Appendix B. Geotechnical Update Report

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Mr. Dan Almquist
31861-31871 CAMINO CAPISTRANO QOZB, LLC
31971 Los Rios Street
San Juan Capistrano, CA 92675

DATE: April 30, 2024

PROJECT: 23-157-00

SUBJECT: Geotechnical Update Report, Proposed Hotel and Associated Improvements, Oyharzabal Project, 31861 Camino Capistrano, San Juan Capistrano, California.

Dear Mr. Almquist:

GMU is pleased to present this geotechnical update report for the subject site, which summarizes our subsurface exploration and accumulated geotechnical data and provides our conclusions and recommendations regarding proposed remedial grading and construction within the site.

This report is a "stand alone" report with complete recommendations for the project. It completely replaces all previous reports and establishes GMU as the new Geotechnical Engineer of Record.

Please note that this report has not been prepared for use by other parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

We appreciate the opportunity to work on this project. Please do not hesitate to contact the undersigned if you have any questions regarding any aspect of this report.

Respectfully submitted,

David Hansen, M.Sc., PE, GE 3056
Associate Geotechnical Engineer

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ENGINEERS & GEOLOGISTS

Geotechnical Update Report,
Proposed Hotel and Associated Improvements,
Oyharzabal Project, 31861 Camino Capistrano,
San Juan Capistrano, California

Prepared for

31861-31871 CAMINO CAPISTRANO QOZB, LLC

April 30, 2024

GMU Project No. 23-157-00

INTRODUCTION

This geotechnical update report for the subject site summarizes the results of our supplemental investigation for the hotel and associated improvements proposed within the site located at 31861 Camino Capistrano in San Juan Capistrano, California.

The purposes of this supplemental investigation are to:

- Determine with more certainty the extent, depth and engineering characteristics of the existing fill materials, native older alluvial materials, and bedrock materials within the site,
- Evaluate in greater detail the settlement and engineering characteristics of the fill and older alluvial materials underlying the site through supplemental laboratory testing,
- Provide updated geotechnical recommendations with respect to site clearing, remedial grading, and design and construction of foundations for the proposed hotel structure and associated exterior site improvements proposed within the subject site based on the supplemental investigation.

This report is a "stand-alone" report that supersedes all previous reports and provides updated geotechnical recommendations for proposed remedial grading and foundation design and construction based on the current grading plans.

SITE LOCATION AND DESCRIPTION

The subject site is known as the Oyharzabal Property and is located west of Camino Capistrano between Yorba Street and Forster Street at 31861 Camino Capistrano in San Juan Capistrano, California. The site is bordered to the north and south by adjacent commercial properties and on the west by railroad tracks and then Los Rios Street. The general location of the site is shown on Plate 1.

The eastern side of the site is located at nearly the same elevation as the adjacent street (Camino Capistrano) at an elevation of approximately 109 feet above mean sea level. This portion of the site is occupied by the Casa Manuel Garcia Adobe building and the Yorba Domingo Adobe building. The remaining portion of the site is mainly vacant with just a metal barn structure to the northwest of the Yorba Domingo Adobe building. This portion of the site descends towards the west with elevations ranging from a high of approximately 109 feet above mean sea level along the east side to a low of approximately 98 feet above mean sea level within the west portion of the property. There are medium to large trees along the west and south sides of the site and the ground surfaces are either covered by old asphalt pavement or by a moderate to thick growth of weeds and grasses.

PREVIOUS SITE HISTORY

It is our understanding that the Casa Manuel Garcia and Yorba Domingo Adobe buildings were constructed in the 1840’s and that between 1880 and 1890 several major additions were made to the Casa Manuel Garcia building that consisted of one to two story wood-frame additions. During the 1930s, a small garage on the north side of the Casa Manuel Garcia building was removed and replaced with a larger flat-roofed structure.

Based on our review of historic aerial photos, it appears the existing metal barn structure along with two other barn structures were also constructed within the central and northwest portions of the site in the 1930’s, as they can be seen on an aerial photo dated 1938. In addition to the adobe buildings and barn structures, a portion of an orchard can be seen on this aerial photo within the southern portion of the site. The orchard was removed and replaced with landscaping sometime between 1952 and 1963. It is our understanding that the previous barn structures within the central and northwest portions of the site were demolished and removed within the past year.

PREVIOUS GEOTECHNICAL REPORTS AND LETTERS

In October of 2021, Salem Engineering Group (Salem) prepared the reference (1) geotechnical engineering investigation report for the subject site. Their investigation included the drilling and sampling of five exploratory borings (B-1 through B-5) within the site with a hollow-stem auger drill rig to depths of 5 to 33 feet below the ground surface. Their report was prepared for the relocation of one barn and the construction of a second two-story barn and a single-story office building. Their report found that the site is susceptible to seismic-induced settlement. As a result, they provided the following options for mitigation:

1. Remove the existing soils to a depth of at least 3 feet below the building foundations and then replace with a compacted fill mat reinforced with at least three layers of geogrid. With this option, conventional foundations could be used.
2. Support the buildings on reinforced structural slab foundation systems (i.e. structural mat slab, post-tensioned slab, or stiffened footings with grade beams).
3. Support the buildings on Stone columns.
4. Support the buildings on deep foundations.

The Salem report provided recommendations for the first two options and stated that recommendations for the second two options could be provided upon request.

In March of 2022, the Salem report was reviewed by Geocon West, the geotechnical consultant for the City of San Juan Capistrano (the City). In their reference (2) review letter, Geocon West did not approve the Salem report and required that Salem address the impact of stormwater infiltration on the potential for liquefaction, soil settlement, and soil expansion on the proposed improvements. They also had several comments regarding the proposed mitigation of the site using a compacted fill mat reinforced with layers of geogrid, and a comment regarding the liquefaction analysis.

In August of 2022, Salem prepared the reference (3) written response to the Geocon West comments. Salem responded that the infiltration system should be located at least 10 feet away from property lines and any proposed foundations and with a maximum depth of 10 feet. Salem also responded to the comments regarding the use of a compacted fill mat with layers of geogrid and provided a document supporting this method of mitigation and responded to the comment regarding liquefaction.

In August of 2022, the Salem response letter was reviewed by Geocon West (see Reference 4). Geocon West approved the Salem report and corresponding response letter as a unit.

Subsequent to the approval of the Salem report and response letter, the improvements proposed within the site were significantly changed to include two on-grade buildings and four buildings and a pool and spa constructed on top of a podium structure with a parking garage on the lowermost level. In addition, an infiltration system was proposed below the parking garage slab within the podium structure. As a result, GMU reviewed the latest plans for the subject site and prepared the reference (5) preliminary update report that provided preliminary recommendations for remedial grading and for the design and construction of proposed improvements. This report supersedes our reference (5) preliminary update report.

TRANSFER OF GEOTECHNICAL CONSULTANT OF RECORD

This serves as notification to the City of San Juan Capistrano that our firm, GMU – Engineers and Geologists (Hereafter "GMU"), Inc., has been retained to serve as the Geotechnical Consultant of Record with respect to all future geotechnical work within our purview to be performed within the subject site located at 31841 to 31871 Camino Capistrano in San Juan Capistrano, California.

In accepting this responsibility, we have reviewed the previously summarized reference (1) geotechnical report and reference (3) response letter for the subject site prepared by Salem Engineering Group and the current reference (6) preliminary grading and drainage plans prepared by C3 Civil Engineering and we generally concur with the findings, conclusions, and recommendations of the previous work with exception of the additional or revised recommendations provided in this report.

As of the date of this letter, GMU Geotechnical, Inc. should be considered as the Geotechnical Consultant of Record for future services to be provided for this project that fall within our purview.

PLANNED IMPROVEMENTS AND DESIGN GRADING

Based on our review of the reference (6) Upper Level Precise Grading Plan prepared by C3 Civil Engineering; it is currently proposed to protect in-place the two historic Adobe buildings along the east side of the site. These buildings are designated as Buildings 1 and 6 on the plans. To the west of the Casa Manuel Garcia Adobe building (Building 1), a new high-ceiling glass and steel-frame "atrium" building that will serve as a covered dining area will be constructed approximately 6 inches away from Building 1 and will be designated as Building 1A.

To the west of the Yorba Domingo Adobe building (Building 6), a new three-story hotel will be constructed and will be designated as Building 5. This hotel will consist of two wood and/or steel-frame buildings separated by a breezeway. The building floor slabs will be constructed on-grade at elevations of approximately 110 to 111 feet above mean sea level. The west side of Building 5 will be constructed immediately adjacent to the podium structure discussed below.

To the west of the Building 5 hotel buildings, a podium structure will be constructed that will consist of a concrete parking garage structure at the lowermost level and three 2- to 3-story hotel buildings, a barn/restaurant structure, and a pool and spa at the upper podium level. The buildings proposed within the upper level of the podium structure are designated as Buildings 2, 3, 4A, 4B, and 4C. Two ramped driveways along the north and south sides of the site will descend and ascend from the lower parking level requiring retaining walls close to the north and south property lines.

A below grade storm water collection and infiltration system will be constructed below the parking garage slab. The system will consist of infiltration in the southwest corner of the slab with traffic rated storage tanks placed at various locations thoughout the slab.

Design grading will consist of fills of only a few inches to approximately 6 feet to reach proposed grades within the area of Building 1A, fills of approximately 1 to 11 feet to reach proposed grades within the area of Building 5, and cuts of approximately 2 feet (largely in the lower western 2/3 of the site) to 6 feet (largely at the eastern edge of the podium structure) to reach proposed grades within the area of the podium structure. Additional cuts will also be required below the lowermost floor of the podium structure as part of corrective grading. Corrective grading recommendations are provided in the Site Preparation and Grading section of this report.

RECENT SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our recent supplemental subsurface exploration within the subject site consisted of the excavation of twelve exploratory test pits (TP-1 through TP-12) with a backhoe within accessible areas of the site to observe depths of existing fill and perform supplemental infiltration testing. We also drilled three (3) additional borings within the site (B-1 through B-3) using a hollow-stem auger drill rig. The borings were advanced to depths of 31.5 to 51 feet to: 1) obtain relatively undisturbed samples of the existing fill and underlying native older alluvial materials and bedrock materials for in-situ density and consolidation testing, 2) collect bulk samples for laboratory testing, and 3) obtain SPT and Cal Mod sample blow counts and perform soil gradation tests and Atterberg Limit tests for liquefaction analysis. Given the deeper depths of the borings and the quality of the drill rig used to obtain the soil samples, a higher level of importance was placed on this data over the existing data collected by Salem.

The locations of our test pits and borings are shown on the attached Plate 2 – Geotechnical Map, which uses the Upper Level Precise Grading Plan as a base map. The logs of our test pits and borings are included in Appendix A-1. The results of our laboratory testing are included in Appendix B-1 – GMU Laboratory Testing.

PREVIOUS SUBSURFACE EXPLORATION AND LABORATORY TESTING

As described previously, a total of five (5) hollow-stem auger borings were previously drilled within the subject site by Salem to depths of 5 to 33 feet. The locations of the previous Salem borings are shown on Plate 2 - Geotechnical Map while the logs of the previous borings are provided in Appendix A-2.

Previous lab testing by Salem included particle size analysis, expansion index testing, corrosion testing, in-situ density and consolidation testing, maximum dry density and optimum moisture content, and shear strength determination. The results of Salem's laboratory testing are included in Appendix B-2 – Salem Laboratory Testing.

GEOLOGIC CONDITIONS

REGIONAL GEOLOGY

The subject site lies within the Capistrano Embayment, which is located at the southeastern-most extremity of the Los Angeles Basin and is bounded by the San Joaquin Hills to the west and the Santa Ana Mountains to the east. These geomorphic features are part of the Peninsular Ranges Geomorphic Province of California and are generally underlain by marine sedimentary rocks of Tertiary age. The Capistrano Embayment was a north-south trending structural trough that existed between 4 and 10 million years ago. Marine waters up to 3,000 feet deep covered the embayment, which was partially filled with fine-grained sands and silts. Subsequent uplift of the embayment and surrounding areas, in addition to global variations in sea level, led to the formation of the gentle hillsides and valleys that exist today. These hills are deeply incised by southerly and southwesterly-flowing creeks such as the Oso Creek, Trabuco Creek, and San Juan Creek.

The site is located within the flood plain of the Trabuco Creek near its confluence with San Juan Creek and south of its confluence with both Oso Creek and Horno Creek. Trabuco Creek eroded channels into the bedrock during a time of lower sea level (late Pleistocene). After the sea level rose to its present level during the Holocene period, the channels filled with alluvium to form the present flood plain. Prior to urban development, the creek meandered back and forth across the flood plain. The creek is now confined to concrete and riprap lined channels to the west of the subject site.

LOCAL GEOLOGY AND SUBSURFACE SOIL CONDITIONS

Based on a review of background geologic, our review of the previous Salem borings, and our recent subsurface exploration, the subject site is underlain by minor amounts of artificial fill and then native old alluvial deposits (Qoal) which, in turn, are underlain by siltstone bedrock materials of the Capistrano Formation (Tc). These subsurface materials are described in detail below.

Artificial Fill (Qaf)

Artificial fill materials were encountered to depths of approximately 1 to 4 feet within our recent borings and test pits. These fill materials were observed to consist predominantly of dark gray to very dark brown, dry to very moist, fine to medium grained, soft to firm sandy silts and medium dense silty sands with some organics and construction debris.

Old Alluvium (Qoal)

Native old alluvial materials underlie the fill and generally consist of dark brown, moist, firm to stiff clays to depths of 4 to 10 feet below the existing ground surface underlain by reddish brown, moist to very moist, loose to medium dense silty sand to a depth of 17.5 feet; brown to reddish brown, moist to very moist, loose to medium dense silty sand with interlayers of medium dense to dense poorly graded sand and firm to stiff sandy clays and sandy silts to a depth of 30 feet; and dark grey, very moist, firm to stiff sandy clay and very dense sandy gravel to a depth of 38 feet.

Capistrano Formation Bedrock (Tc)

The old alluvial materials are underlain by dark grey, moist to very moist, moderately hard to hard, laminated, fresh to slightly weathered siltstone at elevations of 61 to 64 feet above mean sea level within the subject site.

GROUNDWATER

The Seismic Hazard Zone Report for the San Juan Capistrano Quadrangle (CDMG, 2001) indicates that the depth to historically high groundwater within the vicinity of the subject site is 5 feet below the ground surface of the western portion of the site. However, based on actual historic well data within the vicinity, historic high groundwater is much lower.

GMU reviewed data from a monitoring well located to the west of the site and adjacent to Trabuco Creek. The monitoring well has over a decade of groundwater data and indicates groundwater at an elevation of 76 feet above mean sea level or lower. During our recent subsurface exploration, groundwater was encountered at elevations of approximately 82, 83, and 84 feet above mean sea level within Borings B-1, B-2 and B-3, respectively which is approximately 17 feet below the lowest ground surface within the subject site and indicates that the groundwater surface is mirroring (parallel to) the bedrock surface below (See Geologic

Section 1-1'). These measurements were taken following the unusually wet winters of 2022/2023 and 2023/2024 and are considered current seasonally high groundwater levels.

Based on our experience, knowledge of groundwater elevations within the area, and our observations during our recent subsurface exploration, a seasonal high groundwater level which is at least 17 feet below the lowest ground surface within the site is considered justifiable and reasonable for use as a design groundwater table for the site. Thus, groundwater will be at an elevation of 82 feet above mean sea level at the location of the proposed infiltration area.

SEISMIC HAZARD ZONES

According to the referenced Seismic Hazard Zone map for the San Juan Capistrano 7.5-minute quadrangle (CDMG, 2001), the subject site does not lie within an area that is susceptible to earthquake induced landsliding; however, it is located within an area that has been mapped as being possibly susceptible to earthquake-induced liquefaction.

FAULTING AND SEISMICITY

The site is not located within an official Alquist-Priolo Earthquake Fault Zone, and no known active faults are shown on the reviewed geologic maps crossing the site. The site is located approximately 5.6 miles from the Newport Inglewood (N-I) fault, which can generate a maximum earthquake magnitude (Mw) of 7.5. The site is also located approximately 6.7 miles from the San Joaquin Hills Blind Thrust Fault, which can generate a maximum earthquake magnitude (M_w) of 7.1. Given the proximity of the site to these and numerous other active and potentially active faults, the site will likely be subject to earthquake ground motions in the future.

GEOTECHNICAL ENGINEERING FINDINGS

SUPPLEMENTAL INFILTRATION TESTING

Background

Preliminary infiltration testing performed by Salem in their reference (1) report were either too shallow, or performed in areas that were not conducive to the design of the system from a civil engineering perspective. Consequently, supplemental infiltration testing was performed.

Subsurface Investigation

A recent subsurface investigation was performed to observe subsurface conditions below the proposed luxury hotel with a focus on determining materials that were suitable for percolation and design of the infiltration system.

Twelve test pits were excavated to depths of up to 15 feet, with five of the test pits (TP-2, TP-5, TP-10, TP-11 and TP-12) chosen for supplemental infiltration based on observed subsurface geologic conditions. The locations of the test pits are shown on Plate 1 – Geotechnical Map. Logs of the test pits are presented in Appendix A.

Infiltration Soil Characteristics

The proposed infiltration area is underlain by minor (i.e., approximate depth of about 2 feet or less) amounts of undocumented fill which are underlain by native older alluvial materials. The older alluvial materials were observed to consist of sandy clays and then silty sands. A continuous silty sand layer was found starting at depths of approximately 5.5 to 8 feet below the existing ground surfaces (elevations of approximately 91 to 93.5 feet above mean sea level). This layer was selected for infiltration testing.

Groundwater

As described previously, high groundwater is expected to have a seasonally high ground water elevation of 82 feet above mean sea level at the location of the proposed infiltration area within the southwest portion of the podium structure.

Infiltration Testing

The design-level infiltration testing was conducted in general accordance with the open pit falling head procedure for establishing infiltration rates contained in the County of Orange Technical Guidance Document (TGD). After pre-saturating the subsurface soils, a minimum of three trials were conducted at each location and the average infiltration rate over the last trial was used to calculate the observed/unadjusted (pre-factor of safety) infiltration rate.

The table below summarizes the average infiltration rate for the last trial at each test location.

Location	Depth Test Conducted (ft BGS)	Elevation of Test Conducted (MSL)	Avg. Infiltration Rate for Last Trial (in/hr)
TP-2	8.5	90.5	1.13
TP-5	8.5	90.5	1.53
TP-10	8.3	90.7	1.63
TP-11	8.3	90.7	1.75
TP-12	8.7	90.3	2.01

It should be noted that these rates are observed/unadjusted rates, and appropriate safety factors should be applied by the project civil engineer.

Conclusions

- Based on the above findings and recommendations, we conclude that the infiltration system proposed below the southwest portion of the podium structure is feasible from a geotechnical perspective.
- The infiltration system may be designed per the following recommendations.

Geotechnical System Design Recommendations

- The infiltration system shall be located in the southwest quadrant of the hotel site.
- An average unfactored infiltration rate of 1.61 in/hr may be used as a design value for the proposed infiltration system.
- The bottom elevation of the infiltration chamber should be at an approximate elevation of 92 feet above mean sea level, which is within the silty sand layer tested and 10 feet above the estimated seasonal high groundwater elevation.
- If the design bottom elevation of the infiltration chamber is above el. 92 and/or exposes fine grained materials, these materials should be removed and replaced with granular soils.

LIQUEFACTION ANALYSIS

As mentioned previously, the site is located within a zone mapped as having the potential for earthquake-induced liquefaction. In addition, groundwater was encountered at a depth of 17 feet below the existing ground surface and granular soils were encountered below the groundwater. Therefore, liquefaction and related hazards were quantitatively evaluated utilizing the subsurface data from our recent deep borings. Our liquefaction analysis is presented in Appendix C.

Due to potential liquefaction of the subsurface soils, the site would initially be classified as Site Class F. However, ASCE 7-16 Section 20.3.1 provides an exception that, for structures having fundamental periods of vibration equal to or less than 0.5 seconds, site-response analysis is not required to determine spectral accelerations for liquefied soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 of ASCE 7-16 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2. It is our understanding that the proposed buildings will have fundamental periods of less than 0.5 seconds; consequently, standard penetration test values were used to determine the site class. The site N_{30} value was estimated to be between 15 and 50 which corresponds to a “stiff” soil profile. Therefore, Site Class D was used for the determination of the site spectral accelerations.

The Modified Maximum Considered Earthquake (MCE) Peak Ground Acceleration (PGA_M) was determined to be 0.56g in accordance with the ASCE 7-16. For the purposes of our liquefaction analysis, the USGS Interactive De-aggregation Unified Hazard Tool website was used to determine the mean moment magnitude (M_W) for the site. The de-aggregation indicates a magnitude of 6.64 for the site (See Appendix C).

As described previously, a seasonal high groundwater level which is at least 17 feet below the lowest ground surface within the site is considered reasonable for the site and was used in our liquefaction analysis.

SEISMIC-INDUCED SETTLEMENT

Based on our analysis of borings B-1 and B-2, the deeper layers of older alluvium below the anticipated groundwater level do not have the potential to undergo liquefaction-induced settlement. However, some of the soil layers above the anticipated ground water level have the potential for seismic-induced dry sand settlement. The table on the following page summarizes the anticipated settlement amounts.

SUMMARY OF SEISMIC-INDUCED SETTLEMENTS

BORING	DEPTH INTERVAL (feet)	SETTLEMENT TYPE	AMOUNT (inches)
B-1	7 – 17.5	Dry Sand	0.225
B-1	17.5-37.5	Liquefaction	0.00
			Total of 0.225 inches
B-2	7-17	Dry Sand	0.361
B-2	17-38	Liquefaction	0.00
			Total of 0.361 inches

As shown in the table, accumulative total seismic settlements due to both liquefaction and dry sand settlement are estimated to be on the order of approximately ¼ of an inch at the location of B-1 and less than ½ of an inch at the location of B-2. Our liquefaction and seismic-induced settlement analysis is presented in Appendix C.

It should be noted that the above settlements were calculated with a depth weighting factor applied, which in our engineering judgement and experience is technically applicable to the site. If the weighting factor is not applied, the range of seismic settlements would be slightly larger – i.e., ranging from approximately 0.6" to 3.3" for a MCE event. For non-critical building components such as non-structural slabs on grade, flatwork, pavement, etc., use of a higher probability seismic event such as an event with a 10% probability of exceedance in 50 years (475 year return period) is more applicable. For such an event, the use of weighting factor becomes moot as seismic settlements are minor.

STATIC SETTLEMENT

Based on our supplemental subsurface exploration, the existing old alluvial materials consist predominantly of medium dense silty sands with interlayers of firm to stiff sandy clays and sandy silts and medium dense to dense poorly graded sands and sandy gravels. Our laboratory testing (in-situ density and consolidation) indicates that the old alluvial materials are not prone to any hydro-consolidation. However, the alluvial materials are anticipated to undergo compression/settlement under structural loading. Consequently, the podium structure (including

Building 2) and Building 5 will be founded on piles rendering the impact of static settlements to be insignificant. Corrective grading will be limited beneath Building 1A given its proximity to the historic adobe building. Consequently, a mat slab will be required for this building. Pile and mat slab foundations are discussed in more detail in the Foundation Design section of the report.

To further evaluate the potential impact of the infiltration system on the proposed at-grade non-structural slab of the podium structure, two additional consolidation tests with hydro-collapse were performed on the subsurface soils located within the area of the proposed podium structure and are contained in Appendix B (GMU B-1 @ 7.5 feet and 12.5'). The results of these tests indicate a potential hydro-collapse of 0 to 0.01% along with a minimal amount of potential compression. Consequently, it is our opinion that a non-structural slab at grade is suitable for the parking garage as well as buildings 3 and 6 and will not undergo significant distress provided that remedial grading is performed as described in subsequent sections of this report.

SOIL EXPANSION

The expansion potential of the existing fill materials within the site was assessed based on visual classifications, particle size distributions, and expansion index testing performed by both Salem and GMU and were determined to have a low to medium expansion potential (EI values between 31 and 59). Therefore, design parameters based on a moderate expansion potential are provided for the design of the proposed building foundations and slabs constructed on-grade. Expansion test results are provided in Appendices B-1 and B-2.

SOIL CORROSION

Based on the test results for pH, sulfates, soluble chlorides, and minimum resistivity by both Salem and GMU (presented in Appendices B-1 and B-2), the on-site soils should be considered to have:

- Slightly basic pH levels (7.7 to 8)
- Negligible concentrations of sulfates per the ACI 318 Table 4.3.1 (non-corrosive to concrete).
- Elevated chloride contents (severely corrosive to ferrous metals).
- Low to very low minimum resistivity (corrosive to severely corrosive to ferrous metals).

EXCAVATION CHARACTERISTICS

Rippability

The fill and native younger and older alluvial materials underlying the site can be excavated with conventional grading equipment (i.e., backhoes, excavators or loaders). Gravels and some cobbles were observed during our investigation and may be encountered during grading and trenching.

Trenching

We expect that excavation of utility trenches within the existing fill and alluvial materials can be accomplished utilizing conventional trenching machines and backhoes. As mentioned above, gravels and some cobbles will likely be encountered during trenching. Trench support requirements will be limited to those required by safety laws or other locations where trench slopes will need to be flattened or supported by shoring designed to suit the specific conditions exposed.

Volume Change

To aid planning for the anticipated grading, we estimate that the change in volume of on-site soils excavated and placed as compacted fill at an average relative compaction of 90% will result in an average of about 2 to 5% loss in volume.

CONCLUSIONS AND RECOMMENDATIONS

DEVELOPMENT FEASIBILITY

Based on the geologic and geotechnical findings, it is our opinion that the proposed construction shown on the reference (6) Upper Level Precise Grading Plan is feasible and practical from a geotechnical standpoint if accomplished in accordance with the City of San Juan Capistrano grading and building requirements and the recommendations presented herein.

It is also the opinion of GMU that proposed grading and construction will not adversely affect the geologic stability of adjoining properties provided grading and construction are performed in accordance with the recommendations provided in this report.

GENERAL APPROACH

Based on our geotechnical findings and conclusions, the general geotechnical approach to the development is summarized as follows:

- Corrective grading will consist of removing the existing soft, loose artificial fills down to native alluvium.
    - It is envisioned that the entire site will be graded with the required corrective grading occurring first following demolition.
    - The corrective grading will support new fills as well as non-structural slabs for the parking structure, Building 2 and Building 5 as well as exterior flatwork and paving.
    - The below slab infiltration and stormwater storage tanks will be supported by medium dense alluvial deposits existing below the engineered fill placed as part of the corrective grading.

- Pile foundations extending at least 5 feet into bedrock will be used to support the podium structure (including Building 2), Building 5, and the major retaining walls. The parking slab as well as the ground floor slabs for Buildings 2 and 5 will be constructed on-grade and be supported on engineered fill.
- A mat foundation will be used for Building 1A given its proximity to the adjacent historic adobe that will limit the ability to perform corrective grading.

SITE PREPARATION AND GRADING

General

The subject site should be precise graded in accordance with the City of San Juan Capistrano grading code requirements (and all other applicable codes and ordinances) and the recommendations as outlined in the following sections of this report. The geotechnical aspects of future grading plans and/or improvement plans should be reviewed by GMU Geotechnical prior to grading and construction. Particular care should be taken to confirm that all project plans conform to the recommendations provided in this report. All planned and corrective grading must also be monitored by GMU to verify general compliance with the recommendations outlined in this report. All site preparation and grading should be performed in accordance with the City requirements and the recommendations presented in this report.

Demolition, Clearing, and Suitability

Prior to the start of new construction, the existing metal barn and any existing site walls or other structures, including their footings, will need to be demolished and removed from the site. In addition, all significant organic materials such as weeds, shrubs, trees, construction debris, or other decomposable materials should be removed from areas of proposed grading and construction.

The on-site fill materials and older alluvial materials are suitable for use as new compacted fill from a geotechnical perspective if care is taken to remove all significant organic materials and other decomposable debris and any oversize inert material such as cobbles or large concrete fragments greater than 6 inches in diameter. The bedrock materials are only expected to be encountered during the installation of the auger-cast piles.

Cavities and excavations created upon removal of subsurface obstructions, such as tree or shrub root balls, should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment, and then backfilled with properly compacted fill.

GMU should provide periodic observation and testing services during demolition operations to document compliance with the above recommendations. In addition, should unusual or adverse soil conditions or buried structures be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of GMU for corrective recommendations.

Corrective Grading

Podium Structure Foundations: The proposed podium structure will be underlain by deep old alluvial materials that cannot be removed without the need for shoring along the north, west, and south sides of the site. Therefore, the foundations of the podium structure and its retaining walls should be supported on deep foundations that consist of auger-cast piles that extend at least 5 feet into the underlying competent bedrock materials as described in a following section of this report. Remedial grading below the podium structure on-grade parking garage slab will require remedial grading as described below.

Podium Garage Slab: The existing fill and surficial old alluvial materials below the podium garage slab should be removed and recompacted. Removals shall extend a minimum of 2 feet from existing grade. Where existing fills are locally deeper than 2 feet they should be removed and recompacted.

Building 2 Floor Slab: To minimize the potential for adverse total and differential settlement of the Building 2 slab, the existing fill and surficial old alluvial materials should be removed to a depth of 3 feet below the design bottom elevation of the slab. Where existing fills are locally deeper than 2 feet they should be removed and recompacted.

Building 5 Floor Slab: To minimize the potential for adverse total and differential settlement of the Building 5 slab, the existing old alluvial materials should be removed to a depth of 3 feet below the design bottom elevation of the slab.

Building 1A Floor Slab: Because of the location of this structure immediately adjacent to the historic adobe, corrective grading will be limited. Consequently, as described in the FOUNDATION section of this report, a mat slab will be required. Specific limited corrective grading requirements are listed below.

- No corrective grading shall occur within five feet from the exiting historic adobe below the building slab.
- Outside of the 5 feet, corrective grading shall consist of just removing any loose fill soils down to competent soil materials.

Exterior Hardscape and Pavement Areas - The exterior hardscape structure and pavement areas around the new buildings should be over-excavated to a minimum depth of at least 2 feet below proposed hardscape and pavement base course materials and the excavated materials then moisture conditioned (as necessary) to at least 2 percentage points above the optimum moisture content and then replaced as properly compacted fill at a minimum relative compaction of 90 percent.

Major Retaining Walls – Several major retaining walls are planned which are associated with the podium structure and Building 5. These walls should also be supported on piles. Consequently, additional corrective grading for these foundations is not required.

Minor Exterior Site and Retaining Walls – Corrective grading for proposed minor exterior site walls and retaining walls should consist of the over-excavation and re-compaction of the existing soils to a depth of at least 24 inches below the bottoms of the footings.

Lateral Extent of Removals

To provide proper support of all improvements, the lateral limits of the recommended over-excavations should extend horizontally beyond the perimeter edges of the proposed improvements to the same depth as the excavation. However, consideration should be given to the protection of any adjacent onsite or offsite structures. Along the north and south sides of the site, the top of the over-excavation sidewalls should be kept at least 6 inches away from the existing property lines.

FILL MATERIAL AND PLACEMENT

Suitability of Onsite Soils

All on-site soils are considered suitable for use as new compacted fill from a geotechnical perspective if care is taken to remove all significant organic and other decomposable debris, and separate and stockpile rock materials larger than 6 inches in maximum diameter.

Imported Soil Requirements

Based on the precise grading plans, imported soils will be required to bring the site up to proposed grade elevations. The imported soils should not contain any organic materials or decomposable debris and should not contain any oversize inert material such as cobbles or large concrete or brick fragments greater than 6 inches in diameter. Furthermore, the imported soils should generally have a low to low-medium expansion potential (EI<60) and should have low concentrations of sulfates and chlorides and neutral pH values as approved by GMU.

Compaction Standard and Methodology

All existing onsite soils and imported soils to be used as compacted fill, processed in-place, or used to backfill walls and trenches, should be:

- Moistened, dried, or blended as necessary to a minimum of 2 percentage points over the optimum moisture content.
- Placed in 4- to 6-inch-thick lifts for conventional compaction equipment and 8- to 12-inch lifts within trenches where wheel compactors are used.
- Compacted to at least 90% relative compaction as determined by ASTM Test Method D 1557.

In-situ Moisture Content and Material Blending

The existing fill soils and imported soils are expected to have variable moisture contents depending on the season in which work is performed. Recent borings (B-1 and B-2) indicate that soils in the upper 5 feet are significantly over optimum. Existing fill materials and imported soils should be blended prior to being replaced as new fill. Blending and moisture conditioning (i.e., either adding water or drying) will also be required to meet acceptable moisture ranges for sufficient compaction (i.e., minimum 2 percentage points above optimum moisture content).

Use of Oversize Rock

Any existing oversize rock or debris (i.e., cobbles or large concrete or brick fragments) exceeding 6 inches in diameter should not be incorporated into the new fills and should be removed and hauled from the site.

TEMPORARY EXCAVATION STABILITY

During site grading, temporary excavations will be required during the recommended over-excavation and re-compaction of the onsite soils. Trench excavations will also be required for new utility lines.

As shown on Plate 2 – Geotechnical Map and Plate 3 – Geotechnical Section 1-1’, the excavation sidewalls will expose minor amounts of existing fill materials and then old alluvial materials. Based on the anticipated engineering characteristics of these materials, OSHA Type B soil characteristics should be assumed.

From a geotechnical standpoint, excavations into the existing soils may be cut vertical to a height of 4 feet and then those portions of the excavation sidewalls above a height of 4 feet should be cut back at a maximum slope ratio of 1:1, horizontal to vertical.

These temporary excavation recommendations are provided only as general guidelines and all work associated with temporary excavations should also meet the minimal requirements as set forth by CAL-OSHA for Type B soils. Temporary slope and trench excavation construction, maintenance, and safety are the responsibility of the contractor. Other factors that should be considered with respect to the stability of temporary slopes include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures, and weather conditions at the time of construction.

Based on the preliminary grading plans, there appears to be room within most of the site to lay back the sidewalls of the excavations as recommended above. However, it is our understanding that it is desired to create a “Lay Down Area” east of the excavation for the building pad of Building 2 and a construction ramp south of the excavation for the building pad of Building 2. In addition, cuts of approximately 2 to 6 feet will be required directly along the north property line and in close proximity of existing adjacent offsite buildings. Therefore, shoring will be required in these areas. Recommendations for shoring are provided in the following section,

SHORING

General

Recommendations for typical shoring using soldier piles and wood lagging are provided below; however, alternative shoring systems may be required that can be structurally combined with the auger-cast pile system of the property line retaining walls. Recommendations for alternative shoring systems can be provided in a separate letter once the actual type of shoring is determined.

Installation Considerations

The soldier piles may consist of steel H-piles installed within pre-drilled holes. The steel H-piles should be not driven or vibrated into place due to the potential damage to nearby structures due to vibrations. After the H-piles are installed, the drill holes should be backfilled with concrete up to the elevation of the planned excavation bottom. Above the excavation bottom, the drill holes may be filled with 1.5-sack slurry. The drilled shafts for the steel “H” piles should be sufficiently large to allow concrete backfilling around piles to be performed as effectively as possible. Any voids left between the “H” pile and the sides of the hole are expected to reduce the lateral capacity of the soldier pile. To provide an adequate space for concrete backfilling, we recommend that the web height of the steel “H” pile be at least 10 inches less than the diameter of the hole. The concrete and slurry should be poured into the soldier pile excavation from the bottom up using a pump and tremie pipe. The concrete should be lightly vibrated, as necessary, to remove any entrapped air.

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

After the soldier piles have been placed and backfilled, and the concrete and slurry allowed to cure to any strength requirements specified by the structural engineer, excavation of the site may begin. The lagging may be dropped down as the excavation advances or it may be angled and rocked into place. The maximum exposed vertical cut at any given time should not exceed 4 feet. Any gaps in the lagging or behind the lagging should be immediately filled with grout or slurry.

Subdrainage

To prevent the accumulation of water against the future retaining walls, subdrain systems should be installed between the lagging and the retaining walls in one of the following methods:

- If there is sufficient space, a conventional pipe and gravel subdrain system may be installed. Perforated pipe should consist of 4-inch minimum diameter Schedule 40 or SDR-35 PVC pipe with the perforations laid down. The pipe should be encased in a 1-foot-wide column of ¾-inch to 1½-inch open-graded gravel. The open-graded gravel should be completely wrapped in filter fabric consisting of Mirafi 160N, or equivalent. Non-perforated (i.e. solid) outlet pipes should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water. All drain pipes should be installed with suitable positive drainage gradients.

- If a limited area exists between the shoring and the retaining walls, geotextile drain mats such as *Mirafi Miradrain 6000XL*, or equivalent, can be used in lieu of a conventional pipe and gravel drainage system. The drain mats should extend the full height and lengths of the walls, and the filter fabric side of the drain mats should be placed against the backcuts. If room allows, the drain mats may be connected to perforated pipe drain lines. These drain lines should consist of 4-inch minimum diameter Schedule 40 or SDR-35 PVC pipe. The filter fabric on the drain mats should be peeled back and then wrapped around the drain line. If there is no room for conventional perforated pipe drain lines (i.e., back of lagging is directly on the property line), the drain mat may be connected to a flat panel-type connector such as *Mirafi Quickdrain*, or equivalent, that will connect the drain mats to solid drain pipes that can travel through the wall footing and to a sump pump or other suitable discharge point.

Soldier Pile Design Parameters

For a cantilever condition supporting a level ground surface, the shoring systems may be designed using the following soil parameters:

Minimum Soldier Pile Diameter:	24”
Vertical Load Resistance (Existing Fill and Older Alluvium)	
Skin Friction:	300 psf (applied to piles at least 10 feet deep)
End Bearing:	4000 psf (minimum 10 feet embedment).
Lateral Load Resistance(1)(2):	
Existing Fill and Older Alluvium:	250 psf/per foot of pile depth

(1) Disregard the upper 2 feet of lateral load resistance due to possible soil disturbance during drilling.
(2) Can be applied over 2 pile diameters provided piles are spaced at least 2 pile diameters, center to center. (e.g. 500psf/ft. of pile diameter per foot of depth). For permanent soldier piles that are embedded in structural concrete, the pile diameter may be assumed to be equivalent to the diameter of the concrete column.

Point of Fixity:	To be determined by structural engineer but may be assumed to be approximately 2 feet below the adjacent grade.
Lateral Earth Pressures:	35 pcf (Active – Cantilever) 50 pcf (At-Rest – Restrained)

The shoring should also be designed to withstand any surcharge pressures from nearby structures or from stockpiled soils or construction materials. Furthermore, the excavation should be constructed in accordance with California Occupational Safety and Health Administration (CalOSHA) regulations and guidelines.

Since there are nearby residences within the adjacent property to the north; it is recommended that the at-rest or restrained pressure be used to restrict the soldier pile wall system lateral movement along the north side of the site to less than 0.5 inches.

In addition, the shoring elements installed along the north side of the property should be monitored for movement as described in the following section of this report.

PRE-CONSTRUCTION SURVEY AND VIBRATION MONITORING

It is recommended that a pre-construction condition survey of the existing improvements within the adjacent properties be performed to help reduce the risk of future conflicts or litigation. This would involve visual inspection, photo and video documentation, and baseline survey points of any existing walls, fences, and hardscape features.

The proposed grading and construction and installation of shoring is likely to create vibrations in the adjacent properties due to activities such as demolition of existing structures, over-excavation and re-compaction of the onsite soils, and installation of deep piles. We recommend vibrations be monitored along the property lines and within the adjacent properties to reduce the risk of damage to existing buildings and defend against potential future claims. It should be noted that GMU can perform both pre-construction surveys and vibration monitoring services for the project.

POST-GRADING CONSIDERATIONS

SURFACE DRAINAGE DESIGN AND MAINTENANCE

Design of surface drainage is outside GMU's purview and should be designed and confirmed by the project civil engineer. Since the site is underlain by moderately expansive soils, surface drainage should be designed in accordance with Section 1804.4 of the 2022 CBC and Section R401.3 of the 2022 CRC. Surface drainage should be carefully controlled to prevent runoff over graded slope surfaces and ponding of water on flat pad areas. Positive drainage away from graded slopes and pad areas is essential to reduce the potential for erosion or saturation. Maintaining positive drainage of all landscaping areas along with avoiding over-irrigation will help minimize the possibility of "perched" groundwater accumulating slightly below the graded surfaces.

SOIL MOISTURE CONTROL, IRRIGATION, AND LANDSCAPING

The on-site soils may experience volume change (both expansion and contraction) in response to changes in moisture. Future planting, irrigation, landscaping, and maintenance should therefore strive to maintain a uniform soil moisture content that is similar to the moisture content at which the fills were placed. Over-irrigation should be avoided; furthermore, the fills should not be allowed to become excessively dry or saturated. Planter areas placed adjacent to building foundations are not recommended. If planter areas are proposed up against building foundations, irrigation should be carefully controlled. A watering program that maintains a uniform, near optimum moisture condition in the soils should be implemented for the landscape areas. Overwatering and subsequent saturation of the soils will cause excessive soil expansion and heave and, therefore, should be avoided. On the other hand, allowing the soils to dry out will cause excessive soil shrinkage.

As an alternative to a conventional irrigation system, drip irrigation that maintains constant moisture conditions is strongly recommended for all planter areas. The owner is advised that all drainage devices should be properly maintained throughout the lifetime of the development. Plants known to have excessive root systems should also be avoided near structural improvements as they can cause heave conditions. Conversely, the root systems can also dry out the soils and cause excessive soil shrinkage below adjacent footings or slabs. Drought-resistant and maintenance-free plant species are recommended.

UTILITY TRENCH BACKFILL CONSIDERATIONS

General

New utility line pipeline trenches (greater than 2-feet deep) should be backfilled with both select bedding materials beneath and around the pipes (pipe zone) and compacted soil above the pipe bedding. Recommendations for the material types to be used and their proper placement are provided in the following sections.

Pipe Zone (Bedding and Shading)

The pipe bedding materials should extend to at least 12 inches above the crown of pipes that are 8 inches in diameter or greater. Pipes less than 8 inches in diameter should be covered with at least 6 inches of pipe bedding materials. Pipe bedding should consist of clean sand with a sand equivalent (SE) of at least 20. Pipe bedding should also meet the minimum requirements of the County of Orange. If the requirements of the County are more stringent, they should take precedence over the geotechnical recommendations. Sufficient laboratory testing should be performed to verify the bedding meets the minimum requirements of the Greenbook and County grading codes.

Based on our subsurface exploration and knowledge of the onsite materials, the soils that will be excavated from the pipeline trenches will not meet the recommendations for pipe bedding materials; therefore, imported materials will be required for pipe bedding.

Granular pipe bedding material having a sand equivalent of 30 or greater may be placed in thicknesses not exceeding 3 feet and then sufficiently jetted in place. With proper techniques, jetting of SE 30 sand is not expected to have an adverse impact on existing site soils. The top of the jetted sand should be tamped with hand operated compaction equipment prior to placing the trench backfill.

If the sand has a sand equivalent of less than 30 but greater than 20, the sand cannot be properly jetted and should be placed in maximum lifts of approximately 12 inches, moisture conditioned, and then tamped with hand operation compaction equipment.

Trench Backfill

All existing soil material within the limits of the pipeline alignment are considered suitable for use as trench backfill above the pipe bedding zone if care is taken to remove all significant organic and other decomposable debris, moisture condition the soil materials as necessary, and separate and selectively place and/or stockpile any inert materials larger than 6 inches in maximum diameter.

Imported soils are not anticipated for backfill since the on-site soils are suitable. However, if imported soils are used, the soils should consist of clean, granular materials with physical and chemical characteristics similar to those described herein for on-site soils. Any imported soils to be used as backfill should be evaluated and approved by GMU prior to placement.

Soils to be used as trench backfill should be moistened, dried, or blended as necessary to achieve a minimum of 3 to 4 percentage points over optimum moisture content, placed in loose lifts no greater than 8 inches thick, and mechanically compacted/densified to at least 90% relative compaction as determined by ASTM Test Method D 1557. Jetting is not permitted in the trench zone.

BUILDING AND FOUNDATION DESIGN

STRUCTURE SEISMIC DESIGN

The average shear wave velocity for the upper 100 feet of subsurface soils (V_{s30}) within the subject site is estimated to be between 1,200 and 2,500 feet per second which corresponds to a "stiff" soil profile. Therefore, based on this soil profile, the site should be designated as default Site Class D. The seismic design coefficients based on ASCE 7-16 and 2022 CBC are listed in the following table.

2022 CBC and ASCE 7-16 Seismic Design Parameters

Seismic Item	Design Value	ASCE 7-16 and 2022 CBC Reference
Site Class based on soil profile (ASCE 7-16 Table 20.3-1)	D[(a)]	ASCE 7-16 Table 20.3-1
Short Period Spectral Acceleration S_s	1.178[(a)]	CBC Figures 1613.2.1 (1-10)
1-sec. Period Spectral Acceleration S_1	0.423[(a)]	CBC Figures 1613.2.1 (1-10)
Site Coefficient F_a (2022 CBC Table 1613.2.3(1))	1.200[(a)]	CBC Table 1613.2.3 (1)
Site Coefficient F_v (2022 CBC Table 1613.2.3(2))	1.877[(a)]	CBC Table 1613.2.3 (2)
Short Period MCE* Spectral Acceleration S_{MS} $S_{MS} = F_a S_s$	1.413[(b)]	CBC Equation 16-20
1-sec. Period MCE Spectral Acceleration S_{M1} $S_{M1} = F_v S_1$	0.794[(b)]	CBC Equation 16-21
Short Period Design Spectral Acceleration S_{DS} $S_{DS} = 2/3 S_{Ms}$	0.942[(b)]	CBC Equation 16-22
1-sec. Period Design Spectral Acceleration S_{D1} $S_{D1} = 2/3 S_{M1}$	0.529[(b)]	CBC Equation 16-23
Short Period Transition Period T_S (sec) $T_S = S_{D1}/S_{DS}$	0.562[b)]	ASCE 7-16 Section 11.4.6
Long Period Transition Period Tl (sec)	8[(b)]	ASCE 7-16 Figures 22-14 to 22-17
MCE[(c)] Peak Ground Acceleration (PGA)	0.505[(a)]	ASCE 7-16 Figures 22-9 to 22-13
Site Coefficient F_{PGA} (ASCE 7-16 Table 11.8-1)	1.200[(a)]	ASCE 7-16 Table 11.8-1
Design Earthquake ($PGA_D = S_{DS}/2.5$)	0.38g	ASCE 7-16 ; 2002 CBC
Modified MCE[(c)] Peak Ground Acceleration (PGA_M)	0.606[(a)]	ASCE 7-16 Equation 11.8-1
Seismic Design Category	D[(b)]	ASCE 7-16 Tables 11.6.1 and 11.6.2

[(a)] Design Values Obtained from USGS Earthquake Hazards Program website that are based on the ASCE-7-16 and 2022 CBC and site coordinates of N33.5003° and W117.6629°.
[(b)] Design Values Determined per CBC Equations 16-20 through 16-23 and ASCE Tables 11.4-2 and 11.6.1-2.
[(c)] MCE: Maximum Considered Earthquake.

Since the Site Class is designated as D and the S_1 value is greater than or equal to 0.2, the 2022 CBC requires either a site-specific ground motion hazard analysis per Section 21.2 of ASCE 7-16 or the application of Exception 2 of Section 11.4.8 of ASCE 7-16. Exception 2 states that a site-specific ground motion hazard analysis is not required provided that the value of the seismic response coefficient, Cs, is conservatively calculated by the project structural engineer using Eqn. 12.8-2 of ASCE 7-16 for values of $T \leq 1.5Ts$ and taken as equal to 1.5 times the value computed in accordance with either Eqn. 12.8-3 for $T_L \geq T > 1.5Ts$ or Eqn. 12.8-4 for $T > T_L$.

It should be recognized that much of southern California is subject to some level of damaging ground shaking due to movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2022 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

FOUNDATION RECOMMENDATIONS FOR: PODIUM STRUCTURE, BUILDING 5 AND MAJOR RETAINING WALLS

Deep (Pile) Foundation System

Due to potential compression of the old alluvial materials under the loads of the proposed podium structure, and the close proximity of the proposed podium structure to the adjacent property lines, the foundations of the proposed podium structure, Building 5, and major retaining walls should be supported on deep foundations that extend at least 5 feet into bedrock (tip elevation of at least 56 feet above mean sea level). Small diameter (i.e., 12- to 18-inch diameter) Augered Cast-in-Place (ACIP) concrete piles are recommended due to relatively shallow groundwater conditions at the subject site. Driven piles were not considered due to their induced noise and vibrations. The following sections provide design parameters for 12- and 18-inch-diameter Augered Cast-in-Place (ACIP) concrete piles based on our recent analysis.

- Axial Capacity of Deep Foundations:
    - Axial compressive capacities for 12- and 18-inch-diameter ACIP piles are presented within Plate 1 of Appendix D.
    - The axial capacities were estimated based on SHAFT by Ensoft, Inc.
    - ACIP pile capacities shown on Plate 1 of Appendix D are based on skin friction resistance with a safety factor of 2 below the compressible/liquefiable layer.
    - The axial capacities on Plate 1 of Appendix D are for individual piles. A group of piles with pile spacing less than 3 diameters will result in a reduction in axial capacity.
- Settlement:
    - Total static settlement is expected to be less than 1 inch and differential settlement is expected to be less than ½ inch between individual pile caps.
    - Total seismic settlement is estimated to be less than ½ of an inch.
- Lateral Capacity of Deep Foundations:
    - Lateral load analyses for single piles were conducted for pile top deflection of ¼-, ½-, 1-, and 2-inch for a free-head condition.
    - The lateral capacity was estimated based on LPILE by Ensoft, Inc.
    - The lateral pile capacities are summarized in the following tables. No safety factor was applied to the lateral load analyses.

Lateral Pile Capacity – 12-inch Diameter ACIP Piles

Pile Head Condition	Pile Head Deflection (in)	Shear Force at Pile Top (kips)	Maximum Bending Moment M_{max} (kips-in)	Depth to M_{max} (ft)	Minimum Pile Embedment Depth in Bedrock (ft)
Free Head	0.25	10.3	194.9	2-8	5
Free Head	0.5	17.5	359.1	2-8	5
Free Head	1	26.7	642.0	2-8	5
Free Head	2	36.7	1079.0	2-8	5

Lateral Pile Capacity – 18-inch Diameter ACIP Piles

Pile Head Condition	Pile Head Deflection (in)	Shear Force at Pile Top (kips)	Maximum Bending Moment M_{max} (kips-in)	Depth to M_{max} (ft)	Minimum Pile Embedment Depth in Bedrock (ft)
Free Head	0.25	18.9	538.5	2-12	5
Free Head	0.5	29.9	925.7	2-12	5
Free Head	1	47.2	1578.5	2-12	5
Free Head	2	66.9	2644.3	2-12	5

- The lateral pile analyses charts for both 12-inch and 18-inch single piles are presented on Plates 2 and 3 of Appendix D.
- The lateral capacities above are for individual piles. A group of piles with pile spacing less than 7 diameters will result in a reduction in lateral capacity.

  - For the general design of foundations composed of groups of drilled shafts, the above lateral capacities should be multiplied by a reduction factor (i.e., p-multiplier P_m). P-multiplier values for design by row position are summarized in the table below (from California Amendments to AASHTO LRFD Bridge Design Specifications, Table 10.7.2.4-1):

Lateral Pile Capacity - Recommended P-Multiplier, *Pm*

Pile Center-to-Center Spacing (in the Direction of Loading)	P-multiplier, *Pm*		
	Leading Row	Second Row	Third Row and Beyond
2.0B	0.60	0.35	0.25
3.0B	0.75	0.55	0.40
5.0B	1.0	0.85	0.70
7.0B	1.0	1.0	0.90

If the loading direction for a single row of piles is perpendicular to the row, a P-multiplier of less than 1.0 shall only be used if the pile spacing is 4B or less. A P-multiplier of 0.8, 0.9, and 1.0 shall be used for pile spacing of 2.5B, 3B, and 4B, respectively.

- Groundwater:
  - Groundwater was encountered during our subsurface investigation approximately 17 feet below the existing grade. This depth to groundwater can be expected during installation of the ACIP piles.
  - Groundwater was considered in the vertical and lateral pile capacities.

- ACIP Construction:
  - Proper construction of ACIP piles is critical to ensure satisfactory foundation support. Care in drilling and placement of steel and concrete will be essential to the quality of the piles. Drilling of the ACIP pile borings should be observed and accepted by GMU representative. The pile boring should be drilled to the final depth in one continuous process using a continuous flight auger to provide continuous support of the drill hole. Concrete placement shall be continuous from the bottom of the shaft to the top elevation of the shaft through the hollow center of the drilling auger to provide continuous support of the hole as the auger is withdrawn from the hole.

Parking Structure Slab

Determination of the design slab thicknesses and reinforcement for the parking structure slab should be determined by the structural engineer based on vehicle loading and a modulus of subgrade reaction of 50 pci. However, the following minimum design recommendations are provided from a geotechnical perspective.

Slab and Base Thickness:	Minimum - 6-inch-thick slabs underlain by 6 inches of ½ inch or larger clean gravel or crushed rock containing no more than 10 percent of material that passes through the No. 4 sieve.
Deepened Edge:	The perimeter of the garage slab should be provided with an 18-inch deepened edge to act as a cutoff against moisture intrusion below the slab.
Slab Reinforcement:	Minimum - No. 4 bars at 18 inches on center, both ways.
Slab Subgrade Moisture Content:	2 percentage points over optimum to a minimum depth of 18 inches.

Building 2 and Building 5 Slabs

Determination of the design slab thicknesses and reinforcement for slabs of Buildings 2 and 5 should be determined by the structural engineer based on a modulus of subgrade reaction of 50 pci. However, the following minimum design recommendations are provided from a geotechnical perspective.

Slab Thickness:	Minimum - 5-inch-thick slabs.
Slab Reinforcement:	Minimum - No. 4 bars at 18 inches on center, both ways.
Slab Subgrade Moisture Content:	2 percentage points over optimum to a minimum depth of 18 inches.

- Vapor retarders/barriers should be placed below the on-grade slabs of Buildings 3 and 6 in accordance with the following recommendations:
  - 15 Mil Stego® Wrap, Husky Yellow Guard®, or equivalent Constructed below entire floor areas of the foundation system.
  - Installed per manufacture's specifications as well as with all applicable recognized installation procedures such as ASTM E 1643-98.
  - Joints between the sheets and the openings for utility piping should be lapped and taped. If the retarder/barrier is not continuously placed across footings/ribs, the retarder/barrier should, as a minimum, be lapped into the sides of the footing/rib trenches down to the bottom of the trench.
  - Punctures in the vapor retarder/barrier should be repaired prior to concrete placement.

- Concrete may be placed directly on the vapor retarder/barrier or the moisture vapor barrier may be overlain by at least 2 inches of clean sand to promote uniform curing of the concrete (see *Note* below).
  - *Note: some structural engineers, geotechnical professionals and concrete experts consider the clean sand as a layer that can entrap excess water during concrete placement which later migrates up through the slab and adversely impacts moisture-sensitive floor coverings. This potential for future upward moisture intrusion into the concrete slab can be reduced by eliminating the sand layer and placing the concrete directly on the moisture vapor barrier. However, if this sand layer is eliminated, appropriate concrete curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Steps will also need to be taken to prevent puncturing of the vapor barrier during concrete placement*

- **Capillary Break -** The vapor barrier should be underlain by a capillary break consisting of a 4-inch-thick layer of ½ inch or larger clean gravel or crushed rock containing no more than 10 percent of material that passes through the No. 4 sieve. The upper 2 inches of the capillary break can be replaced with clean sand if desired from a concrete curing perspective.

BUILDING 1A FOUNDATION DESIGN RECOMMENDATIONS

General

Building 1A is proposed within approximately 6 inches of the existing historic Casa Manuel Garcia building. Therefore, as described previously, only a limited amount of corrective grading can be performed to avoid causing any distress to the sensitive adobe building. Furthermore, the proposed building will be of glass and steel frame construction and will be sensitive to any differential movement.

As a result, it is recommended that this building be supported on a rigid mat slab that will be stiff enough to bridge potential future soil settlement.

The methods used in the design and construction of the mat slab foundation system should conform to all applicable and current codes, ordinances, and standards. The allowable limits selected for foundation deflection due to any differential soil settlement should be coordinated with the architect and structural engineer responsible for the design of the structure framing and roof systems such that the deflection will not cause excessive distress to those systems or to interior and exterior walls and roof.

General Foundation Design Parameters

Bearing Material: Existing Older Alluvial Materials

Bearing Value: 1500 psf
- One-third increase for wind or seismic loading.

Coefficient of Friction: 0.35
- One-third increase for wind or seismic loading.

Passive Resistance: 250 psf/ft of depth to 2500 psf maximum.
- Disregard upper 6 inches
- One-third increase for wind or seismic loading.

Modulus of Subgrade Reaction: 50 pci

General Settlement

The Building 1A structure proposed within the site should be designed for a maximum differential settlement of 1 inch over a horizontal distance of 40 feet.

Mat Slab Foundation Design Recommendations

It is considered acceptable from a geotechnical point of view to design the foundation system of Building 1A using a strengthened non-pre-stressed uniform mat slab that is designed to resist the anticipated differential soil volume changes. Determination of mat slab thickness and reinforcement should be determined by the structural engineer based on the anticipated differential settlement and a modulus of subgrade reaction of 50 pci. However, the following minimum design recommendations are provided.

Mat Slab Thickness:	Minimum 10-inch-thick slab.
Slab Reinforcement:	No. 4 bars at 18 inches on center, both ways.
Slab Subgrade Moisture Content:	2 percentage points over optimum to minimum depth of 18 inches.
Perimeter Thickened Edge:	Minimum 12-inch embedment from lowest adjacent final grade. (Excluding the side of the slab adjacent to the historic adobe.)

Vapor Retarder/Barrier System

Vapor retarders/barriers should be placed below the on-grade slab of Building 1A in accordance with the following recommendations:

- 15 Mil Stego® Wrap, Husky Yellow Guard®, or equivalent.
  - Constructed below entire floor area of the foundation system.
  - Installed per manufacture's specifications as well as with all applicable recognized installation procedures such as ASTM E 1643-98.
  - Joints between the sheets and the openings for utility piping should be lapped and taped. If the retarder/barrier is not continuously placed across footings/ribs, the retarder/barrier should, as a minimum, be lapped into the sides of the footing/rib trenches down to the bottom of the trench.
  - Punctures in the vapor retarder/barrier should be repaired prior to concrete placement.

- Concrete may be placed directly on the vapor retarder/barrier or the moisture vapor barrier may be overlain by at least 2 inches of clean sand to promote uniform curing of the concrete (see *Note* below).

Note: some structural engineers, geotechnical professionals and concrete experts consider the clean sand as a layer that can entrap excess water during concrete placement which later migrates up through the slab and adversely impacts moisture-sensitive floor coverings. This potential for future upward moisture intrusion into the concrete slab can be reduced by eliminating the sand layer and placing the concrete directly on the moisture vapor barrier. However, if this sand layer is eliminated, appropriate concrete curing methods must be *implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Steps will also need to be taken to prevent puncturing of the vapor barrier during concrete placement.*

Capillary Break

The vapor barrier should be underlain by a capillary break consisting of a 4-inch-thick layer of ½ inch or larger clean gravel or crushed rock containing no more than 10 percent of material that passes through the No. 4 sieve. The upper 2 inches of the capillary break can be replaced with clean sand if desired from a concrete curing perspective.

Water Vapor Transmission

As discussed above, placement of a moisture vapor retarder/barrier below the on-grade slab areas is recommended. This moisture vapor retarder/barrier recommendation is intended only to reduce moisture vapor transmissions from the soil beneath the concrete and is consistent with the current standard of the industry for residential construction in Southern California. It is not intended to provide a "waterproof" or "vapor proof" barrier or reduce vapor transmission from sources above the retarder. Sources above the retarder include any sand placed on top of the retarder (i.e., to be determined by the project structural designer) and from the concrete itself (i.e., vapor emitted during the curing process). The evaluation of water vapor from any source and its effect on any aspect of the proposed living space above the slab (i.e., floor covering applicability, mold growth, etc.) is outside our purview and the scope of this report.

Floor Coverings

Prior to the placement of flooring, the floor slabs should be properly cured and tested to verify that the water vapor transmission rate (WVTR) is compatible with the flooring requirements.

CONCRETE

Based on the results of our laboratory testing, the onsite soils have negligible sulfate concentrations but have high elevated levels of chlorides. Therefore, we recommend using the following:

<u>Structural Elements</u> (i.e., foundations, walls, etc.)

- Cement Type: V
- Maximum Water Cement Ratio: 0.50
- Minimum Strength: 4,000 psi
- Reinforcement steel should be covered by at least 3 inches of concrete where exposed to soil.

Consideration should also be given to including a corrosion inhibiting additive within the concrete mix. These recommendations will serve to minimize the potential of water and/or vapor transmission through the concrete, minimize the potential for physical attack to concrete from non-sulfate based salts, and add additional protection to embedded steel reinforcement. In addition, wet curing of the concrete as described in ACI Publication 308 should be considered.

<u>Non-structural Elements</u> (i.e., flatwork, pavement, etc.)

Non-structural onsite concrete (i.e. walkways, patios, driveways, etc.) may be designed with concrete strengths that are determined by the engineer or designer responsible for that particular site improvement. Specific flatwork concrete requirements are provided in Appendix E.

The aforementioned recommendations in regards to concrete are made from a soils perspective only. Final concrete mix design as well as any concrete testing is outside our purview. All applicable codes, ordinances, regulations, and guidelines should be followed in regard to designing a durable concrete with respect to the potential for detrimental exposure from the on-site soils and/or changes in the environment.

CORROSION PROTECTION OF METAL STRUCTURES

The on-site soils are expected to be severely corrosive to metals. Consequently, metal structures in direct contact with the soil (i.e., underground metal conduits, pipelines, metal signposts, metal door frames, etc.) and/or in close proximity to the soil (wrought iron fencing, post-tension cable end caps, weep screeds, etc.) may be subject to corrosion.

Special protection measures should be implemented to adequately protect the improvements noted above. Use of special coatings or cathodic protection around buried metal structures has been shown to be beneficial in reducing corrosion potential. It should also be noted that a significant potential for copper piping corrosion has been detected in various areas of south Orange County where similar soils are present. The potential for corrosion of ferrous metal reinforcing elements embedded in structural concrete will be minimized by use of the recommended maximum water/cement ratio for concrete.

The above discussion is provided for general guidance regarding the anticipated corrosiveness of the on-site soils to typical metal structures used for construction. Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements is beyond our purview. If detailed testing is required, a corrosion engineer should be consulted to perform the testing and develop appropriate mitigation measures.

SITE WALL/RETAINING WALL DESIGN AND CONSTRUCTION CRITERIA

General

Remedial grading will be performed below proposed exterior free-standing walls and retaining walls that are not a part of the podium structure. For these walls, the criteria contained in the following two sections may be used for their design and construction.

The retaining walls that are part of the podium structure will be supported on deep auger-cast piles in accordance with the recommendations provided for the podium structure foundations.

Soil Parameters (Exterior Walls with Remedial Grading)

Bearing Material: New Compacted Fill

Allowable Bearing Value: 3000 psf, based on an 18 inch deep by 12 inch wide footing:
- May be increased 10% for each additional foot of width and by 20% for each additional foot of depth to a maximum of 4000 psf).
- One-third increase for wind or seismic loading.

Coefficient of Friction: 0.35
- One-third increase for wind or seismic loading.

Passive Resistance: 300 psf/ft of depth (level ground)
- Disregard upper 6 inches (level ground)
- One-third increase for wind or seismic loading.
- May be combined with friction without reduction.

Minimum Footing Design Recommendations (Exterior Walls with Remedial Gading)

Minimum Foundation Width: 24 inches (spread footing)

Minimum Foundation Depth: 18 inches below lowest adjacent.

Minimum Reinforcement: Four #4 bars (two at top and two at bottom).

Retaining Wall Lateral Earth Pressures

- Static Lateral Earth Pressures: 40 pcf (Active – Level Backfill).
60 pcf (Active – 2:1 Backfill).
55 pcf (At-Rest – Level Backfill).
70 pcf (At-Rest – 2:1 Backfill).

The unrestrained values are applicable only when the walls are designed and constructed as cantilevered walls allowing sufficient wall movement to mobilize active pressure conditions. This wall movement should not be less than 0.01 H (H = height of wall) for the unrestrained values to be applicable. For walls where the movement must be held to less than 0.01H, the restrained or at-rest pressures should be used.

Per the 2022 CBC, the following seismic lateral earth coefficients and lateral earth pressures should be utilized for walls with a retaining height <u>in excess of 6 feet.</u> These values are based on a "design level ground" acceleration (PGA) equivalent to S_{DS}/2.5 (0.942/2.5)= 0.38g.

- Seismic Lateral Earth Coefficient: K_H = (0.5)PGA = (0.5)0.38g = 0.19g (Active)
- Seismic Earthquake Pressures (EFP): 15 pcf (normal triangular pressure distribution added to either active or at-rest earth pressure)
- Unit Weight of Backfill: 120 pcf

Construction Joints

Construction joints should be implemented and designed by a structural engineer. As a minimum, construction joints should be provided in free-standing walls at a maximum interval of 20 feet and at all angle points and other locations where differential movement is likely to occur. Joints to consist of a clear vertical break of all masonry materials.

Waterproofing

The back side of all retaining walls should be waterproofed down to and onto the top of the foundation prior to placing subdrains or backfill. The design and selection of the waterproofing system is outside the scope of our report and is outside our purview.

Wall Backfill and Drainage

See the Retaining Wall Construction Detail diagram (Plate E-1) contained within Appendix E for backfill and drainage requirements.

PEDESTRIAN FLATWORK

Pedestrian flatwork areas such as concrete walkways and patios and interlocking pavers should be designed in minimum accordance with the recommendations contained in Table 1 of Appendix F. Aggregate base may consist of either CAB or CMB as per current Greenbook standards and should be compacted to at least 95% relative compaction. The underlying subgrade soils should be compacted to at least 90% relative compaction.

For concrete flatwork, it should be noted that the recommendations contained in the table are largely to improve "post-cure" performance relative to expansive soils. All other aspects of concrete design (i.e., concrete mix design, curing, type, and location of joints, etc.) as well as concrete inspection of any kind is outside our purview. It is recommended that the final flatwork design be reviewed by our office prior to bidding.

Even with extensive crack control and expansive soil mitigation, all concrete flatwork will crack and move (i.e., lift) to some degree due to a variety of mechanisms. Consequently, concrete cracking and movement and hence concrete repair/replacement should be anticipated.

VEHICULAR PAVEMENTS

Asphalt Pavement Design

The following asphalt pavement structural sections are considered applicable for the design of parking stalls, drive isles, and heavy load areas (i.e. trash enclosure pads, fire access lanes and H-20 loading areas). The structural sections are based on an assumed R-value of 15 and traffic indices of 4.5 for the parking stalls, 5.5 for the drive aisles, and 6.0 for the heavy load areas.

Structural Pavement Sections

Location	Traffic Index	AC(in.)/AB (in.)	Alternate Section AC(in.)/AB (in.)
Parking Stalls	4.5	3/7.5	4/5
Light Duty Drive Isles	5.5	3/11	4/8.5
Heavy Load Areas	6.0	4/10.5	5/8

Aggregate base may consist of either CAB or CMB as per current Greenbook standards. The base materials (CAB or CMB) and asphalt concrete materials (AC) should be of a type meeting the minimum City of San Juan Capistrano and Greenbook standards. The AC and CAB or CMB materials should be compacted to at least 95% relative compaction. The underlying subgrade soils should be moisture conditioned to at least 2% above the optimum moisture content and compacted to at least 90% relative compaction to a depth of at least 12 inches.

Concrete Pavement Design

For light duty drive isles, we recommend a design section of 5 inches of Portland Cement Concrete (PCC) over 6 inches of Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB).

For heavy load areas (i.e. trash enclosure pads, fire access lanes and H-20 loading areas), we recommend a design section of 6 inches of Portland Cement Concrete (PCC) over 8 inches of Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB).

The CAB or CMB materials should meet current Greenbook and City of San Juan Capistrano requirements and should be placed at or above optimum moisture content and compacted to at least 95% relative compaction. The underlying subgrade soils should be moisture conditioned to at least 2% above the optimum moisture content and compacted to at least 90% relative compaction to a depth of at least 12 inches.

Interlocking Paver Design

As shown in Table 1 of Appendix F, for light duty drive isles, we recommend a design section of 80mm vehicular pavers over a 1- to 2-inch-thick levelling course of sand over 8 inches of Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB). For heavy load areas (i.e. trash enclosure pads, fire access lanes and H-20 loading areas), we recommend a design section of 80mm vehicular pavers over a 1- to 2-inch-thick levelling course of sand over 10 inches of Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB).

The CAB or CMB materials should meet current Greenbook and City of San Juan Capistrano requirements and should be placed at or above optimum moisture content and compacted to at least 95% relative compaction. The underlying subgrade soils should be moisture conditioned to at least 2% above the optimum moisture content and compacted to at least 90% relative compaction to a depth of at least 12 inches.

FUTURE PLAN REVIEW

GMU Geotechnical, Inc. (GMU) should review future project plans to check for conformance to the recommendations provided herein and to provide additional recommendations as needed. Specifically, GMU should review the following plans:

- Final Precise Grading Plans
- Building Foundation Plans
- Site Wall/Retaining Wall Plans
- Landscape Structure Plans

FUTURE IMPROVEMENTS

Any additional or future improvements that are not specifically addressed in this report or shown on the referenced grading plans should be subject to additional geotechnical evaluation and recommendations. Additional recommendations for future improvements can be provided upon request.

LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgments. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading and site construction will be identical to those observed, sampled, and interpreted during our study, or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property.

We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

Our conclusions and recommendations are based on the assumption that our firm will act as the geotechnical engineer of record during construction and grading of the project to observe the actual conditions exposed, to verify our design concepts and the grading contractor's general compliance with the project geotechnical specifications, and to provide our revised conclusions and recommendations should subsurface conditions differ significantly from those used as the basis for our conclusions and recommendations presented in this report. Since our conclusions and recommendations are based on a limited amount of current and previous geotechnical exploration and analysis, all parties should recognize the need for possible revisions to our conclusions and recommendations during grading of the project.

It should be further noted that the recommendations presented herein are intended solely to minimize the effects of post-construction soil movements. Consequently, minor cracking and/or distortion of all on-site improvements should be anticipated.

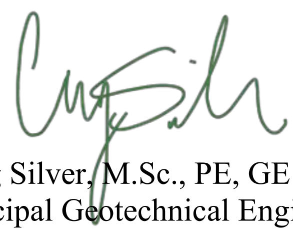
CLOSURE

We are pleased to present the results of our geotechnical investigation for this project. The Plates and Appendices that complete this report are listed in the Table of Contents.

If you have any questions concerning our findings or recommendations, please do not hesitate to contact us and we will be happy to discuss them with you.

Respectfully submitted,

David Hansen, M.Sc., PE, GE 3056
Associate Geotechnical Engineer



Greg Silver, M.Sc., PE, GE 2336
Principal Geotechnical Engineer

dwh/23-157-00R (04-30-24)

SITE SPECIFIC REFERENCES

1) *Geotechnical Engineering Investigation, Proposed Barns and Office Building, Oyharzabal Property, 31871 Camino Capistrano, San Juan Capistrano, CA*; prepared by Salem Engineering Group, Inc., dated October 29, 2021.

2) *Geotechnical Third-Party Review, Geo Review AC22-002, 31871 Camino Capistrano, California;* prepared by GEOCON West, Inc., dated March 16th, 2022.

3) *Response to Geotechnical Third-Party Review Comments, Geotechnical Engineering Investigation, Proposed Barns and Office Building, Oyharzabal Property, 31871 Camino Capistrano, San Juan Capistrano, CA*; prepared by Salem Engineering Group, Inc., dated August 12, 2022.

4) *Geotechnical Third-Party Review, Geo Review AC22-002, 31871 Camino Capistrano, California*; prepared by GEOCON West, Inc., dated August 22, 2022.

5) Our *Preliminary Geotechnical Update Report, Proposed Luxury Hotel, Oyharzabal Property, 31841-31871 Camino Capistrano, San Juan Capistrano, California*, dated December 15, 2023 (GMU Project 23-157-00).

6) Preliminary Grading & Drainage Plans, Oyharzabal Project, 31841-31871 Camino Capistrano, Sheets 1-12; prepared by C3 Civil Engineering, dated 12/15/2023

TECHNICAL REFERENCES

California Division of Mines and Geology, 2001, *Seismic Hazard Zone Report for the San Juan Capistrano 7.5-Minute Quadrangle, Orange County, California*, Seismic Hazard Zone Report 053.

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California Geologic Survey, 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*: California Department of Conservation, Division of Mines and Geology, Special Publication 117A, 98 p.

Morton, P.K., Edgington, W.L., and Fife, D.L., 1974, *Geology and Engineering Geologic Aspects of the San Juan Capistrano Quadrangle, Orange County, California*: California Division of Mines and Geology, Special Report 112.

Morton, P.K., Miller, R.V., and Evans, J.R., 1976, *Environmental Geology of Orange County, California*: California Division of Mines and Geology, Open File Report 79-8 LA.

Morton, P.K., and Miller, R.V., 1981, *Geologic Map of Orange County, Showing Mines and Mineral Deposits*: California Division of Mines and Geology, Scale: 1” = 4000’.