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GEOTECHNICAL INVESTIGATION
FOR PROPOSED
SEA CLIFF BOUTIQUE MOTEL & RESTAURANT
LOCATED AT
402-404 PASADENA COURT
SAN CLEMENTE, CALIFORNIA

Presented to:

LARRY & JOELLE DUNWALD
402 Pasadena Court
San Clemente, CA 92672

c/o:

Mars Hill Studio, Inc.

Attention:

Anthony P. Massaro, Architect

Prepared by:

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October 30, 2023
Project No. MH446.1

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Site: Proposed Boutique Motel & Restaurant: 402-404 Pasadena Court, San Clemente, California

Executive Summary

Based on our geotechnical study of the site, our review of available reports and literature and our experience, it is our opinion that the proposed re-development/rehabilitation is feasible from a geotechnical standpoint. There appear to be no significant geotechnical constraints on-site that cannot be mitigated by proper planning, design, and utilization of sound construction practices. The following key elements are conclusions confirmed from this investigation:

- A review of available geologic records indicates that no active faults cross the subject property.
- The site is located in the seismically active Southern California area, and within 6 kilometers of the active Newport-Inglewood Fault Zone. As such, the proposed development shall be designed in accordance with seismic considerations specified in the 2022 California Building Code (CBC) and the City of San Clemente requirements.
- The two-lot parcel is stable and suitable for the proposed construction.

SUMMARY OF RECOMMENDATIONS

Min. Design Item	**Recommendations**
Conventional Foundations:	
Footing Bearing Pressure:	1,700 psf - building, continuous; 2,200 psf isolated
Passive Lateral Resistence:	150 psf per foot
Perimeter/Interior Footing Widths:	min. 15 inches with two No. 5 bars top and bottom
Perimeter/Interior Footing Depths:	min. 24 inches below lowest adjacent grade
Coefficient of Friction:	0.25
Soil Expansion:	Low to Medium Expansion (EI = 32 to 71);
Soil Plasticity:	Medium to High Plasticity (PI=17 to 32);
Soil Sulfate Content:	Negligible 0-3 ft.; Severe 7 to 25 ft.; Moderate 40-45 ft.;
Grading Over-Excavation depths:	3 ft. bg for upper level; 1 ft. for lower pad (subterranean)
Soil Maximum Density:	120.0 pcf with an optimum moisture content of 9.5%

Conventional Building Slab:

* Concrete slabs cast against properly compacted fill materials shall be a minimum of 5 inches thick (actual) and reinforced with No. 4 rebar at 18 inches on center in both directions.
* Dowel all footings to slabs with No. 4 bars at 24 inches on center.
* Concrete building slabs shall be underlain by 2 inches of clean sand, underlain by a minimum 15 mil. moisture barrier, with all laps sealed, underlain by 4 inches of ¾-inch gravel.

Mat Slab Foundations (advised for Subterranean Slabs):

Mat foundations:	
allowable bearing pressure:	1,500 psf
passive lateral resistence:	150 psf per foot
mat slab thickness:	min. 12 inches with thickened edges (+ 12 inches)
steel reinforcement:	no. 5 bars @ 12" o.c. each way, top and bottom
coefficient of friction:	0.25
Modulus of Subgrade Reaction:	k_s = 80 lbs/in^3

Basement slabs and subterranean walls shall be waterproofed per design engineer specifications.

Executive Summary *(page 2 of 2)*

Site: Proposed Boutique Motel & Restaurant: 402-404 Pasadena Court, San Clemente, California
Retaining/Basement Wall Design

Since the pad is underlain by competent native materials, over-excavation of the basement slab is not required. The following equivalent fluid pressures may be used in the design of the site basement/retaining walls assuming free draining conditions (clean sand or gravel backfill):

	Equivalent Fluid Pressure		
Condition	Level	2 :1 Slope	1 :1 Slope
Active Pressures	40 pcf	60 pcf	85 pcf
At-Rest Pressures	60 pcf	80 pcf	110 pcf
Passive Pressures	300 pcf	100 pcf *(sloping down)*	

Back drains shall be installed to collect and divert migrating groundwater and discharge to streets.

Shoring Installation Recommendations

Due to the presence of property line and slope setbacks, we recommend shoring for the proposed basement/retaining walls.

The shoring system will consist of steel "H" beam soldier piles and either wood or steel sheet lagging. The steel "H" beam soldier piles should be installed within pre-drilled holes. The soldier piles should not be driven or vibrated into place due to the possible damage that could occur to nearby structures. Once a soldier pile boring is advanced to its recommended depth, a steel soldier pile should be place within the boring and the boring then backfilled.

The caissons shall be a minium 24 inches in diameter. Caissons may be designated for both end bearing and friction. Caissons may be designed for an allowable bearing capacity of 4,500 psf and a skin friction of 500 psf (neglect the upper 3 feet). The bearing value may be increased by 1/3 for wind and seismic forces.

Cement Type

Type V Concrete with a minimum strength f'c of 4,500 psi for the concrete in contact with on-site earth materials.

Seismic Values (per CBC 2022, ASCE 7-16):

Site Class Definition (Table 1611.5.2)	*D - Default*
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.151 g
Mapped Spectral Response Acceleration at 1s Period, S_1	0.418 g
Short Period Site Coefficient at 0.2 Period, F_a	1.2
Long Period Site Coefficient at 1s Period, F_v	1.8
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.381 g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.752 g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	0.921 g
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.502 g
	PGAm = 0.597 g

Attachments:

Figure 1: Site Location Map
Figure 2: Plot Plan
Figure 3: Regional Geology Map
Figure 4: Site Geology Map
Figure 5A & 5B: Geologic Cross Sections
Figure 6: Seismic Hazards Map

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LARRY & JOELLE DUNWALD
402 Pasadena Court
San Clemente, CA 92672

cc: Anthony P. Massaro, Architect, Mars Hill Studio, Inc.

Subject: **GEOTECHNICAL INVESTIGATION**
FOR PROPOSED *SEA CLIFF BOUTIQUE MOTEL & RESTAURANT*
LOCATED AT **402-404 PASADENA COURT**
SAN CLEMENTE, CALIFORNIA

Dear Team,

In accordance with your request we have completed our Geotechnical Investigation of the above referenced site located in the City of San Clemente, California. This investigation was performed to determine the site soil conditions and to provide geotechnical parameters for the proposed re-grading and construction at the subject site.

It is our understanding that the proposed commercial re-development shall include the construction of a commercial Motel & Restaurant development with attached subterrean parking, retaining walls, caissons/lagging, and associated improvements.

This opportunity to be of service is appreciated. If you have any questions, please call.

Very truly yours,

EGA Consultants, Inc.

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GEOTECHNICAL INVESTIGATION
FOR PROPOSED
SEA CLIFF BOUTIQUE MOTEL & RESTAURANT
LOCATED AT
402-404 PASADENA COURT
SAN CLEMENTE, CALIFORNIA

INTRODUCTION

In response to your request and in accordance with the City of San Clemente Building Department requirements, we have completed a preliminary geotechnical investigation at the subject site located at 402-404 Pasadena Court, San Clemente, California (see Site Location Map, Figure 1). The 0.41-acre property is located within in the southern end of the "Pier Bowl," mixed-use neighborhood.

The purpose of our investigation was to evaluate the existing geotechnical conditions at the subject site and provide recommendations and geotechnical parameters for site re-development, earthwork, shoring, and foundation design for the proposed re-construction. We were also requested to evaluate the potential for on-site geotechnical hazards. This report presents the results of our findings, as well as our conclusions and recommendations.

SCOPE OF STUDY

The scope of our investigation included the following tasks:

- Review of readily available published and unpublished reports (including a prior Environmental Impact Report by Keeton Kreitzer Consulting, 2004 and geotechnical review report by Peter & Associates, 2001);
- Geologic reconnaissance and mapping;
- Excavation and sampling of three (3) exploratory borings to total depths of up to 50½ feet below existing grade (B-1 through B-3);
- Down-hole geologic logging of the 24-inch diameter boring (B-3) on September 14, 2023;
- Two (2) continuous Cone Penetration Tests (CPT) soundings to a depth of 30 feet below grade (results of the CPT soundings are included herein);
- Laboratory testing of representative samples obtained from the exploratory borings;

- Engineering and geologic analysis including seismicity coefficients in accordance with the 2022 CBC;

- Preparation of this report presenting our findings, conclusions, and recommendations.

GENERAL SITE CONDITIONS

The subject historic property is an irregular-shaped corner and through lot located at 402-404 Pasadena Court within the City of San Clemente, County of Orange, California (see Site Location Map, Figure 1).

The 0.41-acre lot is bound by multi-family residences to the north, and east; by the Driftwood Bluffs Condominiums to the south; and by the Beachcomber Inn and the Amtrak Rail service to the west-southwest. The subject site is setback approximately 100 feet from the top of the coastal bluff (see Figure 1).

The two-parcel lot has a combined area of 18,339 sq. ft. and is legally described as Lots 60, 61 and 62 of Tract No. 785 in the City of San Clemente, California (APN 692-031-04 and 692-031-05).

The site topography consists of a gentle downward seaward slope from the east to west with a total grade change of approximately 30 feet. The site typography is shown in Figures 2 and 4.

The historic and existing property was developed with the existing Spanish Colonial Revival residence, named Vista de Las Olas, in circa, 1927. There are three attached single-car garages located and accessed by Avenida Victoria.

Based on our review of renderings by Mars Hill Studio, the project is designed to preserve and/or rehabilitate the existing residence (402 Pasadena Court) and portions of the garden, fountain, runnel and patio terraces.

The existing site layout and property lines are presented in Figures 1, 2, and 4.

PROPOSED HISTORICAL PRESERVATION AND RE-DEVELOPMENT

Based on our review of the preliminary drawings by the project architect, Anthony Massaro, the proposed re-development shall include and eleven-unit motel with three new buildings, and the preservation of the four-level residence on the southern portion of the lot. Additionally, the proposed motel and restaurant amenities include a meeting room, pool/spa, deck, dining areas and associated improvements. The proposed project will also include a nine-car subterranean garage to be accessed by Avenida Victoria. Specifications for planned basement/retaining walls, soldier piles and lagging are included in this report.

We assume that the proposed structures will consist of wood-frame and masonry block construction or building materials of similar type and load. The building foundations will consist of a combination of isolated and continuous spread footings. Loads on the footings are unknown, but are expected to be less than 2,200 pounds per square foot on the isolated footings and 1,700 pounds per square foot on the continuous footings. If actual loads exceed these assumed values, we should be contacted by the structural engineer to evaluate whether revisions of this report are necessary.

SUBSURFACE EXPLORATION

Our subsurface exploration consisted of the excavation of three (3) exploratory borings (B-1 through B-3) to a maximum depth of 50½ feet below grade and two (2) CPT probes (CPT-1 and CPT-2) to a depth of 30 feet below grade (continuous soil profile). The purpose of the CPT soundings were to obtain in situ lithology and density data. Prior to drilling, the underground detection and markup service (Underground Service Alert of Southern California) was ordered and completed under a USA DigAlert ticket confirmation.

The borings were continuously logged by a geologist from our firm who obtained soil samples for geotechnical laboratory analysis. Representative bulk and relatively undisturbed soil samples were obtained for laboratory testing. Geologic logs of the soil borings are included in Appendix A.

Geotechnical soil samples were obtained using a modified California sampler filled with 2⅜ inch diameter, 1-inch tall brass rings. Bulk samples were obtained by collecting representative bore hole cuttings. Locations of geotechnical samples and other data are presented on the boring logs in Appendix A.

The soils were visually classified according to the Unified Soil Classification System. Classifications are shown on the boring logs included in Appendix A.

The approximate locations of the borings (including those by Peter & Associates, 2001) are presented on Figures 2 and 4.

LABORATORY TESTING

Laboratory testing was performed on representative soil samples obtained during our subsurface exploration. The following tests were performed:

* Dry Density and Moisture Content
  (ASTM: D 2216)

* Maximum Dry Density and Optimum Moisture Content
  (ASTM: D 1557)

* Soil Classification
  (ASTM: D 2487)

* Direct Shear
  (ASTM: D 3080)

* Sulfate Content
  (ACI 318-14, Hach Procedure)

* Expansion Index
  (ASTM: D 4829)

* Atterberg Limits Test
  (ASTM D 4318)

* Consolidation Test
  (ASTM D 2435)

* Grain Size Sieve Analysis
  (ASTM D 1140)

All laboratory testing was performed by our sub-contractor, G3SoilWorks, Inc., of Costa Mesa, California.

Geotechnical test results are included in Appendix B.

SOIL AND GEOLOGIC CONDITIONS

The site soil and geologic conditions are as follows:

Seepage and Groundwater

During the down-hole geologic logging of the boring B-3, slight localized seepage was observed at depth on geologic contacts and interbedded non-cemented sand lenses. However, static groundwater was not encountered in our test excavations to the maximum depth explored of 50½ feet (approximately 32 feet above sea level). According to the Orange County Water District (OCWD), there are no water wells located within the general vicinity of the subject property.

Geologic Setting

Regionally, the project site is located within the western half of the Orange County part of the San Clemente Quadrangle, characterized by rolling hills and canyons and a narrow strip of coastal plain with marine terraces that borders the ocean.

Elevations range from sea level at the beach to more than 1,000 feet. Access to the densely developed coastal plain is primarily by the San Diego Freeway (Interstate 5) and El Camino Real. It is bound by the Santa Ana Mountains to the east, which part of the Peninsular Range Province.

The coastal plain comprises a series of gently-sloped, seaward inclined marine terraces formed by wave-cut bedrock platforms mantled by beach sands. The deposits are regionally distorted by uplift, folding and faulting due to tectonic activity during the Quaternary period. Locally, steeper terrain between the marine terraces represent inter-terrace canyons and seacliffs.

Bedrock of the Mio-Pliocene Capistrano Formation (5 to 10 million years old) includes the wave-cut platforms, seacliffs and deep water siltstone/claystone and interspersed with lenticular sandstone and turbidite sands (sandy submarine channels). The ancient, triangular-shaped marine trough is known as the Capistrano Embayment; bound to the east by the Cristianitos Fault along the Santa Ana Mountains, and to the west by the San Joaquin Hills.

Locally, the site is underlain by a thin mantle of topsoil and Quaternary age alluvial sediments which are underlain by native, older Pleistocene age terrace deposits. Based on available geologic maps the site is underlain by a thin mantle of residual topsoils, slope wash or artificial fill (Af). Below the fill/topsoil, the site is generally underlain non-marine terrace deposits (Qtn) consisting of firm to stiff silty clays and sandy silts. The interfingering non-marine terrace deposits are typically underlain by marine terrace deposits (Qtm) consisting of medium dense to dense sand and silty sand. The terrace deposits are underlain by the Upper Miocene Capistrano Formation (Tc) which in the immediate area consists of massive, diatomaceous, oxidized to non-oxidized, siltstone and claystone with non- to cemented sand interbedding.

The formational deposits are bound on the north by low, rolling coastal hills which are continuous with the Santa Margarita Mountains to the south, in the Camp Pendleton area. Other rock units in the area have been mapped as the Pleistocene Terrace Deposits, the Middle Miocene San Onofre Breccia and the Miocene Monterey Formation (see reference No. 2).

A Regional Geology Map is presented as Figure 3, herein (reference: Robert P. Blanc and George B. Cleveland, CDMG, 1968).

Faulting

A review of available geologic records indicates that no active faults cross or project towards the subject property (see Figure 3).

Seismicity

The seismic hazard most likely to impact the subject site is ground shaking following a large earthquake on the Newport-Inglewood Fault (offshore), or the San Joaquin Hills Blind (Thrust Fault). The governing fault distances and maximum probable magnitudes are listed as follows (reference: USGS 2008 National Seismic Hazard Maps):

FAULT (Seismic Source Type)	DISTANCE FROM SUBJECT SITE (kilometers)	MAXIMUM CREDIBLE EARTHQUAKE MAGNITUDE M_w
Newport-Inglewood (B)	3.9 miles southwest	7.5
San Joaquin Hills Blind Thrust (B)	12.7 miles northwest	7.1
Palos Verdes (B)	19.3 miles northwest	7.7
Whittier-Elsinore (B)	23.0 miles northeast	7.7
San Andreas (A)	55.0 miles northwest	7.8

A 3.9 magnitude earthquake associated with the San Joaquin Hills Blind Thrust Fault occurred on April 23, 2012 at 10:37 a.m. According to the USGS, the epicenter was located approximately 1 mile west of San Juan Capistrano and no significant damage was reported (ref: USGS Earthquake Hazard Program: *http://comcat.cr.usgs.gov/earthquakes/eventpage/ci15139337#summary*).

Additionally, based on Geologic Maps of Orange County, the northerly-trending Cristianitos Fault and graben valley Zone is located in the topographical basin approximately 3½ miles southeast of the subject site. The Cristianitos Fault Zone is potentially active, in lieu of "active".

For design purposes, two-thirds of the maximum anticipated bedrock acceleration may be assumed for the repeatable ground acceleration. The effects of seismic shaking can be mitigated by adhering to the 2022 California Building Code or the standards of care established by the Structural Engineers Association of California.

With respect to this hazard, the site is comparable to others in this general area in similar geologic settings. The grading specifications and guidelines outlined in Appendix C of the referenced report are in part, intended to mitigate seismic shaking.

Based on our review of the "Seismic Zone Map," issued by the State of California, there are no mapped earthquake landslide zones on the site. The proposed development shall be designed in accordance with seismic requirements contained in the 2022 CBC as adopted by the City of San Clemente building codes. A seismic hazards map is presented herein as Figure No. 6, (reference: https://nbgis.newportbeachca.gov/NewportHTML5Viewer/?viewer= publicsite).

Based on Chapter 16 of the 2022 CBC and on Maps of Known Active Near-Source Zones in California and Adjacent Portions of Nevada (ASCE 7-16 Standard), the site shall be designed using the following seismic parameters:

2022 CBC Seismic Design Parameters
(Equivalent Lateral Force Method)
SITE ADDRESS: 402-404 Pasadena Court, San Clemente, CA

Site Latitude (Decimal Degrees)	33.418773
Site Longitude(Decimal Degrees)	-117.617886
Site Class Definition	*D-Default*
Mapped Spectral Response Acceleration at 0.2s Period, S_S	1.151 g
Mapped Spectral Response Acceleration at 1s Period, S_1	0.418 g
Short Period Site Coefficient at 0.2 Period, Fa	1.2
Long Period Site Coefficient at 1s Period, Fv	1.8
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.381 g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.862 g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.921 g
Design Spectral Response Acceleration at 1s Period S_{D1}	0.574 g

Note: EGA Consultants recommends the structural engineer review and confirm associated seismic values for the proposed Commercial development.

In accordance with the USGS Design Maps, and assuming Site Class "D-Default," the mean peak ground acceleration (PGAm) per USGS is 0.597 g. The stated PGAm is based on a 2% probability of exceedance in a 50 year span.

A copy of the site USGS Design Map Summary report provided in Appendix D, herein.

Seismically Induced Landslide Hazard Zones

Based on our review of the State of California Seismic Hazard Zone Map for the San Clemente 7.5-Minute Quadrangle, Orange County, California (1997), the subject building pad is outside any delineated areas of liquefaction/landslide hazard zones. Figure 6, attached shows the applicable seismic hazards (reference: https://nbgis.newportbeachca.gov/ NewportHTML5Viewer/?viewer= publicsite).

Liquefaction

The site is underlain by a dense cap of Quaternary age cohesive sandy clays and sandy silts, and hard bedrock of the Capistrano Formation which are not known to be susceptible to liquefaction. The site elevations range between 72 ft. (northwest and southwest corners) to 109 ft. (at the Pasadena Court-Cazador Lane knuckle) above MSL. Based on the results of our investigation, the site is not considered to have a significant potential for liquefaction.

FINDINGS

Subsurface Soils

As encountered in our test borings, the site is underlain by a thin mantle of fill/topsoil and native earth materials as described from youngest to oldest (also see the Geologic Cross Sections, Figures 5a and 5b):

Fill (Af), Topsoil, or Slopewash

Fill, topsoil and/or slope wash soils were encountered in each of the borings in the upper approximately 1½ feet below grade. The fill soils consist generally of light to dark gray brown, loose to medium dense, fine-grained silty sandy clay, with roots, gravels, and cobbles.

Based on the laboratory results by G3 Soilworks dated August 4, 2023, the project on-site soil maximum density shall be 120.0 pcf with an optimum moisture content of 9.5%. The expansion potential of the upper three feet soils was tested to be low (E.I. = 32) when exposed to an increase in moisture content. Per the Atterberg Test results the clayey sample is "highly-plastic" (PI = 17).

Terrace Non-marine Deposits (Qtn)

Underlying the fill/topsoil materials are dense Quaternary-age non-marine terrace deposits (Qtn). This unit was encountered in each of the test borings (B-1 through B-3) to a depth of approximately 7 to 9 feet below grade. The sedimentary native deposits consist mostly of firm to stiff, fine-grained clayey sand and diatomaceous silt. The expansion potential of the non-marine clays were tested to be medium (E.I. = 71). The non-marine terrace deposits are roughly 7 feet thick in the study area.

Terrace Marine Deposits (Qtm)

Underlying the non-marine terrace deposits at most of the site are marine terrace deposits (Qtm). However, this unit was not encountered in our deep boring B-3 or CPT-1 (located in the lower site elevations). The sedimentary native marine deposits consist of crudely stratified, medium dense to dense, medium- to coarse-grained silty sand. This geologic unit is roughly 14 feet in thickness along the upper elevations of the site.

Oxidized Bedrock of the Capistrano Formation (Tc_{ox})

The native terrace deposits are underlain by the Mio-Pliocene age Capistrano Formation. Bedrock materials consist generally of moderately hard, crudely bedded marine siltstone and claystone (diatomaceous), whin thin fine-grained oxidized sand lenses.

Unoxidized Bedrock of the Capistrano Formation (Tc_{unox})

The oxidized bedrock is underlain by unoxidized bedrock of the Capistrano Formation which was sharply encountered at a depth of 20 feet in our 24-inch diameter Boring B-3. At least 10 (ten) sand layers on the order of 2 to 10-inches thick were encountered within the bedrock below 7 feet in Boring B-3.

Geologic bedding of the bedrock is undulating and generally dips at gentle angles (5 to 12 degrees) to the south and southwest.

GEOLOGIC CROSS SECTIONS

To-scale geologic cross sections spanning the site and running roughly north-south and east-west are presented in Figures 5A and 5B. The section elevations are based on the site survey by RdM Surveying, Inc. The precise limits of the proposed construction, retaining walls, and caissons are not available at this time.

EXPANSIVE SOILS

All foundation and floor slab design shall be in accordance with section 1808.6 of the 2022 CBC using an effective plasticity index of "17 to 32" (high plasticity) and a project soil expansion index of 71 (medium). Based on the laboratory results and the findings of our geotechnical investigation, no additional measures for mitigation of highly expansive or highly plastic soils are warranted.

Note: A large portion of the expansive soils shall be removed to allow for the basement construction.

CONCLUSIONS

Based on our geotechnical investigation of the site, our review of available reports and literature, it is our opinion that the proposed improvements at the site are feasible from a geotechnical standpoint. There appear to be no significant geologic or geotechnical issues on-site that cannot be mitigated by proper planning, design, and utilization of sound construction practices. The engineering properties of the native earth materials encountered on-ste are favorable for site re-development.

As the project is located near the protected coastal lands, proper drainage design is of critical importance and shall be designed by a Licensed Civil Engineer.

RECOMMENDATIONS

The following sections discuss the principle geotechnical concerns which should be considered for proper site re-development. Note: The site elevations range between 72 ft. (northwest and southwest corners) to 109 ft. (at the Pasadena Court-Cazador Lane knuckle) above MSL. The presumed finished floor elevation for the subterranean garage is approximately 73 ft. above MSL.

Earthwork - Upper Pad

Grading and earthwork should be performed in accordance with the following recommendations and the General Earthwork and Grading Guidelines included in Appendix C. It is our understanding that the majority of grading will be in the lower pads (to achieve parking garage grades). In general, it is anticipated that the removal and re-compaction of the upper 3 feet within the outer lying new building footprints (slab-on-grade portion) will be required. For conventional foundations we recommend a minimum 12-inches of engineered fill below the new building footings.

Where feasible, we recommend a 5-ft. envelope be excavated beyond the building limits. The removals should ensure that all old fill and backfill created as part of the previous site use and demolition operations are removed.

Care should be taken to protect the adjacent property improvements. A minimum one foot thick fill blanket should be placed throughout the exterior improvements (approaches, parking and planter areas). The fill blanket will be achieved by re-working (scarifying) the upper 12 inches of the existing grade.

Earthwork - Lower Parking Garage Grades

For the subterranean subgrades, a minimum one foot of compacted fil should be placed. The fill blanket will be achieved by re-working (scarifying) the upper 12 inches of the basement grade. For a more detailed description of the basement slab section, see "Basement Underslab" section below.

Site Preparation

Prior to earthwork or construction operations, the site should be cleared of surface structures and buried obstructions and stripped of any vegetation, trees, and roots in the areas proposed for development. Removed vegetation and debris should then be disposed of off-site. A minimum of 3 feet of the soils below the existing street grade (upper pad) will require removal and recompaction in the areas to receive building pad fill.

Following removals, each excavated area should be inspected by the soils engineer or his designated representative prior to the placement of any fill. Holes or pockets of undocumented fill resulting from removal of buried obstructions discovered during this inspection should be filled with suitable compacted fill.

Fills

The on-site soils are suitable for reuse as compacted fill, provided the soil is free of organic materials, debris, and rock materials larger than four (4) inches in diameter. After removal of any loose, compressible soils, all areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 12 inches, brought to at least 2 percent over optimum moisture conditions and compacted to at least 90 percent relative compaction (based on ASTM: D 1557). If necessary, import soils for near-surface fills should be predominately granular, possess a very low expansion potential, and be approved by the geotechnical engineer.

Lift thicknesses will be dependent on the size and type of equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches. Placement and compaction of fill should be in accordance with local grading ordinances under the observation and testing of the geotechnical consultant. We recommend that fill soils be placed at moisture contents at least 2 percent over optimum (based on ASTM: D 1557).

Backfill Suitability

The on-site soils may be used as trench backfill provided they are screened of rock sizes over 4 inches in mean diameter and any and organic matter. Trench backfill should be compacted in uniform lifts (not exceeding 8 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (ASTM: D1557).

Foundation Setbacks

We recommend that the proposed perimeter foundations be set back from the face of the descending slopes a minimum horizontal distance of 10 feet (measured form the toe of new building footings) to reduce the effects of slope creep. Since deep foundations are proposed (caissons/grade beams), the proposed construction will likely satisfy current building code setback requirements of H/3 where H is the height of the slope.

Slope Creep

Many factors can contribute to a creep condition, such as animal burrows due to rodent activity, inadequate landscaping, poor surface drainage, and soil type. Based on the laboratory results, the upper site soils are not highly expansive. Therefore, a 3-ft. thick creep zone should be anticipated for design of structures near or on-slopes with the condition that surface drainage at the site must be properly provided and the slope must be properly landscaped (such as non-homogeneous, drought-tolerant, deep-rooted plants) and maintained.

Any structure proposed on or near the top of descending slopes should be supported on deepened foundations to mitigate the potential long-term adverse effects of slope creep and to meet the foundation setback requirements.

On-slope structures should be designed for creep loads of 1,000 lbs. per foot of depth for the upper three feet (within the creep zone).

Geotechnical Design Parameters

The following geotechnical recommendations may used in the design of the proposed structures:

Conventional Foundation Design

Structures on properly compacted fill may be supported by conventional, continuous or isolated pad footings. Footings should be a minimum of 24 inches deep by 15 inches wide. At this depth footings founded in fill materials may be designed for an allowable bearing value of 1,700 psf and 2,200 psf (for dead-plus-live load) for continuous wall and isolated pad footings, respectively.

These values may be increased by one-third for loads of short duration, including wind or seismic forces. Continuous perimeter and interior footings should have a minimum width of 15 inches and be reinforced with No. 5 rebar (two at the top and two at the bottom). Reinforcement requirements may be increased if recommended by the project structural engineer. In no case should they be decreased from the previous recommendations.

Lateral Load Resistance - *New Buildings*

Footings founded in fill materials may be designed for a passive lateral bearing pressure of 150 pounds per square foot per foot of depth. A coefficient of friction against sliding between concrete and soil of 0.25 may be assumed.

Interior Grade Beams due to High Soil Plasticity

As stated above, the project effective Plasticity Index is high. Copies of the Atterberg Limit Test results are presented in Appendix "B". Therefore, the proposed new slab and foundations shall be designed with interior grade beams spaced at a maximum distance of *15 feet*. This is the most conservative beam spacing as classified by the 2022 CBC. The slab-on-grade design shall conform with Section 1808.6.2 and 1808.6.3. of the 2022 CBC; as well as the *WRI/CRSI Design of Slab-on-Ground Foundations*.

Slabs-on-grade - *Conventional Option*

Concrete slabs cast against properly compacted fill materials, or approved native material, shall be a minimum of 5 inches thick (actual) and reinforced with No. 4 rebar at 18 inches on center in both directions. The slabs shall be doweled into the footings using No. 4 bars at 24 inches on center. The reinforcement shall be supported on chairs to insure positioning of the reinforcement at mid-center in the slab.

Some slab cracking due to shrinkage should be anticipated. The potential for the slab cracking may be reduced by careful control of water/cement ratios. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to minimize cracking of slabs. We recommend that a slipsheet (or equivalent) be utilized if crack-sensitive flooring is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations.

All living area floor slabs shall comply with the "Capillary Break" section below, (does not apply to basement slab where certified waterproofing upgrade is imperative).

Capillary Break - for Conventional Floor Slab Design

In accordance with the 2022 California Green Building Standards Code Section 4.505.2.1, we provide the following building specification for the subject site:

> Concrete building and basement slabs shall be underlain by 2 inches of washed sand, underlain by a minimum 15 mil plastic membrane (e.g., Stego Wrap), with all laps sealed, and underlain by 4 inches of ¾-inch

gravel. We do not advise placing sand directly on the gravel layer as this would reverse the effects of vapor retardation (due to siltation of fines).

The above specification meets or exceeds the Section 4.505.2.1 requirement.

For waterproofing of below-grade structures see the "Basement Underslab" section below.

Option: Mat Foundation Design

A mat slab foundation system is recommended for the basement (subterranean construction). Mat slabs founded in compacted fill or competent native materials may be designed for an allowable bearing value of 1,500 psf (for dead-plus-live load). These values may be increased by one-third for loads of short duration, including wind or seismic forces. The actual design of the mat slab should be designed by the structural engineer.

MIN. DESIGN ITEM	RECOMMENDATIONS
Mat foundations:	
allowable bearing pressure:	1,500 psf
passive lateral resistence:	150 psf per foot
mat slab thickness:	min. 12 inches with thickened edges (+ 6 inches)
steel reinforcement:	no. 5 bars @ 12" o.c. each way, top and bottom
coefficient of friction:	0.25
Modulus of Subgrade Reaction:	k_s = 80lbs/in^3

Joints in walls and floors, and between the wall and floor, and penetrations of the wall and floor shall be made watertight using suitable methods and materials (e.g. bentonite "water stops").

Basement Underslab

Over-excavation and re-compaction of the basement pad is not necessary (assuming the bottom is approved by the geotechnical consultant of record). We recommend the approved bottom of the basement excavation be overlain by 4 inches of ¾-inch gravel covered by filter fabric. To counter against the effects of migrating nuisance/ perched water we recommend a minimum 4-inch thick weighted slab (a.k.a. "rat" slab) be poured above the gravel and fabric layer.

The waste slab shall be overlain by waterproofing (e.g. "Carlisle Waterproofing Products") which extends up the wall faces, and then overlain by a minimum 4-inch "protection slab" and then overlain by the structural mat slab. We recommend the waterproofing be inspected and certified by a trained expert. The protection slab is crucial in preserving the underlying waterproofing from puncture or damage during construction. Steel reinforcement is not required for the protection slab or the waste slab. We recommend a minimum 4,500 psi concrete pour for the mat slab, to be designed by the project structural engineer.

Waterproofing

Basement wall and slabs shall be waterproofed in accordance with section 1805 of the 2022 CBC. Permanent waterproofing of the basement slab and basement walls is required. Waterproofing shall consist of rubberized asphalt, polymer-modified asphalt, butyl rubber, or other approved materials capable of bridging non-structural cracks (e.g. "Carlisle Waterproofing Products"). Joint in the membrane shall be lapped and sealed in an approved manner. Protection board shall be used to protect the membrane at the face of basement walls, during and after backfilling. Joints and protrusions in walls and floors, and between the wall and floor, and penetrations of the wall and floor shall be made watertight using suitable methods and materials (e.g. bentonite "Water Stops").

The contractors shall strictly follow the manufacturer's recommendations for the for surface preparation and use of water-proofing products.

A third-party waterproofing expert shall be retained to inspect and verify the waterproofing installation.

Cement Type for Concrete in Contact with On-Site Earth Materials

Concrete mix design should be based on sulfate testing with Section 1904.2 of the 2022 CBC. Preliminary laboratory testing indicates the site soils possess moderate to severe sulfate exposure (550 ppm to 2,000 by volume). Test Results are presented in Appendix B.

ACI 318-14 BUILDING CODE (Table 19.3.1.1)
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Water soluble sulfate (SO_4) in soil percent by weight	Sulfate (SO_4) in water, ppm	Cement Type	Maximum water-cementitious material ratio, by weight, normal weight concrete	Minimum fc', normal-weight and light weight concrete, psi
Negligible [S0]	$0.00 \leq SO_4 < 0.10$	$0 \leq SO_4 < 150$	------	-----	-----
Moderate [S1]	$0.10 < SO_4 < 0.20$	$150 < SO_4 < 1500$	II,IP(MS), IS(MS),P(MS) I(PM)(MS), I(SM)(MS)	0.50	4000
Severe [S2]	$0.20 \leq SO_4 < 2.00$	$1500 < SO_4 < 10{,}000$	V	0.45	4500
Very Severe [S3]	$SO_4 > 2.00$	$SO_4 > 10{,}000$	V plus pozzalan	0.45	4500

We recommend Type V cement with a maximum water/cement ratio of 0.45 and a concrete strength f'c of 4,500 psi should be used for concrete in contact with on-site earth materials.

Retaining/Basement Wall Design

To date, the precise limits of the proposed basement and exterior retaining walls are not known. However, we assume shoring will be required along the south side yard and east rear yard boundaries.

All retaining/basement and landscape wall footings will be embedded into competent native materials (bedrock), therefore over-excavation is not required. The following equivalent fluid pressures may be used in the design of the site retaining walls assuming free draining conditions (clean sand or gravel backfill):

	Equivalent Fluid Pressure		
Condition	Level	2 :1 Slope	1 :1 Slope
Active Pressures	40 pcf	60 pcf	85 pcf
At-Rest Pressures	60 pcf	80 pcf	110 pcf
Passive Pressures	300 pcf	100 pcf *(sloping down)*	
Soil Unit Weight	120 pcf		

The above passive pressure values do not contain an appreciable factor of safety. Therefore, the structural engineer should apply the applicable factors of safety and/or load factors during design.

This office shall be contacted to provide additional recommendations if actual conditions are different than those assumed above.

Lateral Pressure - New Retaining Walls founded in competent native materials

A passive earth pressure of 150 pounds per square foot per foot of depth, to a maximum value of 3,500 pounds per square foot, may be used to determine lateral bearing resistance for footings founded in terrace deposits or bedrock. An increase of one-third of the above values may also be used when designing for short duration wind and seismic forces.

The above lateral resistance values are based on footing placed directly against competent native materials. In cases where footing sides are formed, all backfill placed against the footings should be compacted to at least 90 percent of the applicable maximum dry density value.

Seismic Loads

In accordance with Section 1803.5.12 of the 2022 CBC, for design purposes, a seismic earth pressure of 19 pcf (additional equivalent fluid pressure) may be used for the shoring and the basement wall design. For this, the allowable soil pressure may be increased by one-third.

Surcharge Forces

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Foundations may be designated using the allowable bearing, friction, and passive earth pressure found in the Foundation recommendations of the soils report.

Settlement

Utilizing the design recommendations presented herein, the total settlement of the basement and building foundations is expected to be less than 1 inch. The differential settlement between adjacent footings is expected to be less than ½ inch over a horizontal span of 30 feet. It is anticipated that the majority of the footing settlements will occur during construction or shortly thereafter as building loads are applied.

Retaining Wall Backfill Material

It is recommended that a minimum 2-foot thick layer of free-draining granular material (less than 5 % passing the No. 200 sieve) be placed against the back face of the retaining walls. This material should be approved by the geotechnical engineer. This layer of granular material should be separated from the adjacent soils using a suitable geotextile fabric. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thickness to ensure a minimum in-place density of 90 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Retaining Wall Back Drains

The retaining walls shall be provided with water- or damp-proofing in accordance with the architects recommendations. Back drains and chimney drains shall be installed to collect and divert migrating groundwater. As a minimum, each wall shall be drained by placing a 4-inch diameter pipe perforated (faced down) PVC Schedule 40 pipe or approved equivalent, located behind the base of the wall. The pipe shall be covered by ¾-inch crushed rock at a rate of not less than 2 cubic ft. per linear ft. of pipe surrounded in turn by geofabric such as Supac 4NP or equivalent.

All wall backfill shall be compacted to a minimum 90 percent relative compaction in accordance with ASTM D-1557. Wall back drains shall outlet separately and not be combined with area drains.

This office shall be contacted to provide additional recommendations if actual conditions are different than those assumed above. During construction, drainage devices shall be inspected by a representative of EGA Consultants.

Landscape/Retaining Wall Waterproofing

In accordance with Section 1805.3 of the 2022 CBC, the retaining walls shall be sealed and waterproofed using the miradrain and miraclay (i.e. Grace 3000) waterproofing system, or equivalent. Joints in the membrane shall be lapped and sealed in an approved manner. Protection board shall be used to protect the membrane during and after backfilling.

The contractors shall strictly follow the manufacturer's recommendations for the for use of water-proofing products.

Retaining Wall Back Drains Along P.L. *if any*

If applicable, water in the retained earth will be drained into the channel drains at each bay via the miradrain panels which will be attached to the lagging (sealed with miraclay waterproofing).

Due to space constraints along the side yard property lines, the typical gravel encased backdrains pipes should be replaced with the implementation of the miradrain, waterproofing, and outlet drains at each bay (a.k.a. "J-Drains").

SHORING PARAMETERS

The precise limits of the shoring and construction are not yet available. It appears that shoring will be required where temporary construction slopes cannot be created to achieve the basement level grades. In cases where the shoring may need to double as landscape walls, and/or landscape features (e.g. exterior stairs, boundary fences, walls), then those sections should be designed as permanent. The structural engineers should interface with the architect and landscape designer to determine location of exterior elements.

The limits of the shoring soldier piles and temporary excavation layback cuts shall be shown in the future grading plan.

The following recommendations assume the competent native materials are encountered at a depth of 3 feet below grade. The following equivalent fluid pressures may be used in the design of the permanent or temporary shoring:

Caissons

The caissons shall be a minium 24 inches in diameter and embedded a minimum 10 feet into the unoxidized Capistrano Formation bedrock. Caissons may be designated for both end bearing and friction. Caissons may be designed for an allowable bearing capacity of 4,500 psf and a skin friction of 500 psf (neglect the upper 3 feet).

Point of fixity to be determined to the project structural engineer and in conformance with the 2022 CBC.

The maximum clear spacing between steel cages or H-beams should *not exceed* five effective H-beam diameters, sidewall to sidewall.

Caissons which support vertical loads only (i.e. permanent structural features), should be designed to resist an active earth pressure of 40 pcf for the upper 3 feet below the design finish grades (in addition to supporting the vertical loads).

No additional significant surcharge will be added since the slope structures shall be supported by deepened footings embedded into competent bedrock. The fill/topsoil shall be removed in those areas. All excavation bottoms should be inspected/approved by the geotechnical consultant.

To protect adjacent properties we recommend that the annulus spaces behind the lagging be backfilled with a minium 2-sack slurry. The slurry backfill shall be performed as soon as possible during the shoring installation and shall be monitored/documented by the geotechnical consultant.

After the shoring system is in-place, the excavation of the basement may begin. If concrete and slurry is used for backfill, these materials should be allowed to cure prior to excavation of the basement. Care should be taken to ensure that the lagging drops down as the excavation advances. Any gaps in the lagging could cause undermining of the adjacent structures. To prevent caving of the sidewalls, the lagging elements should be forced down either behind the soldier piles or at an appropriate place within the flanges of the "H" and through the existing soils. The slurry materials that were placed within the soldier pile borings may be broken an removed during the lagging process. The lagging elements should not be driven or vibrated into place due to the possible damage that could occur to nearby structures.

These shall be considered minimum requirements and incorporated into the Foundation, Grading, Retaining Wall, and Shoring Plans. The survey monitoring requirements shall be posted in the Shoring Plan by others. This office should review the plans when available.

Active Earth Pressures

For cantilever shoring systems, an active earth pressure (equivalent fluid pressure) of 40 pounds per cubic foot may be considered for the on-site fill and the native soils. It should be noted that under this condition, the movement of shoring H-beams are not restrained so that the soil internal strength can be fully mobilized.

At-Rest Earth Pressure

If the piles are restrained at the top, then an at-rest earth pressure of 60 pounds per cubic foot should be used in design.

Passive Pressure

A passive earth pressure increasing of *300 psf per foot* of width of a shoring H-beam, per foot of depth, to a maximum value of 4,500 pounds per square foot may be used to determine lateral resistance for the piles (*600 psf-ft* may be used for temporary shoring). All soldier piles should extend at least 10 feet into competent bedrock. The passive resistance should be ignored for the upper 2 feet of the H-beams embedded below the lowest cut grade.

This office shall be contacted to provide additional recommendations if actual conditions are different than those assumed above. During construction, all waterproofing and drainage devices shall be inspected by a representative of EGA Consultants.

Soldier Pile Installation Observations

All soldier pile drilling and installation should be observed by the project geotechnical consultant to verify that they are cast against the anticipated geotechnical conditions, that pile excavations are properly prepared, that proper dimensions are achieved, and that proper installation procedures are followed.

Soldier Pile Monitoring by Others

We recommend a minimum of 8 (eight) monitoring points installed by a Licensed Surveying company. At least four of the monitoring points shall be established near each of the side yard property lines on the drilled shoring piles. The settlement monitoring points shall be monitored for horizontal and vertical movement prior and subsequent to the completion of construction, and on a daily basis during the grading and basement construction.

Stability of Temporary Excavations

The shoring layout shall be determined when the construction plans are finalized and made available.

Based on the results of our subsurface investigation, it is expected that the temporary excavation sidewalls will expose dense native materials with favorable shear strength.

Vertical cuts will not exceed 5 feet in height. Based upon the cohesive nature of the subsurface soils, vertical cuts may extend to 5 feet, and laid back (tapered) at an inclination of 1:1 (horizontal to vertical). The temporary excavation parameters shall be shown in the future grading plan.

Temporary unsupported sidewalls constructed at the recommended maximum gradient are expected to remain stable for the duration of the remedial grading operations; however, all temporary slopes should be observed by a representative of the project geotechnical consultant for any evidence of potential instability.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed.

New Fences/Garden Walls

New fences or garden wall footings adjacent to slope edges shall be supported by the deep permanent caisson footings. Other garden wall footings, if any, should be founded a minimum of 18-inches into approved firm materials. To reduce the potential for unsightly cracks due to expansion forces, we recommend inclusion of construction joints at 8-ft to 15-ft intervals.

Surface Drainage

Surface drainage shall be controlled at all times. Positive surface drainage should be provided to direct surface water away from structures and toward the street or suitable drainage facilities. Ponding of water should be avoided adjacent to the structures. Roof gutter discharge should be directed away from the building areas through solid PVC pipes to suitable discharge points. Area drains should be provided for planter areas and drainage shall be directed away from the top of slopes.

The site slopes shall be properly drained and landscaped (such as non-homogeneous, drought-tolerant, deep-rooted plants) and maintained. All interior retaining walls shall be constructed with backdrains encased in gravel and geofabric, overlain by very low to medium expansive soils.

In general, the more the surface water can be controlled, the less surface water will infiltrate the underlying earth materials. All drainage should be controlled and diverted away from slopes and to a suitable discharge.

Review of Plans

The specifications and parameters outlined in this report shall be considered minimum requirements and incorporated into the Grading, Foundation, Shoring, Landscape, Pool/Spa and Shoring plans. This office should review the Plans when available. If approved, the geotechnical consultant shall sign/stamp the applicable Plans from a geotechnical standpoint.

Based on the findings of our geotechnical investigation and our professional experience working on similar sites in the area, the proposed construction will not impact the stability/safety of the subject or surrounding sites.

Geotechnical observations/testing should be performed during all grading operations, including excavations, waterproofing, drain device installments, removals, filling, compaction, and backfilling, etc.

PRE-CONSTRUCTION MEETING

It is recommended that no clearing of the site or any grading operation be performed without the presence of a representative of this office. An on site pre-grading meeting should be arranged between the soils engineer and the grading contractor prior to any construction.

GEOTECHNICAL OBSERVATION AND TESTING DURING CONSTRUCTION

We recommend that a qualified geotechnical consultant be retained to provide geotechnical engineering services, including geotechnical observation/testing, during the construction phase of the project. This is to verify the compliance with the design, specifications and or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated.

Geotechnical observations/testing should be performed at the following stages:

- During ANY grading operations, including excavation, removal, filling, compaction, and backfilling, etc.
- After excavations for footings/grade beams and/or drilling for soldier piles/caissons, if any to verify the adequacy of underlying materials.
- After excavation for retaining wall footings to verify the adequacy of underlying earth materials.
- During/after installation of water proofing for retaining walls, if any prior to installation of sub-drain/backfilling.
- During/after installation of retaining wall sub-drain, prior to backfilling.

- During compaction of retaining wall backfill materials to verify proper compaction.
- After pre-soaking of new slab sub-grade earth materials, prior to pouring concrete.
- Verification of the placement of the slab underlayment prior to pouring concrete.
- Prior to slab pours to ensure proper subgrade compaction, capillary breaks, and moisture barriers.
- Placement of waterproofing at cold joints and penetrations (e.g. bentonitic "Water Stops").
- During backfill of drainage and utility line trenches, to verify proper compaction.
- When/if any unusual geotechnical conditions are encountered.
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Please schedule an inspection with the geotechnical consultant prior to the pouring of ALL interior and exterior slabs (includes waste and protection slabs).

LIMITATIONS

The geotechnical services described herein have been conducted in a manner consistent with the level of care and skill ordinarily exercised by members of the geotechnical engineering profession practicing contemporaneously under similar conditions in the subject locality. Under no circumstance is any warranty, expressed or implied, made in connection with the providing of services described herein. Data, interpretations, and recommendations presented herein are based solely on information available to this office at the time work was performed. EGA Consultants will not be responsible for other parties' interpretations or use of the information developed in this report.

The interpolated subsurface conditions should be checked in the field during construction by a representative of EGA Consultants. We recommend that all foundation excavations and grading operations be observed by a representative of this firm to ensure that construction is performed in accordance with the specifications outlined in this report.

We do not direct the contractor's operations, and we cannot be responsible for the safety of others. The contractor should notify the owner if he considers any of the recommended actions presented herein to be unsafe.

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16. "Preliminary Geotechnical Review, Sea Cliff Condominiums, 402 Pasadena Court, San Clemente, California," by Peter and Associates, dated January 2, 2001.