in size to others that have historically occurred along the same segment. The portion of the San Andreas Fault closest to the property is the Creeping Section which is located between Pajaro Gap near San Juan Bautista and Parkfield (Sykes & Nishenko, 1984; U.S. Geological Survey, 1988). This segment is approximately 40 miles east of the subject property and is characterized by a high fault slip rate (>3 mm per year) (Wallace, 1990) and persistent micro seismic activity.

The average time between large magnitude earthquake events is referred to as recurrence time. The average recurrence time between earthquakes on the North Coast, San Francisco Peninsula and Southern Santa Cruz Mountains segments is summarized in Table 1. The average expected recurrence time is directly related to the magnitude of the "characteristic earthquake". The longer the average expected recurrence time, the larger the magnitude of the characteristic earthquake. The maximum earthquake for the Southern Santa Cruz Mountains Segment, the portion of the San Andreas Fault System closest to the property, is a Magnitude 7.0. The 1906 rupture section, approximately 30 miles northeast of the subject property, has a maximum Magnitude of 7.9. A Magnitude 6.5 is the maximum earthquake for the Creeping Segment.

TABLE 1
SAN ANDREAS FAULTS
RECURRENCE TIMES AND CONDITIONAL PROBABILITIES OF EARTHQUAKES
From USGS Working Group, 1990

Fault Segment	Date of Most Recent Event	Expected Magnitude	Expected Recurrence Interval (years)	Level Of Conditional Reliability	Level of Reliability (A) being most)
North Coast	1906	8	201-281	0.02	B
San Francisco Peninsula	1906	7	128-188	0.23	C
Southern Santa Cruz Mountains	1989*	7	84-100	0.01	B
Creeping Segment	1966	6	20-30	0.30	A

The probability of a large (Magnitude 7.0 or greater) earthquake occurring on the various segments of the San Andreas Fault has been estimated using a time-dependent increase in earthquake probability model (Plafker and Galloway, 1989; Lindh, 1983; Sykes and Nishenko, 1984; U.S. Geological Survey, 1988; Nishenko, 1989). This model is based upon the assumption that the potential for a large earthquake on a segment is initially small following a large earthquake and increases as a function of time. Prior to the 1989 Loma Prieta Earthquake, the U.S. Geological Survey (1988) predicted a 20 percent probability of a Magnitude 7.0 earthquake occurring on the San Francisco Peninsula segment and a 30 percent probability of a 6.5 Magnitude earthquake on the Southern Santa Cruz Mountains sub segment between 1988 and 2018. The 1989 Loma Prieta Earthquake probably relieved some stress along the portion between San Juan Bautista and San Jose. Because some stress has been released along this portion of the fault, it is considered likely that the probability of an additional large magnitude earthquake (Magnitude greater than 6.5) in the next 30 years along this segment has been considerably reduced. The probability of a large magnitude earthquake on the North Coast segment of the San Andreas Fault, however, has most likely increased. The effect of the Loma Prieta earthquake on the Creeping Section is not known. The previous discussion applies only to large magnitude earthquakes capable of rupturing the entire fault segment. Small magnitude earthquakes can occur more frequently.

The maximum credible earthquake (MCE) is the largest magnitude earthquake a fault can generate within the presently understood tectonic environment, and is typically higher than the maximum probable earthquake (MPE). The likelihood of a Magnitude 8.0 (MCE) occurring on the San Francisco segment or the Southern Santa Cruz Mountain sub segment is considered very low (U.S. Geological Survey, 1988; U.S. Geological Survey, 1990). The foregoing data suggests the project area should have incorporated into the planning a large Magnitude earthquake (7.5 or higher) along the Southern Santa Cruz Mountains segment of the San Andreas fault during the next fifty years. The data also suggests an extreme event of Magnitude 8.0 or higher is unlikely within the next fifty years.

The inexact science of probabilistic modeling of large magnitude earthquakes is currently being researched, analyzed and modified. The probabilities listed in the report and summarized in Table 1 are based on data collected prior to and since the Loma Prieta Earthquake. This event reduced the likelihood of seismic activity on the Southern Santa Cruz Mountains while increasing the likelihood of earthquakes on other segments.

Palo Colorado-San Gregorio Fault

The main trace of the Palo Colorado-San Gregorio fault (hereafter referred to as the San Gregorio fault) is located approximately 1.16 mile offshore west of the of the subject property (Greene et al., 1973). This fault is oriented sub-parallel to the San Andreas Fault and stratigraphic offsets across the fault demonstrate right lateral strike-slip motion. The San Gregorio fault is considered highly active. Throughout its length, the San Gregorio fault zone shows stratigraphic evidence of late Pleistocene to Holocene displacement (Clark, et. al., 1984; Weber, et. al., 1979, Buchanan-Banks, et. al., 1978; Graham and Dickenson, 1978; Weber and LaJoie, 1974). In addition, historic seismic activity in the Monterey Bay region may also be attributed to the San Gregorio fault (Greene, 1977; Mitchell, 1928). Hamilton and others (1979) present data showing an average net slip through Neogene time (225 to 1.8 MYBP) of about 0.1 cm/year. They conclude this slow slip rate, with respect to the 1.4 cm/year slip rate on the San Andreas Fault, indicates the San Gregorio fault is not the primary structural element of the translational plate boundary. They further conclude the San Andreas Fault represents the principal plate boundary. More recent research on the slip rate of the San Andreas Fault suggests that the slip rate north of San Juan Bautista may be closer to 1.9 cm per year (Working Group on California Earthquake Probabilities, 1990), which reinforces the interpretation that the San Andreas Fault is the principal plate boundary, rather than the San Gregorio fault.

Greene (1977) uses an empirical relationship between fault half-length and potential earthquake magnitude to suggest the San Gregorio fault zone is capable of Magnitude 7.2-7.9 earthquake activity. Weber and Cotton (1981) present evidence suggesting the recurrence interval for earthquakes producing ground rupture within the San Gregorio fault system is 6,000 years or less. Wesnousky (1986) suggests the recurrence interval of a Magnitude 7.7 earthquake on the San Gregorio fault is about 824 years.

Tuttle (1985) studied seismicity patterns along the San Gregorio fault and noted that certain segments exhibited abnormally low seismic activity. She concluded that the segments from Santa Cruz to San Francisco, and from Monterey to Ragged Point, represented seismic gaps, which she theorized were capable of generating earthquakes of Magnitude 7.2 to 7.4. Tuttle (1985) also observed that the number of Magnitude 4 to 6 earthquakes increased during the twenty year periods preceding the 1926 Monterey Bay (M6.1) and the 1952 Bryson (M6.0) earthquakes.

Rosenberg (1993) noted that four recent earthquakes (Magnitudes 4.6 to 5.2) associated with the southern end of the Ragged Point segment occurred between 1984 and 1991. According to Rosenberg (1993), if Tuttle (1985) is correct in her hypothesis, a Magnitude 6 or larger earthquake is likely in the next decade. Plafker and Galloway (1989) noted a similar pattern of seismicity on the San Andreas Fault before the 1989 Loma Prieta earthquake. Based on the foregoing mapping and analyses, along with calculations performed by EQFault version 3.0, the maximum credible earthquake (MCE) on the San Gregorio fault for the purposes of this report is considered to be a magnitude 7.3.

Active Fault Summary

The subject property is situated in a seismically active region in close proximity to known or suspected active faults. The active San Gregorio fault is within approximately 1.16 mile of the site and is considered to be the most likely source of strong seismic shaking (Cao, et al., 2003). The Palo Colorado fault is situated about 1.16 mile west of the site. This fault is considered to be active (Rosenberg, 2001), and may be connected to the San Gregorio fault but the size and expected frequency of earthquakes on this fault are unknown. The San Andreas fault is considered highly active and the fault most likely to generate a large magnitude earthquake within the next fifty years. Ground shaking parameters associated with an event along the San Andreas or Palo Colorado-San Gregorio should be used for design purposes. Because the numerous minor faults in the region around the project area have not been active during historical time, recurrence intervals for them are difficult to predict. In addition, because earthquake magnitude is directly related to fault length, the effect of these shorter faults will be masked by the San Andreas and Palo Colorado-San Gregorio faults. Based on deterministic methodology, these smaller potentially active or less active faults are not considered to represent a significant seismic hazard to site development relative to the San Andreas and Palo Colorado-San Gregorio faults.

Major Earthquakes

The epicenter of the October 17, 1989, Loma Prieta Earthquake (M=7.1) occurred near the northern end of the San Andreas fault Southern Santa Cruz Mountains sub segment at a depth of eleven miles below the ground surface. This is approximately 60 miles north of the proposed project. The fault plane in this area dips about 70 degrees to the southwest. There was about forty miles of fault rupture at depth. Geodetic data suggests a maximum of 67-inches right lateral motion and 51-inches of vertical thrust motion along the fault zone.

The California Division of Mines and Geology network of accelerographs measured the local ground response during the Loma Prieta earthquake. Accelerations in the vicinity of the Loma Prieta earthquake's epicenter were measured to be between 0.55g-0.64g. The ground accelerations in Monterey were measured at 0.07g and ground accelerations at Lucia in southern Monterey County were measured at 0.06g. Ground motions in Salinas were measured at 0.12g, while ground motions in Moss Landing are estimated to have been 0.25g (Woodward-Clyde, 1989), and ground motions near Gonzales were measured at 0.06g. According to Plafker and Galloway (1989), for a site 60 miles from the epicenter of the Loma Prieta earthquake, such as the subject property, there is approximately a two-thirds likelihood that ground motion during the earthquake would have been approximately 0.10g or less.

The term ‘Maximum Credible Earthquake’ (MCE) has been defined as the strongest earthquake that is likely to be generated along an active fault zone. The magnitude of the MCE is estimated from the geologic character (length, displacement, segmentation) of the fault and the earthquake history of the fault. Special geologic studies are needed, often with detailed field work, to develop the data needed to determine the most accurate MCE, and the results, in the best of studies, are susceptible to an error of about plus or minus 1/4 of a Richter magnitude. A Magnitude 7.9 on the San Andreas Fault or Magnitude 7.3 on the San Gregorio fault approximates the MCE that would generate the most shaking for this site. MCE magnitudes have been used for design purposes since they are independent of time restrictions. Probability approaches to magnitudes using statistical techniques on necessarily limited data do contain statistical error, as well as bias errors due to lack of randomness. For design considerations, the most shaking that can be expected from large nearby faults would be likely to originate from the San Andreas Fault.

SEISMIC HAZARDS

Seismic hazards in the vicinity of the proposed project can be placed in three general categories: (1) surface ground rupture, (2) seismic shaking, and (3) seismically induced ground failure which includes liquefaction. Each of these areas are individually discussed.

Surface Ground Rupture

Surface ground rupture occurs when fault movement breaks the ground surface. In general, fault-related surface rupture occurs most commonly on, or in close proximity to, pre-existing active fault traces. It is therefore imperative to locate site improvements away from, and in particular not straddling, active fault traces. An examination of published maps and reports combined with an analysis of aerial photographs from 1956 to 2018 along with a site visit did not reveal any evidence of a fault trace on the subject property. There is therefore a low probability of fault related surface ground rupture at the proposed project site during the next fifty years.

Seismic Shaking

Ground shaking is the soil column response to seismic energy transmission. Intensity of ground shaking and the potential for structural damage is greatly influenced by local soil conditions. In the event of a large magnitude earthquake on any of the nearby active or potentially active faults, ground shaking at the proposed project will range from moderate to severe.

Although there are several faults capable of generating ground shaking in the proposed project area, the most likely cause of intense ground shaking during the next fifty years will be an earthquake on the San Andreas Fault, or the nearby Palo Colorado-San Gregorio fault system. It is important that all structures be designed in accordance with the requirements set forth by county ordinance and within the Uniform Building Code's conditions (current edition).
The expected on-site ground accelerations shown in Table 2 for the listed earthquakes are based on the Next Generation Attenuation Ground Motion Project equations (NGA, 2008) that relate distance from an earthquake and ground shaking intensity. The results shown are based on equal weighting

between the five Next Generation attenuation relationships and are for the mean and mean plus one standard deviation ground motions.

Table 2: Seismic Shaking Parameters					
Fault and Segment	Expected Earthquake Size	Distance from Site (km)	Recurrence Interval for Expected Earthquake	Expected On-Site Ground Acceleration (g)	
				Mean	Mean + 1 ó
San Gregorio, Southern Segment Rupture	7.0	1.2	540	0.44	0.76
San Gregorio, Multi-segment Rupture	7.4	1.2	1202	0.47	0.80

Site seismic design should consider the expected on-site accelerations listed in Table 2. At a minimum, any new structures should be designed to the seismic design standards of the most current California Building Code in force at the time the project is designed. Redevelopment of the property with a residence designed to the most current California Building Code can be expected to reduce the risk posed to persons and property by seismic shaking.

Lateral Spreading

Lateral spreading is the horizontal movement of soil masses caused by seismic shaking. Usually such movement is towards an open face and occurs along a weakened strata of saturated soils.

The subject area appears generally well-drained and slopes range from gentle to steep. In general, groundwater conditions on the subject property are unlikely to favor a high water table. At present there are some open faces within the subject area where lateral spreading could occur. Care should be taken not to create open faces during any construction in the subject area.

Landslides and Slope Instability Hazards

Landsliding is defined as the downward and outward movement of slope-forming materials composed of natural rock, soils, artificial fills or combinations of these materials (Varnes, 1958). In the region of Big Sur landslides are a common event. This is caused by the steep mountains that drop into erosional coastline. After reviewing aerial photographs and visiting the site we did not identify any evidence of landslides on the coastal bluffs that were close enough to impact the site. California Geologic Survey (CGS, 2001) created landslide maps of Highway 1 and these did not identify any land sliding on or near the project site. The nearest mapped landslides (CGS, 2001) are about 1/2 mile away upslope at an elevation of 400 feet.

Settlement and Differential Compaction

Settlement and differential compaction are the result of a loss of volume resulting from seismic ground shaking. Compaction is more likely in water saturated, low density alluvial material. The most likely areas are paleo-swamps and/or marsh, or strata of fine grained silts and sands. Generally, for this phenomena to occur, the site soils must be of low relative density and be dilatant. The soils at the subject property do not meet these criteria and therefore are not typically prone to such phenomena.

DRAINAGE AND EROSION HAZARDS

Erosion is the removal of surface soil, sediment, and rock by wind, water and ice. Rainfall erosion is the most common type of erosion. Rainfall and runoff can initiate slope wash, gullying, siltation, and sedimentation. Rainfall erosion is a function of climatic conditions, topography, soil erodibility, and vegetation type and coverage. Wind erosion is controlled by the same basic factors as rainfall erosion.

Regional drainage is generally west towards the Pacific Ocean. Drainage on the property consists of surface runoff and subsurface flow and is controlled by topography and earth materials.

COASTAL BLUFF RETREAT

Coastal Bluff Erosion

Our investigation of the coastal bluff erosion hazards have led us to suggest a single set back line for the property to prevent future construction from being subject to coastal bluff erosion and related ocean bluff landslides. This is reasonable as landsliding and erosion are related; in that the presence of landslide deposits can result in high erosion rates and bluff erosion can create landslides.

Coastal Bluff Erosion Rate Study

The coastal bluff erosion study was conducted by analyzing stereographic aerial photos and reviewing published coastal bluff retreat rates in the Big Sur area. The aerial photos included in this study; 1956, 1970, 1994, 2002, 2007, and 2018 were selected for their similar scales and observable details. Figure 4 (Historical Coastal Bluffs: Aerial Photograph Analysis), displays the crests of the historical coastal bluffs outlined against a 2018 aerial photograph as the base map.

This method of measuring sea cliff retreat rates is the most widely employed method for studying coastal erosion. Newer methods involving use of LIDAR imagery and digital techniques have been developed that are valuable in providing an accessible and standardized methodology for studying coastal retreat over large areas (Hapke and Reid, 2007). These new methods are not expected to improve accuracy for small project site studies.

Figure 4 does not show a steady regression of the sea cliffs over time. The sea cliffs seem to move back and forth across the base map. This is caused by radial distortion and variation in viewing angle that is inherent to aerial photography. Distortion is also caused by the differences in the scales of the photographs. As a certain amount of error is associated with this method it is most accurate in areas with moderate to high retreat rates. In these areas the changes in the coastal bluffs locations are easily distinguishable. This lack of evidence for sea cliff erosion indicates that there has been less than moderate retreat rates in this area since 1956. The morphology of the cliff has also not changed significantly during the study period, 1956-2018. This lack of change in the shape of the cliff suggests that there have been no large scale erosional events during the study period.

There has been significant research done by Hapke and Green (2004) and Hapke and Reid (2007) on the erosion rates for the Big Sur section of the California coast. Hapke and Green (2004) estimated erosion rates for the Big Sur coast. The specific site is located approximately at the 67.5 post mile near Study Section 1 outside of the Hapke and Green report (Figure 5). The retreat rate for this section was 12 ± 7 cm/yr (~0.39 ft/yr) and the average erosion rate is 0.12 m/yr (~0.39 ft/yr). However these high average rates of erosion are skewed because the results include a few high retreat rates. As a result of these study sections being so long and covering an area with vast topographic variety we examined the transect locations closest to the property. The data from these locations were interpreted to estimate an average erosion rate of 0.1 feet/yr. The erosion rate of 0.1 feet per year seems reasonable in light of the lack of cliff retreat evident in the aerial photographs covering the period from 1956 to 2018.

To insure the safety of the structures it is necessary to have a safety buffer. We recommend that all construction be setback a minimum of 25 feet from the top of the cliff face. An erosion rate of 0.1 feet/yr of the bedrock amounts to 5.0 feet of erosion in 50 years and 10.0 feet of erosion in 100 years. Consideration of the erosion of the top debris fan sediment indicates that a 25 foot setback is the minimum necessary to account for erosion. We based our analysis of the hazards of landsliding and erosion on conservative expectations. This analysis was qualitative and it is expected that analytical evaluation of slope stability through quantitative slope stability modeling may result in smaller setbacks than those provided here.

It is significant that this study specifically measured average erosion rates for the coastal bluffs. Average numbers are very useful for long-term planning but the actual process of erosion occurs episodically. This means that a large retreat event could account for most of the erosion for in any given area for an interval spanning decades.

THE IMPACTS OF SEA LEVEL RISE AND EROSION RATES

Sea level is dynamic and has varied greatly over Pleistocene time. In part this variation is caused by the occurrence of ice ages. Our sea level is at or near the maximum for the last few million years. This is because we are in between ice ages. The lower sea level during ice ages is caused by existence of continental ice sheets that hold much of Earth's water. The periodic melting and reformation of these ice sheets has caused sea level to rise and fall by as much as 426 feet during the time frame of hundreds of thousands of years.

There has also been a shorter time scale that has shown a gradual rise since the late 1800's. Douglas (1997) asserts that the average rate of this rise is about 1.8 mm (0.07 inch) per year. Recently satellite altimetry has been used to measure sea level, this research has measured an increase of about 3.4 mm per year between 1993 and 2010. Ice sheets and glaciers have been melting, due to global climate change, and have been contributing melt water to the ocean. This climate change has been caused by greenhouse gases being trapped in the atmosphere. The source of these greenhouse gases is the burning of fossil fuels. This makes estimating the rise of sea level complicated and difficult as one has to consider the socioeconomic trends that affect the rate at which these fossil fuels are burned. This causes there to be a large lack of consensus among the scientific community about potential sea level rise over the next century. Vermeer and Rahmstorf (2009) estimate sea level rise of 81 to 179 cm (32 to 70 inches) by 2100. The California Ocean Protection Council has formally adopted these estimates for this time line. These estimates are listed in the table below.

Table 3: Sea Level Rise Estimations

Year		Average of Models	Range of Models
2030		7 in (18 cm)	5-8 in (13-21 cm)
2050		14 in (36 cm)	10-17 in (26-43 cm)
2070	Low	23 in (59 cm)	17-27 in (43-70 cm)
	Medium	24 in (62 cm)	18-29 in (46-74 cm)
	High	27 in (69 cm)	20-32 in (51-81 cm)
2100	Low	40 in (101 cm)	31-50 in (78-128 cm)
	Medium	47 in (121 cm)	37-60 in (95-152 cm)
	High	55 in (140 cm)	43-69 in (110-176 cm)

Rising sea level will increase coastal bluff exposure to storm waves which will accelerate erosion in coastal areas. A study of the California coast and the potential for increase in erosion in coastal areas caused by sea level change was performed by Philip Williams and Associates (PWA, 2009). This study covered an area that stretched from Santa Barbara to the Oregon border. As the study area was large and the scope of the project did not allow for coastal erosion estimates for specific sites. The results of this study were created into GIS shape files where one can distinguish land features and hazard zones. However because of the large uncertainty the authors do not wish anyone to use these to assess the risk at a specific location. Instead these estimates were created based on county. The estimated average and maximum erosion distance in the 2100 for cliffs in Monterey County is 180 m and 400 m.

There is at the present time no established method for calculating the increase in erosion caused by sea level rise at this site. Our coastal erosion estimates contained buffers that should compensate for any increase in erosion rates over the next 100 years.

FLOODING

A registered civil engineer should be consulted for an estimation of flood hazards and risks associated with this project. Such an estimation is beyond the scope of this report.

Tsunamis and Seiches

Tsunamis are inundations by oceanic waves generally generated by seismic events. According to the California Geological Survey (2009), low-lying coastal areas generally less than ten feet above sea level are most susceptible to tsunami inundation. There is no historical record of a tsunami higher than nine feet above sea level occurring in the state of California. Since the building portion of the subject property is located approximately 50 feet above mean sea level, a tsunami of average proportions is not considered a hazard. However, any tsunami must be viewed as a potential hazard and evacuation plans be developed accordingly.

According to the Geotechnical Study for the Seismic Safety Element, Monterey County, seiches (fresh water tsunamis) characteristically do not raise the water level in an inland body of water more than a few feet. A seiche is not considered a relevant hazard at the subject property.

CONCLUSIONS

The geologic risks associated with the proposed project are ordinary and similar to those affecting other sites in coastal Monterey County. To help reduce risks associated with these hazards to ordinary levels, we have made engineering geologic recommendations for the project design. Your project engineers and designers should carefully review our conclusions and recommendations and incorporate them into the project plans. Our recommendations are intended principally to lower the risks posed to habitable structures by geologic hazards to an “ordinary” level of risk. An “ordinary” risk is the level of risk to which structures in similar settings are typically exposed (Joint Committee on Seismic Safety, 1974). Any building must have a well-designed, site specific, foundation. Such a foundation is also crucial to surviving the strong shaking that could be generated at the subject property during a large-magnitude earthquake and related ground movement.

LIMITATIONS

In performing our professional services, we have applied present engineering and scientific judgment and used a level of effort consistent with the standard of practice on the date of this report and the locale of the subject property for similar type studies. CapRock makes no warranty, expressed or implied, in fact or by law, whether of merchantability, fitness for any particular purpose, or otherwise, concerning any of the materials or "services" furnished by CapRock to the client.

This report in no way implies that the subject property will not be subject to earthquake shaking, landsliding, faulting, or other acts of nature. Such events could damage the property and affect the property’s value or its viability in ways other than damage to habitable structures. We have not attempted to investigate or mitigate all such risks and we do not warrant the project against them. We would be happy to discuss these risks with you, at your request. This report does not make any attempt to evaluate appropriate foundation design, and is not a Geotechnical Report or a Slope Stability Investigation. Subsurface soil conditions can vary both vertically and horizontally. Should you have any questions or comments concerning this Geological Report, please contact us at (831) 595-1544.

Sincerely
CapRock Geology, Inc.

Robert Barminski, R.G., C.E.G.
Principal Geologist

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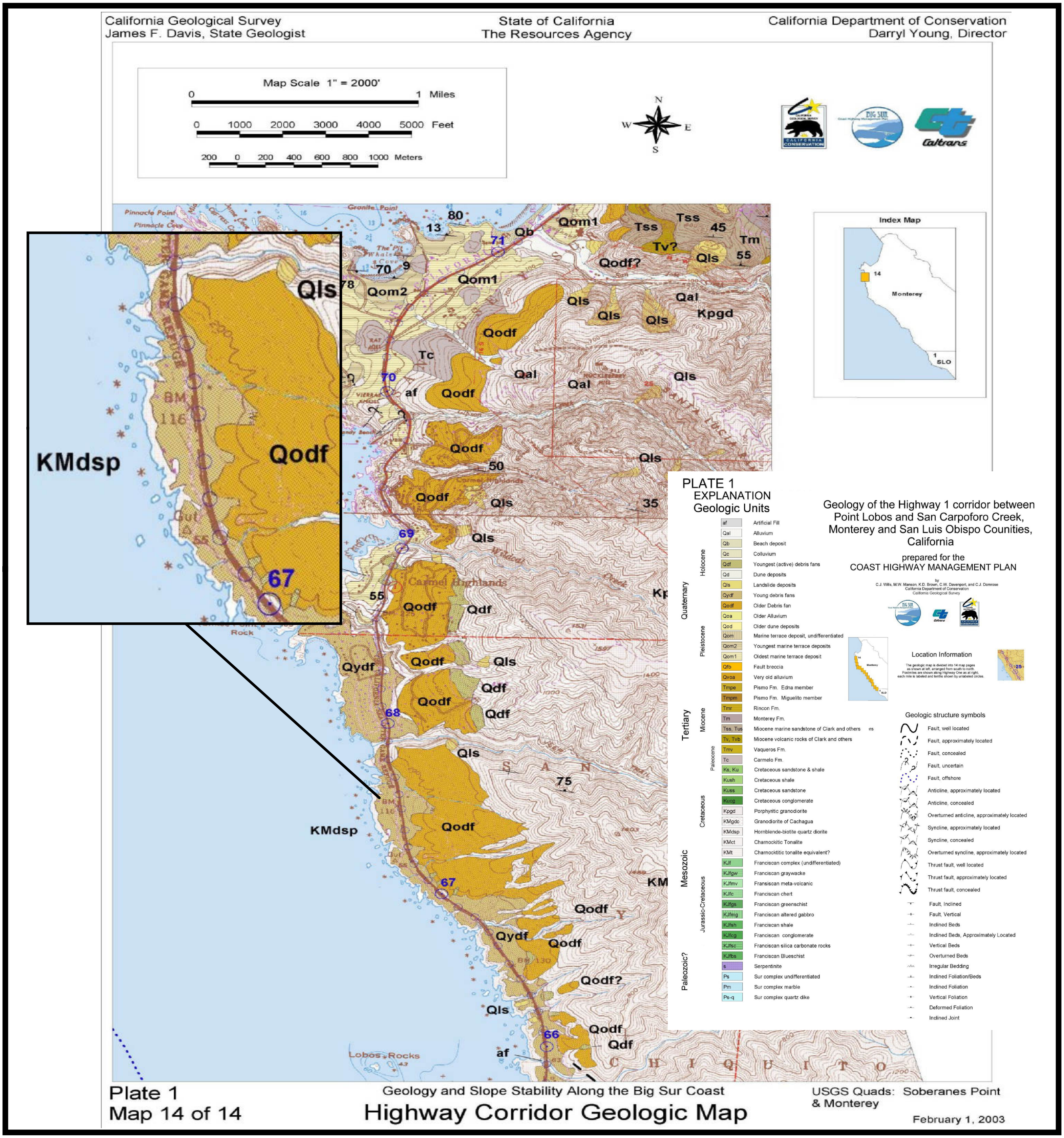
0 1 Mile

Site Location Map

Slawson Residence
30770 Aurora Del Mar
Carmel-By-The-Sea, CA
243-341-005-000

CapRock 2018

CapRock Environmental, Engineering & Marine Geology	Site Location Map Slawson Residence Carmel-By-The-Sea, CA APN#243-341-005-000	FIGURE 1



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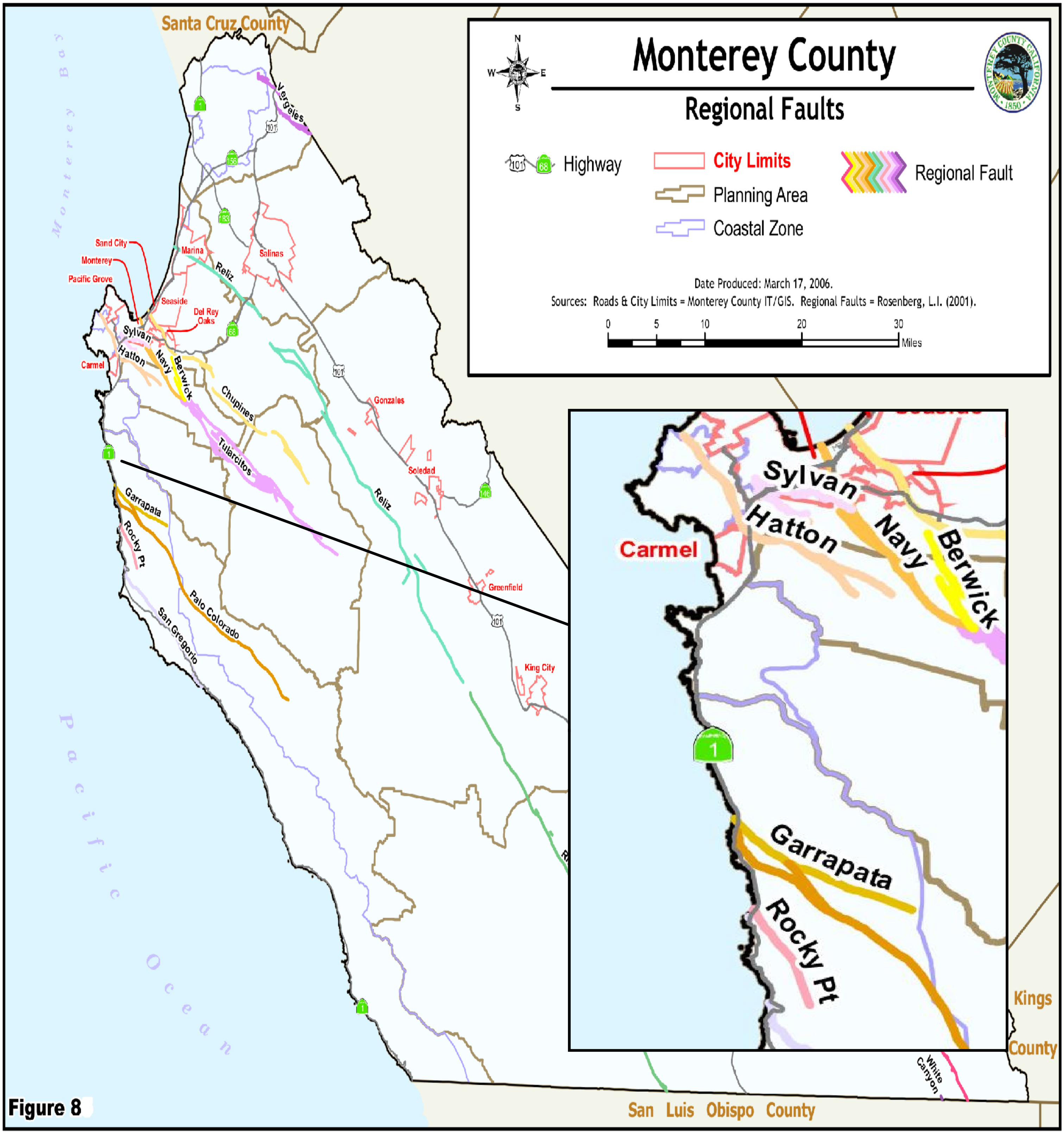
Highway Corridor Geologic Map (2003)

Slawson Residence
Carmel-By-The-Sea, CA
APN#243-341-005-000

FIGURE 2



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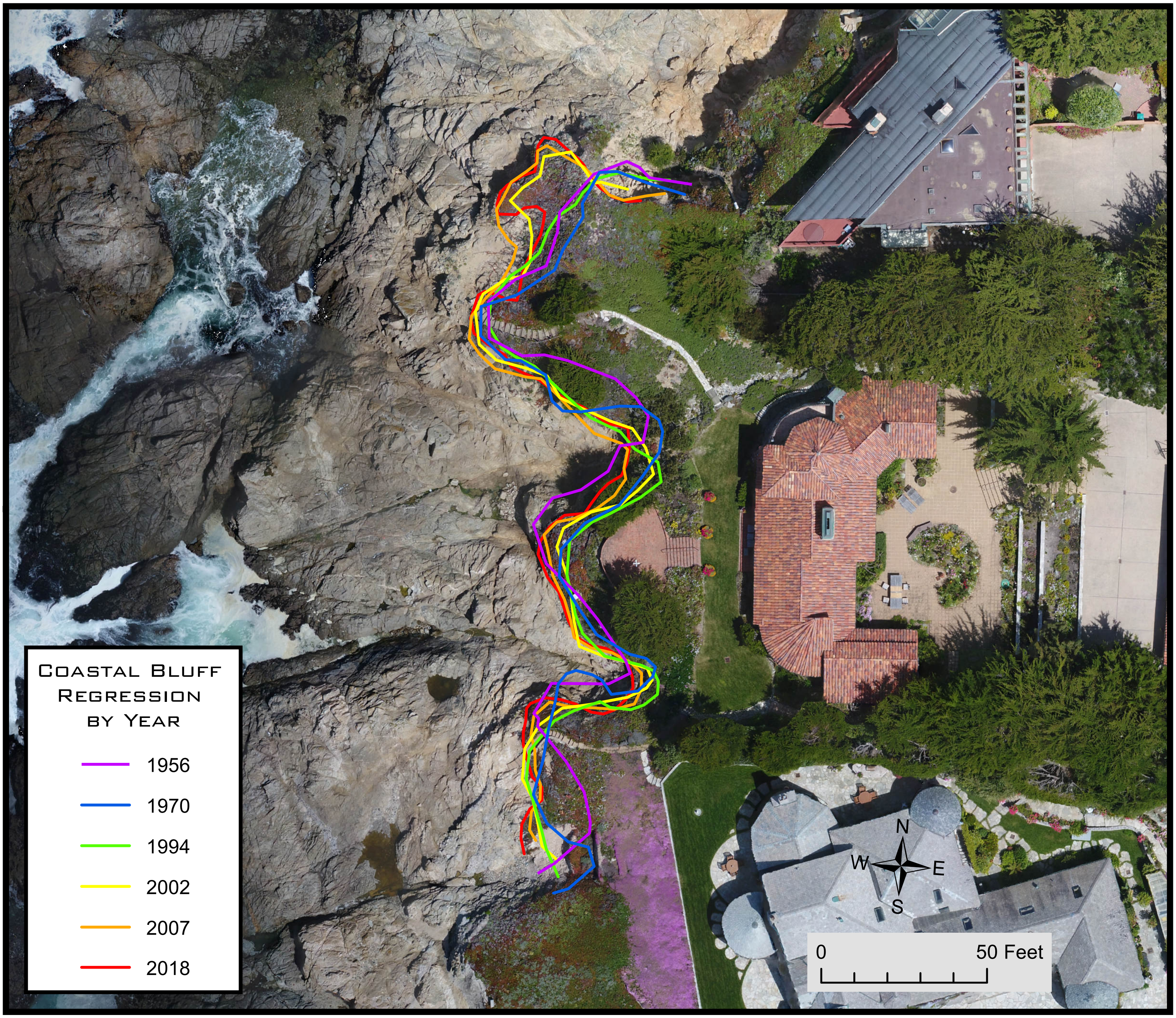
Regional Fault Map (Monterey County, 2016)

Slawson Residence
Carmel-by-the-Sea, CA
APN#243-341-005-000

FIGURE 3



Historical Coastal Bluffs: Aerial Photograph Analysis

1956 1970 1994 2002 2007

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Historical Coastal Bluffs Analysis
Slawson Residence
Carmel-by-the-Sea, CA
APN#243-341-005-000

FIGURE 4

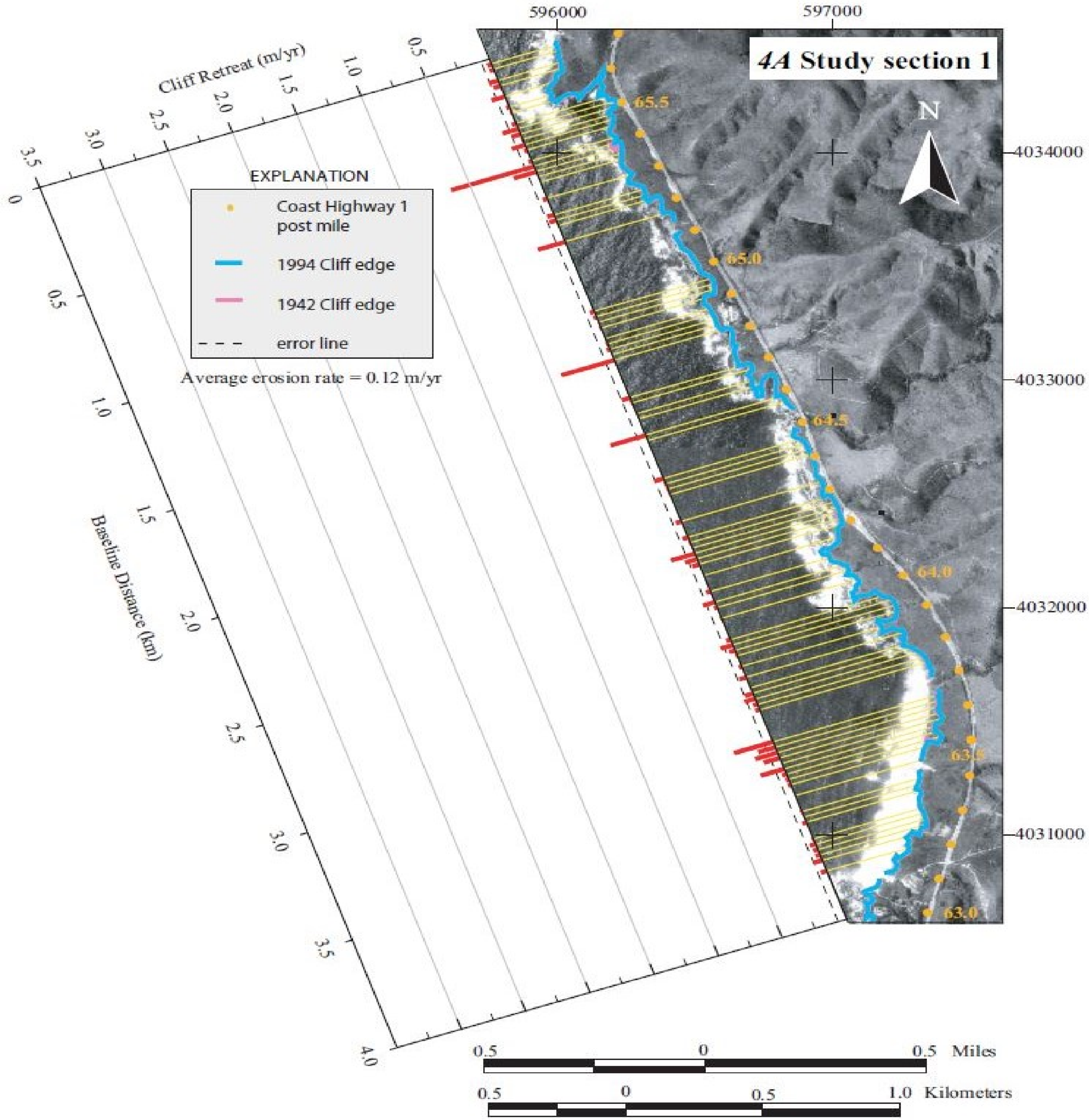
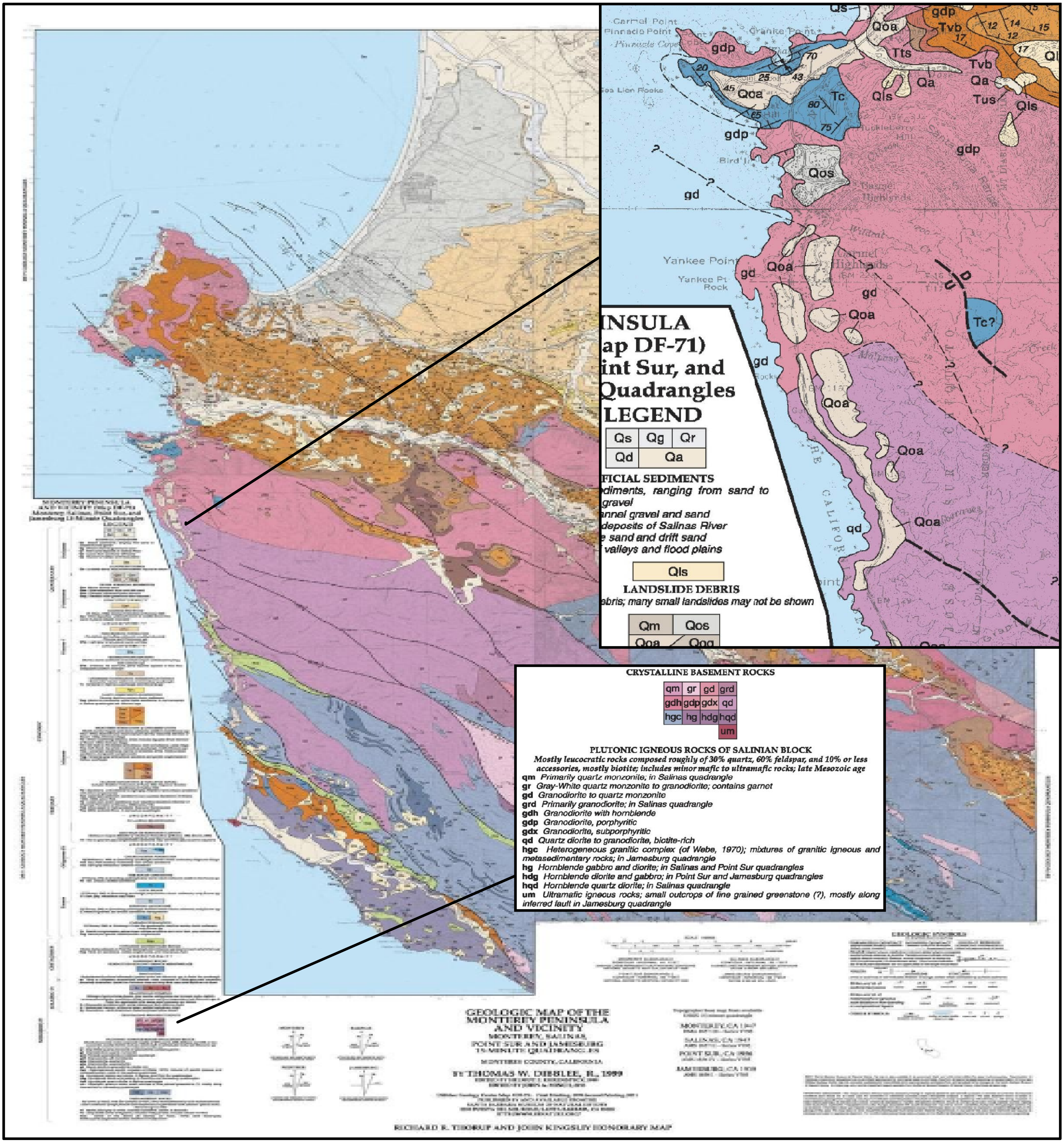


Figure 4A. Cliff retreat rates for study section 1 of the Big Sur coast.

CapRock Environmental, Engineering & Marine Geology	Coastal Cliff Retreat (Hapke and Green, 2004) slawson residence carmel-by-the-sea, ca apn# 243-341-005-000	FIGURE 5



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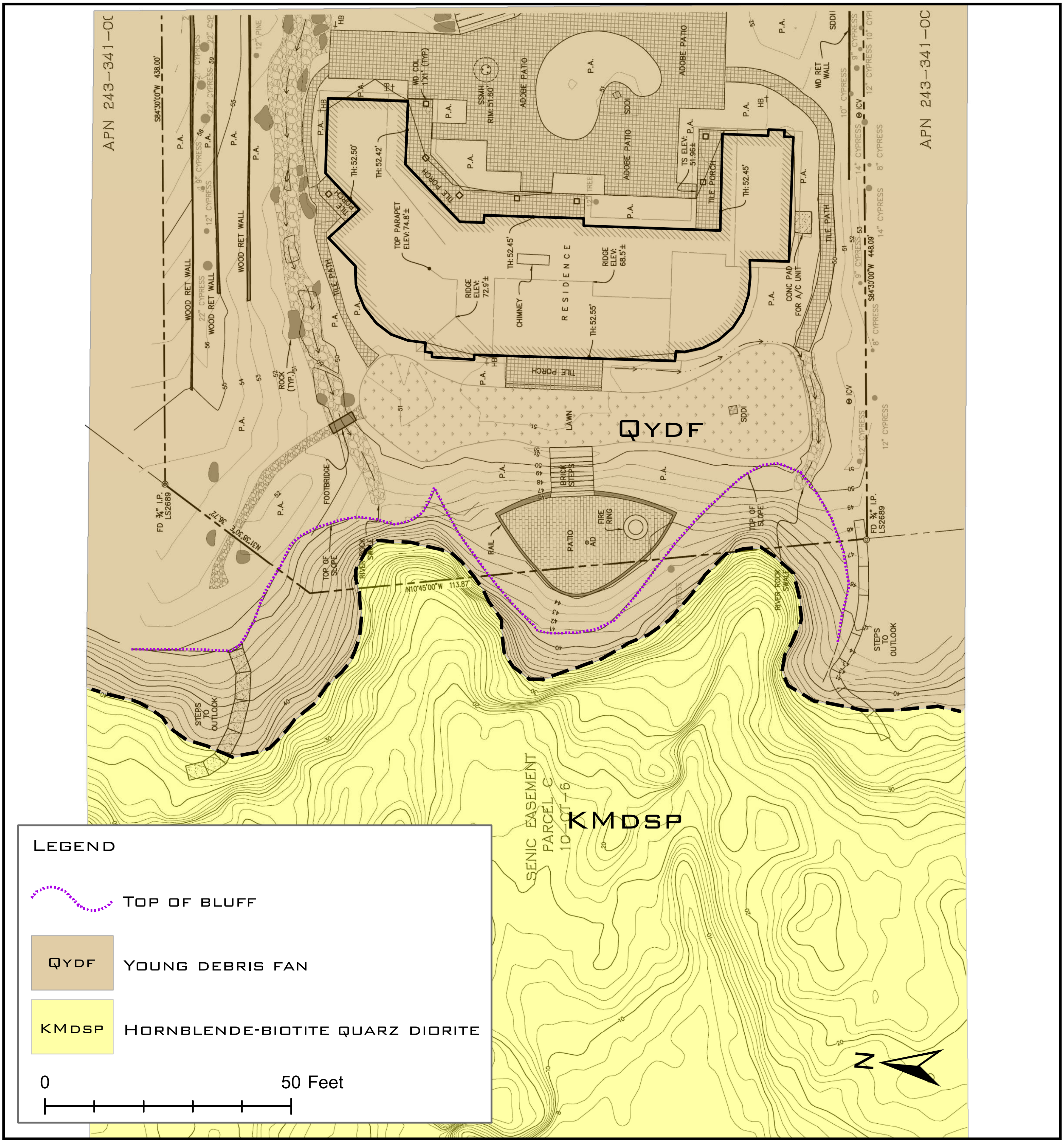
GEOLOGIC MAP OF MONTEREY PENINSULA (DIBBLEE, 1999)
SLAWSON RESIDENCE
CARMEL-BY-THE-SEA, CA
APN# 243-341-005-000

FIGURE 6



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SITE SPECIFIC GEOLOGY MAP

SLAWSON RESIDENCE

CARMEL-BY-THE-SEA, CA

APN#243-341-005-000

FIGURE 7